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Stream Restoration Design

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Cover photo: Streams and rivers are as complex as they are beautiful. A combination of the principles and analytical tools used in the fields of engineering, landscape architecture, geology, hydraulics, hydrology, ecology, and fluvial geomorphology are necessary to properly analyze and design stream and riverine projects.

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Preface

The management of streams is a continuing balance between what people want and what plants and animals need. In an ideal world, a stream can satisfy both—in reality, the balance is ephemeral, at best, as streams evolve and humans continue to imprint their desires on the adjoining or upland landscape. Intervention is often needed when the balance becomes so skewed that the function of streams for either people or nature is at risk.

Just as one would consult a doctor regarding an illness affecting the body's function, one should consult a hydraulic engineer, stream ecologist, geomorphologist, aquatic biologist, or other riparian specialist for the diagnosis or treatment of a stream disorder or problem. An inappropriate or poorly designed restoration project can worsen or broaden the disorder. Site-specific designs based on sound, scientific experience are needed to properly select the size, orientation, and location of stream restoration techniques. Effective designs also need to include appropriate management techniques that remove sources of disturbance, allow the design elements to function well together, and enhance the stream's ability for ecological regeneration.

In planning and designing solutions to some stream problems, simply modifying adjacent land and riparian management practices may be all that is needed to improve degraded stream conditions. Streams are integrators of all upland problems, so some stream conditions are symptomatic of mismanagement of their surrounding watershed(s). In these cases, solutions may lie not only in restoring the stream directly, but in changing land uses and management practices throughout the entire watershed.

In a response to heightened environmental sensitivity, softer approaches are increasingly preferred by permitting agencies and the public. Green or natural engineering is making a strong foothold in the restoration of streams. One green technique, streambank soil bioengineering, has been used for centuries, historically with rock, wood, and native vegetation and now including developed plant materials and geosynthetics. Several large soil bioengineering projects were installed on United States streams (and rivers) in the 1930s, but these labor-intensive methods fell from favor largely until the 1960s. Many of the 1960s projects were not designed and constructed for habitat and landscape enhancement but primarily for structural controls. Przedwojski, Blazejewski, and Pilarczyk (1995) noted that the "application of living materials in civil engineering, including river training, is not as well managed as … earth and concrete structures." In the late 1980s to the present, stream restoration practitioners began to fully embrace green engineering and how-to guides and a one-size-fits-all design approach proliferated. New products and materials emerged, such as geosynthetics, specialized planting equipment, as well as selection and release of improved plant species for riparian areas. Engineers, hydrologists, and biologists also recognized the importance of including other disciplines such as fluvial geomorphologists to achieve comprehensive restoration goals.

Though there has clearly been impressive and needed movement toward green stream restoration, a paucity of supporting design research, engineering principles and scholarship exists. Robinson (2002) found that natural stream techniques had not been proven to the degree that conventional

riprap has been, and, thus, often appeared more risky to landowners, permitters, and designers.

The state-of-the-art is still developing, as well as the supporting science and technology. This handbook marks a beginning. It contains tools and guidance to support stream restoration activities—specifically tools to use in designing restoration solutions. The focus of this handbook is on the how-to. It provides the user with specific tools to perform analyses and designs. This handbook presents engineering and ecological assessment and design tools that are applicable to a wide range of stream restoration work. The information contained herein represents both green techniques and structural approaches.

Please note that this handbook makes no endorsement of one particular approach over another and is not intended as a requirement document for purposes of funding or permitting. The guidance provided can be used to design and implement some of the techniques used in stream restorations. It is anticipated that as new methods are validated, they will be added to this guidance document or a supporting Web site.

Acknowledgments

Numerous people have provided source information, as well as expert reviews and comments. Their contributions are acknowledged and very much appreciated. The experience that they have helped document in this handbook will promote the science and art of stream restoration design.

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Executive Summary

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) has worked with private landowners since the 1930s to implement conservation plans that address their farm or ranch natural resource needs. Those plans often include voluntary measures to address problems associated with streams and, increasingly, to enhance habitat functions important to aquatic species of concern. In short, the agency works with the public in managing streams to meet their ecological needs and the needs of people who work and live nearby. NRCS technical assistance is based on science-based solutions that result in installed projects that range from relatively simple streambank protection to more complex plans covering watershed-scale stream and riparian restoration efforts involving multiple partners and agencies. There is a recognized need for the agency's technical guidance documents to be consistent, accurate, available, and current with stream-related innovations and improvements.

In 1998, an NRCS-led effort resulted in 15 Federal agencies producing the document entitled "Stream Corridor Restoration: Principles, Processes, and Practices" (NEH 653). Diverse groups of users, both nationally and internationally, are using this interagency document to plan stream corridor restoration projects. However, this document stopped short of providing specific design guidance tools that are required as the NRCS increasingly becomes involved in stream restoration projects that cover the full range of treatments, from natural to management to structural. These stream restoration projects require designs that can best be developed from a balance of skills in both engineering and ecology. This extensive document was assembled to ensure NRCS specialists and field personnel have the best design tools available.

The primary emphasis of this handbook is on how-to techniques; theory is only briefly discussed. Concise outlines, tables, and formulas are presented. While primarily an NRCS effort, stream and aquatic ecology experts from a variety of Federal, state, and local agencies, as well as private consultants and universities, contributed to the content.

Much of the information herein is not new; it is compiled from a rich system of existing guides used to treat or restore streams. Many of these legacy guides, however, consist of narrowly focused technologies primarily for engineered solutions, constructed earth channels, or bank armor, and do not fully integrate ecological, biological, or geomorphic criteria. NRCS developed guidance in the late 1980s and early 1990s for soil bioengineering practices, but these documents are dated and do not provide a system-based or holistic approach to analysis and design. Other information written and published by others, both inside and outside NRCS, provides guidance for balancing ecological goals with appropriate combinations of management and engineering designs. Guidance, tools, and procedures contained in this design handbook are those currently available for use—no additional research or development was specifically fostered for this effort. As appropriate, information was updated, reformatted, and edited to fit within the handbook's structure.

This handbook does not prescribe specific design procedures, nor does it assume that all stream restorations or rehabilitations will require structural

treatments. Successful and sustainable stream work requires a thorough, contextual understanding of dynamic physical, chemical, and biological processes; risks and limitations; and range of applications for appropriate tools. It also involves weighing the wide array of management and intervention options that can be used to attain the desired and achievable condition. The overall stream restoration planning process should result in clear and obtainable goals, which should be implemented through appropriate designs. The best-designed treatment cannot make up for rushed, cookie-cut, or poorly defined plans.

In summary, this assembly of tools will help designers achieve a balance of management and engineering techniques. It does this by providing NRCS and other stream practitioners with principles and methods to restore functions in ways that enhance the natural abilities of streams and stream corridors to self-repair and adjust to variations in sediment and water loads without substantially compromising the needs and goals of the adjacent landowners.

Part 654 Stream Restoration Design

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Terminology

This section provides a ready reference for some of the words and phrases used in the field of stream restoration design to the section or sections of the handbook where it is most thoroughly addressed. Other institutional and legal definitions exist for these terms, and many other definitions may exist in published sources. The definitions provided here are in the context of the scope and content of this handbook.

Adaptive management	An approach to management that addresses changing site and project conditions, as well as taking into affect new knowledge; a management approach that incorporates monitoring of project outcomes and uses the monitoring results to make revisions and refinements to ongoing management and operations actions.	Ch. 16
Adfluvial fish	Species that hatch in rivers or streams, migrate to lakes as juveniles to grow, and return to rivers or streams to spawn.	TS 14N
Aggradation	Long-term sediment deposition occurs on the bed of a channel; opposite is degradation or bed erosion.	Ch. 13
Alaska Steeppass Fishway	See Denil Fishway.	TS 14N
Alignment	Planform of a channel.	Ch. 12
Allowable shear stress design method	A threshold channel design technique whereby channel dimensions are selected so that the average applied grain bed shear stress is less than the allowable shear stress for the boundary material.	Ch. 8
Allowable velocity	The greatest mean velocity that will not cause the channel boundary to erode.	Ch. 8
Allowable velocity design method	A threshold channel design technique whereby channel dimensions are selected so that the applied velocity during design conditions is less than the limiting velocity of the channel boundary.	Ch. 8
Alluvial channel	Streams and channels that have bed and banks formed of material transported by the stream under present flow conditions. There is an exchange of material between the inflowing sediment load and the bed and banks of an alluvial channel.	Ch. 7
Alluvial channel design	A design approach whereby a channel configuration is selected so that it is in balance with the inflowing sediment and water discharges.	Ch. 9
Amphidromous fish	Species that move between fresh and salt water during some part of their life cycle, but not for breeding.	TS 14N
Anadromous fish	Species that incubate and hatch in freshwater, migrate to saltwater as juveniles to grow, and return to freshwater as adults to spawn.	TS 14N

Analogy design method	A design approach that is based on the premise that conditions in a reference reach with similar characteristics and watershed conditions can be copied or adapted to the project reach.	Ch. 7 Ch. 9
Analytical design method	The use of bed resistance and sediment transport equations to calculate channel design variables.	Ch. 7
Anastomosed channels	Multiple-thread streams. The multiple channels tend to be narrow and deep because their banks are typically cohesive sediments; often found on alluvial fans.	Ch. 1
Anthroprogenic constraints	Constraints on a stream or river that are caused by human activities or constructed projects.	Ch. 2
Annual duration gage analysis	The analysis of the recorded peak flow values that have occurred for each year in the duration of interest; typically used for the estimate of flows with return intervals in excess of 2 years.	Ch. 5
Annual flood	The highest peak discharge that can be expected to occur on average in a given year.	Ch. 5
Areal sediment sampling	See Surface sediment sampling.	TS 13A
Arid	An area which generally has insufficient rainfall to support conventional agriculture without supplemental irrigation.	TS 14I
Armor layer	A streambed containing at least some sediment that is too large to be transported by the hydraulic flow conditions, finer particles are selectively removed, leaving a layer of coarser materials.	Ch. 7, TS 13A
Armor layer (sampling)	Technique used to sample the upper layer of coarse surface layer material.	TS 13A
Articulating concrete block (ACB)	A matrix of interconnected concrete block units installed to provide an erosion resistant revetment for streams and rivers.	TS 14L
Attenuation	The subsidence or flattening of a floodwave as it moves down the channel.	Ch. 6
Avulsions	Occur when bank erosion and longitudinal adjustment occur at a large scale and is typically characterized by rapid changes in channel planform.	Ch. 1
Barb	See Stream barb.	
Baseflow	See Low flow.	
Band-aid solution	Treatment techniques used to address small, local issues.	Ch. 14
Bank zone	The area above the toe zone, located between the average water level and the bankfull discharge elevation.	Ch. 4, TS 14I

Bankfull depth	The distance from the deepest part of the channel to the bankfull elevation line, typically measured across a straight section (riffle) of a channel.	Ch. 3
Bankfull discharge	Used as a surrogate for channel-forming discharge, defined, in part, by the visual identification of morphological bankfull indices.	Ch. 5
Bankfull indices	Field indicators of bankfull discharge.	CH. 5, TS 5
Bankfull width	The width of channel at bankfull elevation.	CH. 3
Bankline migration	The adjustment of planform in natural meandering channels.	Ch. 12
Bat	A flying mammal (*Chiroptera*). TS13D	Ch. 3
Bed control structure	A type of grade control structure that is designed to provide a hard point in the streambed that is capable of resisting the erosive forces of the stream.	TS 14G
Bed zone	The bottom of the channel.	Ch. 4, TS 14I
Bedding layer	See Filter layer.	TS 14K
Bedform scour	Vertical channel bed movement that results from the troughs between crests of the bedforms.	TS 14B
Bedrock	A solid rock on the face or beneath the Earth's surface.	Ch. 3
Bend scour	Bed erosion along the outside of a river or stream bend.	TS 14B
Bendway weirs	A flow-changing bank stabilization technique used to protect and stabilize stream and river banks. Flows are directed over the weir perpendicular to the angle of the weir.	TS 14H
Biota	The plants and animals of a region.	Ch. 1
Braided streams	Multiple-thread streams formed in response to erodible banks, high bed-material sediment load, and rapid and frequent variations in stream discharge. The multiple channels of braided streams tend to be shallow and wide.	Ch. 1
Branch packing	A soil bioengineering technique used to fill localized slumps and gullies. It involves the use of alternating layers of live cuttings and soil.	TS 14I
Bridge pier scour	Erosion of a streambed around the piers of bridges.	TS 14B
Brush layering	A soil bioengineering technique that provides protection against surface erosion and shallow-seated slope failure. It involves the use of alternating layers of live cuttings and soil.	TS 14I

Brush mattress	A streambank soil bioengineering technique that includes a layer of live cuttings placed flat against the sloped face of the bank.	TS 14I
Brush revetments	A soil bioengineering technique used to stabilize streambanks. Brush and tree revetments are nonsprouting shrubs or trees installed along the toe of the streambank to provide bank erosion protection and to capture sediments.	TS 14I
Brush spur	A long, box-like structure of brush that extends from within the bank into the streambed. They function very similarly to stone stream barbs.	TSs 14I, 14J
Brush trench	A soil bioengineering technique that is a row of live cuttings that is inserted into a trench along the top of an eroding streambank, parallel to the stream. The live cuttings form a fence that filters runoff and reduces the likelihood of rilling.	TS 14I
Brush wattle fence	See Wattle.	
Bulk sediment sampling	See Volumetric sediment sampling.	TS 13A
Burst swimming speed	Refers to the highest swimming speeds of a fish; generally lasts less than 20 seconds and ends in extreme fatigue.	TS 14N
Catadromous fish	Species that hatch in saltwater, migrate to freshwater as juveniles to grow, and return to saltwater to spawn.	TS 14N
Catchment	See Drainage area.	
Celerity	The speed that a floodwave moves down the channel.	Ch. 6
Channel alignment design	Techniques used to establish a stable channel planform.	Ch. 12
Channel classification	See Classification.	
Channel evolution	Systematic changes of a stream channel to a perturbation.	Ch. 3
Channel evolution model (CEM)	A model that illustrates the stages through which a stream progresses when subjected to destabilizing influences.	Ch. 3
Channel evolution model classification	A classification system that provides a predictable sequence of change in a disturbed channel system.	Ch. 3
Channel-forming discharge	Concept based on the idea that for a given alluvial stream, there exists a single discharge that, given enough time, would produce the width, depth, and slope equivalent to those produced by the natural flow in the stream. This discharge, therefore, dominates channel form and process.	Ch. 5
Channel slope	The average slope of the longitudinal thalweg profile.	Ch. 1
Channel stage classification	A stream classification system based on the channel evolution model.	Ch. 3
Channel stages	See Channel evolution model.	

Channel storage	Water that is temporarily stored in a natural or constructed channel while en route to an outlet.	Ch. 5
Channelization	The alteration of an existing river or stream for a specific physical, biologic, or aesthetic purpose.	Ch. 1
Check dam	A small dam constructed to slow stream velocity and/or prevent degradation.	TS 14P
Classification	The categorization of a stream reach into a specific class based on factors and measurements such as dominant mode of sediment transport, entrenchment ratio, and sinuosity. Streams can also be classified by their biota, habitat conditions, baseflow levels, and direct measures of water quality.	Ch. 3
Clear water scour	Occurs when there is insignificant transport of bed-material sediment from the upstream into the contracted section.	TS 14B
Coefficient of determination	Usually expressed as R^2, this commonly used measure of the goodness of fit is a dimensionless ratio of the explained variation in the dependent variable over the total variation of the dependent variable.	Ch. 5 Ch. 9
Coir fascine	A soil bioengineering technique used to stabilize streambanks. A manufactured product consisting of coconut husk fibers bound together in a cylindrical bundle held by natural or synthetic netting.	TS 14I
Compaction	The process of densifying soil so that air is expelled and the pore space is reduced.	TS 14I
Conditional letter of map amendment (CLOMA)	Provides Federal Emergency Management Agency's comment on whether a proposed project would be excluded from the Special Flood Hazard Area.	Ch. 17
Conditional letter of map revision (CLOMR)	Provides for a review of whether a proposed project within the Special Flood Hazard Area meets the minimum flood plain management criteria of the National Flood Insurance Program.	Ch. 17
Confidence limits	Provide a measure of the uncertainty or spread in an estimate. In hydrologic gage analysis, they are a measure of the uncertainty of the discharge at a selected exceedance probability.	Ch. 5
Confluence	The point where two streams or rivers merge. If they are of approximate equal size, this point may be called a fork.	Ch. 2
Conservation management unit (CMU)	An area having similar land use and treatment needs and management plan.	Ch. 4
Constraints	Limitations on the physical or biologic behavior and characteristics of a stream.	Ch. 2
Constructed channel	A ditch or reconstructed natural channel.	Ch. 2

Construction inspector	The person responsible for the day-to-day quality control inspection required to ensure that the work is installed according to the design, industry standards, and contract requirements.	Ch. 15
Contour fascines	See Fascines.	
Contract types	The many methods used to direct and pay for the installation of stream restoration or stabilization. The contract types vary primarily by administrative burden, construction oversight, and incentive for the contractor to control cost.	Ch. 15
Contracting officer (CO)	The person responsible for administering the contract including ensuring that the proper type of contract is being used and funds are spent according to regulations.	Ch. 15
Contracting officer's representative (COR)	The person responsible to the state engineer and the contracting officer to see that the work is carried out as designed and in accordance with the contract requirements.	Ch. 15
Contraction scour	Erosion of a streambed that occurs when the flow cross section is reduced by natural features, such as stone outcrops, ice jams, or debris accumulations, or by constructed features such as bridge abutments.	TS 14B
Conveyance	A measure of the flow-carrying capacity of a cross section.	Ch. 6
Cost reimbursement contract	A contract type whereby the contractor is paid for identified costs that are defined as reimbursable. See Contract types.	Ch. 15
Crib wall	A soil bioengineering technique used to stabilize streambanks. The crib is a hollow, box-like structure of interlocking logs or timbers. The structure is filled with rock, soil, and live cuttings or rooted plants.	TS 14K
Crimping and seeding	A soil bioengineering surface roughening treatment that secures straw to the surface. It is a temporary surface treatment that protects and promotes the establishment of permanent grasses and vegetation.	TS 14I
Critical shear stress	The shear stress at the initiation of particle motion.	Ch. 8
Cross-section area	See Flow area.	
Cross vane structure	A structure that provides grade control and a pool for fish habitat.	Ch. 11, TS 14G
Crumb test	A common field test for dispersive clays.	TS 14A
Darting speed	See Burst swimming speed.	
Dead stout stakes	Diagonally cut 2- by 4-inch lumber used to secure soil bioengineering practices.	TS 14I
Deflector	A structure that forms a physical barrier to protect the bank, and forces the flow to change direction either by direct impact or deflection.	TS 14H

Degradation	Long-term sediment removal occurring through increased erosion from the channel bed.	Ch. 13
Denil fishway	A rectangular channel fitted with a series of symmetrical, closely spaced baffles that redirect flowing water and allow fish to swim around or over a barrier.	TS 14N
Denil ladder	See Denil fishway.	
Depth	The distance between the channel bottom and the water surface.	Ch. 6
Design flows	Stream restoration design should consider a variety of flow conditions. These flows should be considered from both an ecological, as well as a physical, perspective.	Ch. 5
Design layout	The physical location of design elements in a stream restoration project; the most common methods used to locate features on a drawing include referencing to a baseline or centerline, creating a grid, or using a global positioning system (GPS).	Ch. 15
Design storm	A prescribed precipitation distribution and associated recurrence interval.	Ch. 5
Dimensionless shear stress	The ratio of the critical shear stress and the product of the grain diameter and the submerged specific weight of the particle, also referred to as the Shields parameter.	Ch. 8
Discharge	The rate of flow, often expressed in cubic feet per second, or ft^3/s.	Ch. 5
Disturbances	Changes to the physical or ecologic condition that are outside of the normal range of natural variations. Disturbances can be natural or anthropogenic.	Ch. 1
Ditch	A long, relatively narrow, constructed channel.	Ch. 10
Do Nothing option	See Future without action alternative.	
Dominant channel processes	Dominant channel processes are the forces at work in the watershed, which cause and limit channel change.	Ch. 13
Dominant discharge	See Channel-forming discharge.	
Dormant post planting	A soil bioengineering technique involving the use of large dormant stems, branches, or trunks of live woody plant material, that are planted for bank erosion control and creation of riparian vegetation.	TS 14I
Drag	The fluid force component acting on a sediment particle, which is parallel to the mean flow.	TS 14J
Drainage area	The area from which surface rainfall runoff is contributed to a specific point.	Ch. 5

Drained soil conditions	This is not a description of the water level in the soils, but rather a description of the pore pressure condition in the soil when it is loaded. A drained condition implies that either no significant pore pressures are generated from the applied load or that the load is applied so slowly that the pressure dissipates during the slowly applied loading. See Undrained soil conditions.	TS 14A
Duration	The length of time that water flows at a given discharge or a given depth.	Ch. 6
Effective discharge	The mean of the arithmetic discharge increment that transports the largest fraction of the annual sediment load over a period of years; often used as a surrogate for channel-forming discharge.	Ch. 5
Embankment bench	A technique used to stabilize steep banks with little or no disturbance at the top of the slope and minimal disturbance to the streambed. A gravel bench is constructed along the toe and protected with riprap.	TS 14K
Endangered Species Act (ESA)	A 1973 Act of Congress instructing Federal agencies to carry out programs to conserve endangered and threatened species and to conserve the ecosystems on which these species depend.	Ch. 17
Energy	A property of a body or physical system which enables it to move against a force. It is the amount of work required to move a mass through a distance.	Ch. 6
Engineer	The person responsible for the technical requirements of project installation and represents the owner.	Ch. 15
Entrenchment	The extent of vertical containment of a channel relative to its adjacent flood plain.	Ch. 3
Entrenchment ratio	The flood-prone width divided by the bankfull width.	Ch. 3
Ephemeral stream	A stream or reach of a stream that flows only in direct response to precipitation, and whose channel is above the water table at all times. The term may be arbitrarily restricted to a stream that does not flow continuously during periods of as much as a month.	Ch. 7
Ephemeroptera, Plecoptera, and Trichoptera Index (EPT)	A biologic assessment technique that is used to assess land use and water quality within a watershed. It uses benthic macroinvertebrates, such as stoneflies, mayflies, and caddis flies as indicators.	Ch. 3
Equilibrium bed slope	The slope at which the sediment transport capacity of the reach is in balance with the sediment transported into it.	Ch. 13, TS 14B
Equipment rental contracts	A contract type used in instances where a fixed-price construction contract would be impractical because of the nature of the work and when it would not be feasible to prepare detailed drawings and specifications. It requires substantial construction oversight. See Contract types.	Ch. 15

Equilibrium slope	The slope of a channel at which the sediment transport capacity of the reach is in balance with the sediment transported into it.	Ch. 13, TS 14G
Erosion	The wearing away of soil by running water, wind, or ice.	Ch. 1
Erosion control blankets (ECB)	A temporary protective blanket laid on top of bare soil vulnerable to erosion; commonly made of mulch, wood fiber, or synthetics.	TSs 14D, 14I
Erosion control fabric	See Erosion control blankets.	TSs 14D, 14I
Erosion stop wattle fence	See Wattle.	
Excavated bench	A technique used to stabilize steep banks with little or no disturbance at the top of the slope and minimal disturbance to the streambed. It involves shaping the upper half or more of the high bank to allow the formation of a bench to stabilize the toe of the slope.	TS 14K
Extremal hypothesis	A hypothesis that assumes a channel will adjust its geometry so that the time rate of energy expenditure is minimized.	Ch. 9
Facet	A distinct morphological segment of a longitudinal profile; riffle, pool, run, or glide (tail-out).	TS 3E
Fascine	A soil bioengineering technique used to provide stabilization to the toe of streambanks. A long bundle of live cuttings bound together into a rope or sausage-like bundles.	TS 14I
Federal Acquisition Regulations (FAR)	Regulations that govern Federal contracts.	Ch. 15
Filter layer	A layer that prevents the smaller grained particles from being lost through the interstitial spaces of the riprap material, while allowing seepage from the banks to pass. This layer typically consists of a geosynthetic layer or sand, gravel, or quarry spalls.	TS 14K
First-order stream	An unbranched tributary.	Ch. 3
Fish ladders	The broad category of techniques used to provide migrating fish with upstream passage around or through fish passage barriers.	TS 14N
Fish screens	The broad category of devices used to preclude adult and juvenile fish from entering flow diversion structures, pump intakes, diversion channels, pipes, or penstocks.	TS 14N
Fishways	See Fish ladders.	
Fixed-price contract	In most cases, considered to be the preferable type of construction contract. However, it requires an accurate cost estimate and construction details. See Contract types.	Ch. 15

Flood	A general term given to a relatively high flow measured in height or discharge quantity.	Ch. 5
Flood frequency	The anticipated period in years before a given flood will reoccur.	Ch. 5
Flood insurance rate map (FIRM)	The official map of a community on which the Federal Emergency Management Agency has delineated both the special hazard areas and the risk premium zones applicable to the community.	Ch. 17
Flood plain maps	Maps developed by the National Flood Insurance Program to reduce damages and loss of life caused by floods. The basis for flood management, regulation, and insurance requirements by identifying areas subject to flooding are provided.	Ch. 17
Flood-prone width	The width of the active flood plain at the flood plain elevation (twice the maximum bankfull depth); composed of the active channel (bankfull width) and left and right flood plain (flood-prone) widths.	Ch. 3
Floodway	The channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation by more than designated height.	Ch. 17
Flow area	The area of the cross section between ground and water surface.	Ch. 6
Flow-changing devices	A broad category of structures which can be used to divert flows away from eroding banks.	TS 14H
Flow depth	See Depth.	
Flow duration	The percentage of time that a flow level is equaled or exceeded in a stream or river, typically represented with a flow-duration curve.	Ch. 5
Flow-frequency analysis	A consistent, statistical method for denoting the probability of occurrence of flows at a specific point in a stream system.	Ch. 5
Fluvial	Streams and rivers, in geography and Earth science, it is used to refer to all topics related to flowing water.	Ch. 1
Fluvial fish	Species that live in the flowing waters of rivers or streams, but migrate between rivers and tributaries for breeding, feeding, or sheltering.	TS 14N
Fluvial geomorphology	The study of the origin and evolution of landforms shaped by river processes.	Ch. 1
Force account agreements	Used when the sponsor performs the work using its own equipment and personnel.	Ch. 15

Formal contract	Under the Federal Acquisition Regulations as of 2005, formal contracts must be used for projects with a value greater than $100,000.	Ch. 15
Friction factor (*f*)	The roughness coefficient in the Darcy-Weisbach velocity equation.	Ch. 6
Froude number	A dimensionless ratio, relating inertial forces to gravitational forces, and representing the effect of gravity on the state of flow in a stream.	Ch. 6
Future without Action alternative	The option that involves allowing the site to progress without a project. The resources, both physical and ecological, that may be lost by not implementing the project are assessed as part of this alternative.	Chs. 2, 14
Gabion	A rock-filled wire mesh basket used to stabilize streambanks and slopes.	TS 14K
Gabion grade control	Grade control structures built with rock-filled wire mesh baskets.	TS 14G
Gage analysis	The use of statistical techniques to estimate probable frequency of flow events from recorded stream or river gage records.	Ch. 5
General permits	Issued Nationwide or regionally for categories of activities that are either similar in nature and cause only minimal individual and cumulative adverse impacts.	Ch. 17
General scour	Streambed erosion affecting the entire channel cross section.	TS 14B
Geocell	A product composed of polyethylene strips, connected by a series of offset, full-depth welds to form a three-dimensional honeycomb system.	TS 14D
Geogrid	A geosynthetic formed by a regular network of integrally connected elements with apertures greater than a quarter inch to allow interlocking with surrounding soil, rock, earth, and other surrounding materials to function primarily as reinforcement.	TS 14D
Geologic assessment	The review of both the surface and subsurface features of geology and their possible impacts on a stream or river.	Ch. 3
Geomorphic analog	The use of a stable stream reach as a template for restoration design.	Ch. 2
Geomorphic goals	Goals or objectives based on concepts of landscape position, landforms, and ongoing processes that change them.	Ch. 2
Geomorphology	The study of the origin and evolution of landforms.	Ch. 1
Geonet	A geosynthetic consisting of integrally connected parallel sets of ribs overlying similar sets at various angles for planar drainage of liquids and gases.	TS 14D

Geosynthetic	A planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as part of a manmade project structure, or system.	TS 14D
Geotechnical analysis	The evaluation of the forces involved in bank instability problems including gravity acting on the soils in the slope, the internal resistance of soils in the slope, and the seepage forces in the soils in the bank.	TS 14A
Geotextile	A permeable geosynthetic comprised solely of textiles.	TS 14D
Glide	The downstream end of pools, just upstream of the next riffle, where the channel slope becomes adverse as the deeper section is intercepted by the tailing off point bar.	Ch. 11
Goals	The overall desired outcome, such as restore channel to preflood conditions.	Chs. 2, 16
Grade control	See Grade stabilization techniques.	
Grade stabilization techniques	Techniques used to stop channel degradation, typically accomplished by the construction of inchannel structures.	TS 14G
Grain Reynolds number	The ratio of the product of shear velocity and grain diameter to kinematic viscosity.	Ch. 8
Grass-lined channel design method	A threshold channel design technique used where climate and soils can support permanent vegetation, and baseflow does not exist. The approach is similar to the allowable velocity channel design method.	Ch. 8
Gravelometer	Device used to assist with the measurement of particles sampled as part of a pebble count.	TS 13A
Ground water	Water in a saturated zone or stratum beneath the land surface.	Ch. 1
Grout	See Grouted riprap.	
Grouted riprap	A riprap bed where the voids have been filled with concrete; often used where the required stone size cannot be obtained or at sites where a significant and damaging debris load is expected.	TSs 14K, 14G
Gully/gullies	Entrenched channels extending into areas with previously undefined or weakly defined channel conditions.	TS 14P
Gully plug	A small earthen dam constructed at one or more locations along the gully.	TS 14P
Habitat	A specific environment in which a particular plant or animal lives.	Ch. 1
Hybrid design methods	The use of a combination of analytical, as well as analogy and hydraulic geometry design methods, to calculate design variables.	Ch. 7

Hydraulic control structure	A type of grade control structure designed to reduce the energy slope along the degradational zone to the degree that the stream can no longer scour the bed.	TS 14G
Hydraulic depth	The ratio of the cross-section area of flow to the free surface or top width.	Ch. 6
Hydraulic geometry design method	Design approach based on the concept that a river system tends to develop in a predictable way, producing an approximate equilibrium between the channel and the inflowing water and sediment.	Chs. 7, 9
Hydraulic radius	The ratio of the cross-sectional area of flow to the wetted perimeter or flow boundary.	Ch. 6
Hydrodrill	See Waterjet stinger.	
Hydrodrill stinger	See Waterjet stinger.	
Hydro-physiographic area	A drainage basin where the combination of the mean annual precipitation, lithology, and land use produces similar discharge for a given drainage basin.	Ch. 3
Incentive contracts	A contract type that links the contractor's profit to performance by establishing reasonable and attainable targets that are clearly communicated to the contractor. See Contract types.	Ch. 15
Incipient motion design	See Threshold channel design.	
Index of Biotic Integrity (IBI)	A biological assessment technique that uses fish surveys to assess human effects on a stream and its watershed.	Ch. 3
Individual permit	A type of permit that involves the evaluation of a specific project.	Ch. 17
Infiltration	The downward movement of water into the surface of soil.	Ch. 1
Informal contract	Under the Federal Acquisition Regulations as of 2005, informal contracts and contracting procedures can be used for projects with a value of $100,000 or less. Informal contracts are those put in place using simplified acquisition procedures.	Ch. 15
Intermittent stream	A stream that flows only at certain times of the year when it receives water from springs or from some surface source such as melting snow in mountainous areas. The term may be arbitrarily restricted to a stream that flows continuously during periods of at least 1 month; also may be a stream that does not flow continuously, as when water losses from evaporation or seepage exceed the available streamflow.	Ch. 7
Irrigation ditch	A long, narrow constructed channel used to convey irrigation water from its source to place of use.	Ch. 1

Jetties	A flow-changing technique used to stabilize and protect stream and river banks. Fence-like structures extending from the bank and into the stream.	TS 14H
J-hook	A rock structure used to provide bank stabilization.	Ch. 11, TS 14G
Joint planting	A streambank soil bioengineering technique that includes cuttings of live woody plant material inserted in the voids of riprap, and into the ground below the rock.	TSs 14I, 14K
Jumping height	The maximum height obtained by a specific species and age of fish. Older and larger fish have greater maximum jumping heights, although some species have no jumping abilities at any age.	TS 14N
Labor-hour contracts	A variation of the time-and-materials contract, differing only in that materials are not supplied by the contractor. See Contract types.	Ch. 15
Lane's relationship	A qualitative conceptual model, also known as a stream balance used as an aid to visually assess stream responses to changes in flow, slope, and sediment load.	Ch. 13
Lane's tractive force design method	See Allowable shear stress design method.	
Large woody materials (LWM)	Habitat and bank stabilization provided until woody riparian vegetation and stable bank slopes can be established. Trees, branches, and rootwads are considered large woody materials. Also called large woody debris.	TS 14J
Letter contracts	Written preliminary contractual instruments that authorize the contractor to begin work immediately.	Ch. 15
Letter of map amendment (LOMA)	An amendment to the currently effective Federal Emergency Management Agency map establishing that a property is not located in a Special Flood Hazard Area.	Ch. 17
Letter of map revision (LOMR)	An official amendment to the currently effective Federal Emergency Management Agency map.	Ch. 17
Letter of permission (LOP)	A type of permit issued through an abbreviated processing procedure.	Ch. 17
Lift	The fluid force component on sediment particles perpendicular to the mean flow direction.	TS 14C
Live bed conditions	May be assumed at a site if the mean velocity upstream exceeds the critical velocity for the beginning of motion for the median size of bed material available for transport.	TS 14B

Live brush sills	A soil bioengineering technique that involves rows of live cuttings inserted into an excavated trench. This treatment is intended to promote sediment deposition and can function as erosion stops.	TS 14I
Live pole cuttings	A soil bioengineering technique that involves the use of dormant stems, branches or trunks of live woody plant material inserted into the ground that are planted for bank erosion control and creation of riparian vegetation.	TS 14I
Live post planting	See Dormant post planting.	
Live siltation	See Live brush sills.	
Live stakes	See Live pole cuttings.	
Local scour	Erosion of the streambed immediately adjacent to some obstruction to flow.	TS 14B
Log crib	See Crib wall.	
Log-Pearson type III distribution	The most commonly used frequency distribution for peak flows in the United States. It applies to nearly all series of natural floods; commonly used for stream gage analysis.	Ch. 5
Longitudinal peak stone toe (LPST)	A type of bank protection involving the placement of a windrow of stone in a peak ridge along the toe of an eroding bank.	TS 14K
Loose rock grade control structure	A simple type of a grade control structure consisting of placing natural stone or other nonerodible elements across the channel to form a hard point.	TS 14G
Low flow	A general term that refers to the average low flows in a stream. It is typically due to soil moisture and ground water. Critical habitat conditions often occur during low flows.	Ch. 5
Low-flow channel	A portion of a channel that conveys low or baseflows.	Ch. 10
LUNKERS	Little Underwater Neighborhood Keepers Encompassing Rheotactic Salmonids—a technique providing both streambank stability and edge cover aquatic habitat.	TS 14O
Maintenance	Actions taken to ensure that the stream restoration project performs as designed and attaining project objectives.	Ch. 16
Manning's *n*	An empirical factor in Manning's equation which accounts for frictional resistance of the flow boundary.	Ch. 6
Meander	Deviation of the stream direction from the shortest possible path down a stream valley.	Ch. 12
Meander geometry	The five parameters commonly used in the description of meander patterns, including wavelength, radius of curvature, arc length, amplitude, and beltwidth.	Ch. 12

Meander length	The product of the meander wavelength and the valley slope divided by the channel slope.	Ch. 12
Meander ratio	The length of the stream divided by the length of the valley.	Ch. 12
Mobile boundary stability	The rate at which sediment enters the channel reach from upstream equal to the capacity of the reach to transport sediment of the same composition on downstream.	Ch. 7
Model (1–D)	One-dimensional models only consider forces that occur in one direction (usually the streamwise). Velocity and other stream properties may vary upstream and downstream, but not from bank to bank and not from the bed to the water surface.	Chs. 1, 5
Model (2–D)	Models are usually depth-averaged. They simulate variation in the horizontal plane, but assume no variation in the vertical.	Chs. 1, 5
Model–conceptual	Describes the objects and relationships either with words or diagrams.	Ch. 1
Model–empirical	Contains any empirical relationship, one based on data. An empirical model is based, at least in part, on observed data, rather than a thorough understanding of the underlying physical principles.	Ch. 1
Model–lumped	Describes processes on a scale larger than a point, while a distributed model describes all processes at a point, then integrates processes over space and time to produce a total system response.	Ch. 1
Model–mathematical	Formal mathematical models representing objects and interactions quantitatively with equations.	Ch. 1
Model–parametric	Has parameters that must be estimated in some fashion.	Ch. 1
Model–physical	Three-dimensional representations, usually at some relevant scale.	Ch. 1
Model–steady	Predict conditions that occur for a given set of boundary conditions. For example, a flow model might predict the water surface elevation, given a fixed channel geometry and a constant flow.	Ch. 1
Model–stochastic	Outputs are predictable only in a statistical sense. Repeated use of a given set of model inputs produces outputs that are not the same, but follow certain statistical patterns.	Ch. 1
Model–unsteady	Predicted variations that occur with time, such as during the passage of a storm hydrograph, by dividing such an event into a series of steady-state time steps. Complex, unsteady models have feedback loops that allow channel boundaries or other key variables to respond to inputs and change between time steps.	Chs. 1, 5
Momentum	The mass of a body times its velocity.	Ch. 6

Monitoring	The process of measuring or assessing specific physical, chemical, and/or biological parameters of a project.	Ch. 16
Montgomery and Buffington classification	A classification system based on defining channel processes. It is a geomorphic process-based system.	Ch. 3
Muddying-in	The practice of pouring a slurry mix of water and soil into the hole around the cutting stem of a plant to achieve good soil to stem contact.	TS 14I
National Environmental Policy Act (NEPA)	The Federal law establishing a national policy for the environment and requires specific actions by Federal agencies.	Ch. 17
National Flood Insurance Program (NFIP)	A program administered by the Federal Emergency Management Agency providing for flood insurance, flood plain hazard mapping, and flood plain management.	Ch. 17
Nationwide General Permit (NWP)	A type of general permit issued nationally by the U.S. Army Corps of Engineers for specific dredge or fill activities.	Ch. 17
National Pollutant Discharge Elimination System (NPDES)	A provision of the Clean water Act regulating point discharges into waters of the United States.	Ch. 17
Natural channel	A river, stream, creek, or swale that has existed long enough and without significant alteration to establish a dynamically stable route.	Ch. 2
Navigable waters	Defined for U.S. Army Corps of Engineers regulatory purposes as those waters that are subject to the ebb and flow of the tide and/or are presently used, or have been used in the past, or may be susceptible for use to transport interstate or foreign commerce.	Ch. 17
Newbury riffle	A type of constructed loose rock grade control structure.	TS 14G
No Action alternative	See Future without action alternative.	
NRCS Conservation Practice Standards	Guidance provided for applying conservation technology and set the minimum criteria for acceptable application of the technology. State variations on these standards may be more restrictive.	Ch. 4
NRCS contract specialist	The person who assists the administrative officer in contract matters for contracts and agreements.	Ch. 15
NRCS Planning Process	Steps used to develop an appropriate plan for natural resource protection or improvement.	Chs. 2, 4
NRCS State Conservation Practice Standards	Each state determines which NRCS National Conservation Practice Standards are applicable in their state. States add the technical detail needed to effectively use the standards at the field office level, and issue them as state conservation practice standards. Minimum criteria may be more restrictive than the national standards.	Ch. 4

Objectives	The detailed, focused outputs or outcomes that achieve the project goals.	Chs. 2, 16
Open channel flow	Flow where one surface is open to the atmosphere.	Ch. 6
Ordinary high water	The limit of U.S. Army Corps of Engineers jurisdiction in nontidal waters of the United States, in the absence of adjacent wetlands; defined as that line on the shore established by the fluctuations of water and indicated by physical characteristics.	Ch. 17
Outliers	Data points that depart significantly from the trend of the remaining data.	Ch. 5
Owner	The person responsible for contracting for construction. For NRCS Federal contracts, NRCS is considered the owner during construction.	Ch. 15
Partial duration gage analysis	The analysis of the recorded peak flow values above a preselected base value that have occurred for each year in the duration of interest; typically used for the estimate of flows with return intervals less than 2 years.	Ch. 5
Pattern	Plan view of a stream reach.	Chs. 3, 12
Pebble count	Technique used to sample the surface layer of sediments in gravel-bed streams.	TS 13A
Perennial stream	A stream that flows continuously. Streams flowing continuously throughout the year and are generally lower than the water table in the region adjoining the stream.	Ch. 7
Performance of work agreement	An agreement that requires that the value of work to be performed by the sponsoring local organization be determined by negotiation between the sponsoring local organization and NRCS and be included in the project agreement. NRCS must estimate the cost of the work to establish the maximum value of work before signing the agreement.	Ch. 15
Pile foundations	Used to transfer foundation forces through relatively weak soil to stronger strata to minimize settlement. The most likely applications for pile foundations in stream restoration and stabilization projects are as support for bank stabilization structures (retaining wall) and as anchors for large woody material.	TS 14F
Pin deflectors	Variations of the permeable jetty, generally used in streams where only a small reduction in velocity is needed. Generally wood pilings are used for their construction.	TS 14H
Piston aerial sampler	Device used to facilitate underwater aerial sediment sampling of fine material.	TS 13A
Plan	A sequence of logical steps followed to reach a goal or objective.	Ch. 2

Planform	Horizontal alignment of a channel; view is perpendicular to the Earth's surface.	Ch. 12
Point bar	A depositional area formed on the inside bank of a meander that sometimes remains bare of vegetation due to the frequent recurrence of the bankfull discharge.	Ch. 12
Pool	The area in a natural channel deeper and somewhat narrower than the average channel section.	Ch. 12
Practice standards	See NRCS Conservation Practice Standards.	
Pressure head	The potential energy of water, usually the result of its mass and the Earth's gravitational pull.	Ch. 6
Programmatic General Permit (PGP)	A type of general permit issued to avoid unnecessary duplication of regulatory control exercised by another Federal, state, or local agency.	Ch. 17
Project agreements	Any agreement(s) entered into by NRCS and sponsors, in which detailed working arrangements are established for the installation of cost-shared measures.	Ch. 15
Pump intake fish screens	See Fish screens.	
Quality assurance (QA)	Tasks or procedures undertaken to ensure that procedures are adhered to that will assure that the work will meet the minimum requirements. Quality assurance activities vary in accordance with the complexity and hazard class of the stream restoration project.	Ch. 15
Quality assurance plan (QAP)	Identifies the individuals with the expertise to perform various QA tasks, outline the frequency and timing of testing, estimate the contract completion date, and be co-approved by all responsible supervisors.	Ch. 15
Quality control (QC)	Tasks or procedures undertaken to ensure that the work installed meets the minimum requirements of the contract.	Ch. 15
R^2	The coefficient of determination. This commonly used measure of the goodness of fit is a dimensionless ratio of the explained variation in the dependent variable over the total variation of the independent variable.	Chs. 5, 9
Reach	A length of stream or river having some defined uniform characteristics.	Ch. 1
Reclamation	A series of activities intended to change the biophysical capacity of an ecosystem. The resulting ecosystem is different from the ecosystem existing prior to recovery. The term has implied the process of adapting wild or natural resources to serve a utilitarian human purpose, such as the conversion of riparian or wetland ecosystems to agricultural, industrial, or urban uses.	Ch. 1

Reconnaissance	A preliminary investigation not involving detailed investigation and relying heavily on existing data and observations.	Ch. 3
Redirective structure	A flow-changing bank stabilization technique. Designed to be placed in the stream, minimize direct impact, and rely more on the characteristics of fluid mechanics to modify the streamflow direction.	TS 14H
Reference reach design method	An alluvial channel design approach whereby channel dimensions are selected from a similar stable channel.	Chs. 2, 9
Regime design method	An alluvial channel design approach whereby channel dimensions are selected with the aid of empirically derived equations.	Ch. 9
Regional curves	A tool frequently associated with the Rosgen geomorphic channel design approach, but also applicable to other design methods. It involves bankfull dimensions correlated to drainage area. See Hydraulic geometry design.	Ch. 11, TS 5
Regional general permits (RGPs)	A type of general permit issued regionally.	Ch. 17
Regression equations (gage analysis)	Used to transfer flood characteristics from gaged to ungaged sites through use of watershed and climatic characteristics as predictor variables.	Ch. 5
Regulated stream systems	Streams or rivers that are cleared of wood, dammed, channelized, leveed or constrained by other types of hard structures.	Ch. 1
Rehabilitation	Making the land useful again after a disturbance. It involves the recovery of ecosystem functions and processes in a degraded habitat.	Chs. 1, 2
Resource management systems (RMS)	Sets of approved conservation practices.	Chs. 2, 4
Restoration	The reestablishment of the structure and function of ecosystems. Ecological restoration is the process of returning an ecosystem as closely as possible to predisturbance conditions and functions.	Ch. 1
Retard	A flow-changing bank stabilization technique. A retard structure increases flow resistance by increasing drag, thereby slowing the velocity in the vicinity of the structure. These structures are more porous with a high percentage of open area.	TS 14H
Reynolds number	A dimensionless ratio, relating the effect of viscosity to inertia, used to determine (index) whether fluid flow is laminar or turbulent.	Ch. 6
Riffle	The area in a natural channel that is wider and shallower than the average channel section.	Ch. 12
Riffle pool spacing	The distance between the riffles and the pools in a channel.	Ch. 12

Rigid boundary stability	Attained when the interaction between flow and the material forming the channel boundary is such that the soil boundary effectively resists the erosive efforts of the flow.	Ch. 8
Rigid drop grade control structure	A complex type of grade control structure that is used for large drops. These structures are frequently constructed of concrete or a combination of sheet pile and concrete.	TS 14G
Riparian zones	The areas between aquatic and upland habitats.	Ch. 1
Riprap	Large stone used to provide immediate and permanent stream and river bank protection.	TS 14K
Riprap sizing	See Stone sizing.	
Risk	The exposure of life, property, and/or the environment to loss or harm.	Chs. 2, 5
Risk analysis	The assessment of the consequences of specific action or inaction to life, property, and/or the environment.	Ch. 2
River	A large natural waterway confined within a bed and banks. In the context of this handbook, the term stream is often used interchangeably with river.	Ch. 1
River classification	See Classification.	Ch. 3
Rolled erosion control products	Consist of both erosion control blankets used for temporary erosion protection and turf reinforcement mats for more permanent erosion protection.	TS 14D
Rootwad revetments	Use of locally available logs and root fans to add physical habitat to streams in the form of coarse woody debris and deep scour pockets.	TS 14I
Rosgen classification	A stream classification system based on measurements of existing morphology.	Ch. 3
Rosgen geomorphic channel design method	A hybrid channel design approach that incorporates geomorphic measurements, hydraulic geometry and some analytical calculations.	Ch. 11
Rosgen stream type	See Rosgen classification.	
Rotary drum fish screens	See Fish screens.	
Run	The steepest section and shortest longitudinally, starting at the downstream end of a riffle as the channel enters the next pool.	Ch. 11
Salmonid	Family of fish which includes the salmons, trouts and chars. All of the species breed in freshwater, are migratory, and spend part of their life cycle in the ocean.	TS 14N
Scour	Downward vertical erosion in a channel bed.	TS 14B

Seasonal stream	An intermittent stream that flows only during a certain climatic season, such as a winterbourne. A stream or segments of a stream that normally goes dry during a year of normal rainfall. Seasonal streams often receive water from springs and/or long-continued water supply from melting snow or other sources.	Ch. 7
Sediment budget analysis	A quantitative sediment impact assessment of channel stability using the magnitude and frequency of all sediment-transporting flows done by comparing the mean annual sediment load for the project channel with that of the supply reach.	Ch. 13, TS 13B
Sediment competence	The ability to move the largest particle made available to the channel.	Ch. 11
Sediment continuity analysis	The volume of sediment deposited in or eroded from a reach during a given period of time is computed as the difference between the volumes of sediment entering and leaving the reach.	Ch. 13
Sediment impact assessment	An evaluation of a designed channel's ability to transport the inflowing water and sediment load, without excessive sediment deposition or scouring on the channel bed.	Ch. 13
Sediment rating curve	Correlates sediment flow to discharge for a stream reach or section.	Ch. 13
Sediment rating curve analysis	Sediment impact assessment technique used to assess the sediment transport characteristics of an existing or proposed stream project. This approach uses sediment rating curves to compare the sediment transport capacity of the supply reach to the existing and proposed project reach conditions.	Ch. 13
Sediment sampling	Technique used to quantify sediment in streams and rivers.	TS 13A
Shear	The pull of water on the wetted area in the direction of flow, and measured in units of force/area.	Ch. 9
Shear stress (average)	The product of the energy slope, hydraulic radius, and unit weight of water. Spatial and temporal variation may result in a higher or lower point value for shear stress.	Ch. 8
Sheet pile	Flat panels of steel, concrete, vinyl, synthetic fiber, reinforced polymer, or wood. Typical applications include toe walls, flanking and undermining protection, grade stabilization structures, slope stabilization, and earth retaining walls.	TS 14R
Shields diagram	Classic method for determining critical shear stress.	Ch. 8
Shields parameter	See Dimensionless shear stress.	
Sinuosity	The channel centerline length divided by the length of the valley centerline.	Chs. 3, 12
Slope stability	See Geotechnical analysis.	

Soil anchor	Technique used to anchor woody material to the streambed or bank to resist fluvial forces.	TS 14E
Soil bioengineering	The use of live and dead plant materials in combination with natural and synthetic support materials for slope stabilization, erosion reduction, and vegetative establishment.	TS 14I
Soil cement grade control	Structures constructed with a mix of Portland Cement and onsite soils.	TS 14G
Specific energy	The energy per unit weight of water at a given cross section with respect to the channel bottom.	Ch. 6
Specific force	The horizontal force of flowing water per unit weight of water.	Ch. 6
Spur dikes	Short dikes that extend out perpendicular from the bank into the channel along a reach of eroded bank.	TS 14H
Stability	A channel is considered stable (or in dynamic equilibrium) when the prevailing flow and sediment regimes do not lead to long-term aggradation or degradation.	Ch. 13
Stakeholders	Individuals or groups who fund a project or are affected by the project.	Ch. 2
Standard individual permit (SP)	A type of permit issued for activities that have more than minimal adverse impacts to waters of the United States. The evaluation of each permit application involves more thorough review of the potential effects of the proposed activity.	Ch. 17
State administrative officer (SAO)	The person responsible for all administrative matters for contracts and most agreements.	Ch. 15
State conservation engineer (SCE)	The person responsible for the design and ultimately responsible for ensuring proper construction of projects in a given state.	Ch. 15
Steady state models	Predict conditions that occur for a given set of boundary conditions.	Ch. 1
Stinger	Metal rod used to facilitate planting live cuttings into rock riprap.	TS 14I
Stone sizing	Technique used to determine the minimum size stone to resist stream velocity.	TS 14
Stream	A small natural waterway with a detectable current. Defined within a bed or banks. In the context of this handbook, the term stream is often used interchangeably with river.	Ch. 1
Streambank	The sides of a stream or river.	Ch. 2
Stream barbs	A flow-changing bank stabilization technique that are low dikes or sill-like structures that extend from the bank towards the stream in an upstream direction. As flow passes over the sill of the stream barb, it discharges normal to the face of the weir.	TS 14H

Streambed	The bottom of a stream or river.	Ch. 1
Stream classification	See Classification.	
Stream corridor	Includes the stream and extends in cross section from the channel's bankfull level towards the upland (perpendicular to the direction of streamflow) to a point on the landscape where channel-related surface and/or soil moisture no longer influence the plant community.	Ch. 1
Stream corridor restoration	One or more conservation practices used to overcome resource impairments and reach identified purposes.	Ch. 1
Stream order classification	A stream classification system based upon the degree of channel branching. An nth order stream is formed by the intersection of two or more (n-1) order streams.	Ch. 3
Stream power	The product of shear stress and mean velocity. A measure of the available energy a stream has for moving sediment, rock, woody, or other debris.	Chs. 6, 11
Stream setbacks	A width required to allow a stream to self-adjust its meander pattern.	TS 14S
Surface sediment sampling	Techniques used to characterize the surface of a gravel bed.	TS 13A
Sustained swimming speed	Refers to the low swimming speeds of a fish species. In general, it can be maintained for extended time periods with little to no fatigue.	TS 14N
S_e or $S_{Y,X}$	The standard error of estimate, typically expressed as S_e or $S_{Y,X}$. This is a measure of the quality of a regression equation and is the root mean square of the estimates. It is a measure of the scatter about the regression line of the independent variable.	Ch. 5
Thalweg	The deepest portion of the channel, sometimes referred to as the low-flow channel.	Ch. 1
Threshold channel	A channel in which channel boundary material has no significant movement during the design flow. The term threshold is used because the channel geometry is designed so that applied forces from the flow are below the threshold for movement of the boundary material.	Ch. 7
Threshold channel design	A design approach whereby a channel configuration is selected so that the stress applied during design conditions is below the allowable stress for the channel boundary.	Ch. 8
Timber crib	See Crib wall.	
Time-and-materials contract	Contract used to procure supplies or services on the basis of direct labor and materials costs. See Contract types.	Ch. 15
Toe zone	The portion of the bank between the average water level and the upper edge of the bottom of the channel.	Ch. 4, TS 14I

Top width	The width of a channel cross section at the water surface.	Ch. 6
Tractive power design method	A threshold channel design technique used in the assessment of channels in cemented and partially lithified (hardened) soils.	Ch. 8
Transfer methods (gage analysis)	Technique used to extrapolate peak discharges upstream or downstream from a stream gage or from gage data from a nearby stream with similar basin characteristics.	Ch. 5
Transition channel	A stream or river which may behave as an alluvial channel in one flow condition and as a threshold channel in another flow condition.	Ch. 7
Tree revetment	See Brush revetments.	
Tributary	A continuous perennial stream.	Ch. 1
Turf reinforcement mats (TRM)	Used to provide permanent erosion protection.	TS 14D
Two-stage channel design method	A hybrid channel design approach that incorporates a natural alluvial channel nested with a constructed flood plain bench.	Ch. 10
U.S. Army Corps of Engineers Regulatory Program	Program that evaluates permit applications for most construction activities that occur in the Nation's waters, including wetlands.	Ch. 17
U.S. Forest Service: Framework of Aquatic Ecological Units	An aquatic framework containing standard terms and classification criteria for aquatic systems and their linkages to terrestrial systems at all spatial scales.	Ch. 3
Uncertainty	The likelihood of a consequence occurring.	Ch. 2
Undrained soil conditions	This is not a description of the water level in the soils, but rather a description of the pore pressure condition in the soil when loaded. An undrained condition assumes pore pressures will develop due to a change in load. The assumption is that the pore pressures that develop are not known and thus must be implicitly considered in the methods used to test samples for this condition. See Drained soil conditions.	TS 14A
Uniform flow	Occurs when the gravitational forces that are pushing the flow along the channel are in balance with the frictional forces exerted by the wetted perimeter that are retarding the flow.	Ch. 6
Unsteady models	Predict variations that occur with time, such as during the passage of a storm hydrograph, by dividing such an event into a series of steady-state time steps.	Ch. 1
Valley slope	The maximum possible slope for the channel invert and is determined by the local topography, and a channel with a slope equal to the valley slope would be straight.	Ch. 9

Vanes	Flow-changing structures constructed in the stream designed to redirect flow by changing the rotational eddies normally associated with streamflow. They are used extensively as part of natural stream restoration efforts to improve instream habitat.	TS 14H
Vegetated gabion	A vegetated gabion incorporates topsoil into the void spaces of the gabion. Woody plantings and/or grass are planted into or through the structure.	TS 14K
Vegetated geogrid	See Vegetated reinforced soil slope.	
Vegetated reinforced soil slope (VRSS)	A soil bioengineering technique that is made up of layers of soil wrapped in synthetic geogrid or geotextile, with live cuttings or rooted plants installed between the wrapped soil layers.	TS 14I
Vegetated riprap	See Joint planting.	
Vegetated rock wall	A mixed-construction soil bioengineering streambank stabilization technique. The structural-mechanical and the vegetative elements work together to prevent surface erosion and shallow mass movement by stabilizing and protecting the toe of steep slopes.	TS 14M
Vegetated soil lifts	See Vegetated reinforced soil slope.	
Vegetated stone	Combining rock with soil bioengineering treatments can achieve benefits from both techniques.	TSs 14I, 14K
Velocity head	The kinetic energy of water.	Ch. 6
Vertical fixed plate fish screen	See Fish screens.	
Vertical traveling fish screen	See Fish screens.	
Visual geomorphic assessment	A qualitative assessment that includes judgment of current conditions, expected future conditions, and the river's anticipated response to the designed project.	Ch. 13
Volumetric sediment sampling	The techniques generally considered to be the standard sediment sampling procedure. It involves the removal of a predetermined volume of material that is large enough to be independent of the maximum particle size.	TS 13A
W-weir	Technique used to provide grade control on large rivers.	Ch. 11
Waterjet	See Waterjet stinger.	
Waterjet stinger	A device that uses high-pressure water to hydrodrill a hole in the ground to plant unrooted cuttings.	TS 14I
Watershed	A topographically bounded area of land that captures precipitation, filters and stores water, and regulates its release through a channel network into a lake, another watershed, or an estuary and the ocean.	Ch. 1

Wattle	A soil bioengineering technique made up of rows of live stakes or poles with live plant materials woven in a basket-like fashion. A wattle fence can be used to deter erosion in ditches or in small dry channel beds to resist the formation of rills and gullies.	TS 14I
Wetlands	Defined for U.S. Army Corps of Engineers regulatory purposes as those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support a prevalence of vegetation typically adapted for life in saturated soil conditions.	Ch. 17
Wetted perimeter	The length of cross-section boundary between water and ground.	Ch. 6
Width-to-depth ratio	The bankfull width divided by the mean bankfull depth (dimensionless.	Ch. 3
Wolman pebble count	See Pebble count.	
Wolman walk	See Pebble count.	
Woody debris	See Large woody materials.	
Work	Force applied over a distance.	Ch. 6

Chapter Summaries

The NRCS Stream Restoration Design Handbook (NEH654) presents a variety of engineering and ecological assessment and design tools. This handbook is not meant to be read linearly as a book; it is a set of tools and approaches that can be applied to stream restoration designs. The terms river or stream may be used in this handbook, but the terms do not denote a statutory size or watershed drainage area limitation or requirement. Any work performed on rivers and streams is under the purview of all applicable Federal, state, tribal, and local guidelines.

Chapter 1 Introduction: Ecological and Physical Considerations for Stream Projects

The NRCS Stream Restoration Design Handbook provides guidance for teams of engineers, biologists, geomorphologists, hydrologists, landowners, and resource managers who are planning and designing stream restorations. Goals may include controlling floods or sediment sources, improving stormwater drainage, stabilizing banks, improving fish habitat, or restoring the ecological functions and processes of a stream and its flood plain. Many approaches and techniques can be used to reach these goals, but a good understanding of the living and nonliving components of the stream ecosystem, its watershed, how they interact and affect each other, and the timeframes over which stream processes occur will improve the probability of desirable outcomes. Chapter 1 presents a brief overview of current knowledge regarding stream ecosystem processes and functions important to consider when designing stream improvements. For a more comprehensive treatment of these processes, readers may wish to review one of several excellent references, including Stream Corridor Restoration: Principles, Processes, and Practices, developed by the Federal Interagency Stream Restoration Working Group.

Chapter 2 Goals, Objectives, and Risk

Chapter 2 addresses the development of goals and objectives and the assessment of risk from an ecological, as well as a life and property perspective. Identification of stream problems and their causes is a critical step in the overall planning process. Understanding the true nature of stream problems is challenging because of the dynamic nature of streams, their seasonal changes, responses to disturbances, and their ability to recover. Recognizing the current condition of a stream, comparing it to historical conditions, and projecting its future conditions are challenging; nonetheless, the conditions to be documented determine goals and objectives met through the outcomes of the plan. Risk and risk assessment is introduced in this chapter and also described throughout this handbook.

Chapter 3 Site Assessment and Investigation

Chapter 3 describes procedures for assessing watershed and site conditions. Stream corridor inventory and assessment techniques are identified and compared. Information is provided on stream stability, as well as geological and biological assessments. A description of the uses, advantages, and disadvantages of various geomorphic stream classification systems is also provided. This chapter addresses fluvial processes and broader geologic issues related to ecological function, as well as stream design and behavior.

Chapter 4 Stream Restoration Design Process

Conservationists are frequently faced with conditions along and in streams that are characterized as problems because certain functions are not being provided or simply that the overall character of the stream system has changed. It may be that the system is damaged and needs to be repaired or that a shift in perception of stream functions and values has occurred, spurring some sort of action to be taken.

Often, solutions to identified stream problems are suggested at the time that they are identified, such as: "The streambank is sloughing in. We need to put rock riprap on it." It could be that the problem merits that response. It could also be that the nature of the bank erosion problem is more complex and may be related to a general instability of the stream system. This chapter describes design approach that is applicable to the variety of potential treatment alternatives that are employed.

Chapter 5 Stream Hydrology

Stream restoration designs should consider a variety of flow conditions from both an ecological, as well as a physical perspective. A wide variety of sources and techniques for obtaining hydrologic data are available to the designer. Chapter 5 provides a description of the flows and their analysis that should be considered for assessment and design. The computation of frequency distributions is presented. Transfer equations, risk, and low flow methods are also addressed. This chapter also describes advantages and limitations of four general approaches for estimating channel-forming discharge or dominant discharge for stable channel design.

Chapter 6 Stream Hydraulics

Human intervention in the river environment, especially with projects intended to restore a riverine ecosystem to some healthier state, must take full consideration of streamflow, or stream hydraulics. Chapter 6 provides working professionals, both engineers and non-engineers, with practical information about hydraulic parameters and associated computations. It provides example calculations, as well as information about the role of hydraulic engineers in the project design process.

Chapter 7 Basic Principles of Channel Design

Channel design may involve the stabilization or realignment of an existing stream, or it may involve the creation of an entirely new channel. A wide variety of sources and techniques for designing stable channels are available to the designer. These techniques may focus on open channel design work ranging from natural stream restoration to primarily structural rehabilitations. The purpose of chapter 7 is to provide a framework to the designer to assess the use and application of several analysis and design techniques, which are presented in greater detail in subsequent chapters. Chapter 7 provides background that should be useful in the evaluation of the appropriateness of these techniques to address specific goals, constraints, and conditions. To provide a context for the different design techniques, a clear description of threshold and alluvial channels is presented. In addition, a general description of channel design variables and approaches is presented. These broad and occasionally overlapping categories of stream types and design approaches can be used to evaluate the appropriateness of the design techniques for a specific objective and site.

Chapter 8 Threshold Channel Design

Threshold channel design techniques are used for rigid boundary systems. In a threshold channel, movement of the channel boundary is minimal or nonexistent for stresses at or below the design condition. Therefore, the design approach for a threshold channel is to select a channel where the stress applied during design conditions is below the allowable stress of the channel boundary. There are a wide variety of sources and techniques for designing stable threshold channels that are available to the designer. Chapter 8 provides an overview and description of some of the most common threshold channel design techniques. Examples are provided to illustrate the methods.

Chapter 9 Alluvial Channel Design

Alluvial channel design techniques are generally used for movable boundary systems. In an alluvial channel, there is a continual exchange of channel boundary material with the flow. Therefore, the design of an alluvial channel requires an assessment of sediment continuity and channel performance for a range of flows. Many sources and techniques for designing stable alluvial channels are available to the designer. Chapter 9 provides an overview and description of some of the most common alluvial channel design techniques. The use and application of regime, analogy, hydraulic geometry, and analytical methods are presented and described. Examples are provided to illustrate the methods.

Chapter 10 Two-Stage Channel Design

Constructed channels are part of extensive portions of productive agricultural land in the United States. These channels provide important drainage and flood control functions. However, these agricultural channels are often constructed as traditional trapezoidal ditches using threshold design techniques. While this approach is suitable in some areas, channels of this design can require frequent and expensive maintenance in other parts of the country. This maintenance is often in the form of dredging and cleanout of deposited sediment. In addition, natural ecological functions can be lost. This chapter presents an alternative design to the conventional drainage channel, which seeks to mimic natural alluvial channel processes through the use of a two-stage channel design. This two-stage channel system incorporates benches that function as flood plains. However, these two-stage channels are not an exact copy of natural streams, as the width of the benches is often small due to the confining geometry of the constructed channel. This chapter outlines measurement and analysis procedures that can used to design two-stage channel systems that are more self-sustaining than conventional one-stage constructed channels.

Chapter 11 Rosgen Geomorphic Channel Design

Chapter 11 outlines a channel design technique based on the morphological and morphometric qualities of the Rosgen classification system. This approach has been implemented throughout numerous locations in the United States and is often referred to as the Rosgen design approach. The essence for this design approach is based on measured morphological relations associated with bankfull flow, geomorphic valley type, and geomorphic stream type. This channel design technique involves a combination of hydraulic geometry, analytical calculation, regionalized relationships, and analogy in a precise series of steps. While this technique may appear to be straightforward in its application, it actually requires a series of precise measurements and assessments.

Chapter 12 Channel Alignment and Variability Design

Natural channel design includes establishment of a stable planform and often the incorporation of variability within the channel. The designer of a channel is also often asked to provide an assessment of natural bankline migration, as well. The purpose of chapter 12 is to provide systematic hydraulic design methodologies that can be used in the performance of these tasks. A wide variety of sources and techniques for these assessments are available to the designer. An overview and description of some of the most common design techniques are described. Examples are provided to illustrate the methods.

Chapter 13 Sediment Impact Assessments

Sedimentation analysis is a key aspect of design since many projects fail due to excessive erosion or deposition. A sediment impact assessment is conducted to assess the effect that a full range of natural flows will have on possible significant aggradation or degradation within a project area. Chapter 13 provides a brief overview of several types of sediment impact assessments along with their rigor and level of uncertainty. The focus of this chapter is primarily on techniques that are appropriate for the analysis of alluvial channel, but threshold channels are also described. While there are variants in each of the presented techniques, and more information may be required to perform the assessments described, it is the intent of this chapter to provide the reader with an introduction to sediment impact assessments sufficient to select the appropriate approach for many circumstances. References are provided that outline specifics regarding the mentioned techniques.

It should also be supplemented that while this analysis of the sediment impact assessment is presented in the context of following the channel design, much of this analysis should also be done in the sediment assessment phase of the design process that precedes channel design. However, it is supplemented here as an important closure loop on any proposed design.

Chapter 14 Treatment Technique Design

Stream design and restoration often includes specific treatments on the riparian area, on the bank, or in the bed of a stream. Treatments can include techniques that provide ecological enhancement, as well as protection of these areas. This chapter provides an overview of some of the frequently used treatment techniques for bank protection, grade protection, and habitat enhancement, using a wide range of plant materials, rock, and other inert materials. In addition, analysis techniques that are needed for the successful design of these and other techniques are provided. Where information is available, the benefits, flexibility, risks, and costs of each technique are described from a physical, as well as an ecological perspective.

The list of techniques in this chapter should not be interpreted as an endorsement of any product that is mentioned, nor should it be inferred that one treatment or approach is superior to another. The approaches listed are not exhaustive. Other techniques, as well as variations of each of the ones described, exist and may be appropriate and applicable for use in restoration designs. This chapter provides techniques which often focus on the treatment of local problems, but these techniques and other design elements are often used to provide a holistic approach in larger or more complex restoration projects.

Chapter 15 Project Implementation

Chapter 15 addresses general project implementation issues with an emphasis on NRCS programs, requirements, and guidance. The four phases involved in project implementation are planning, design, contracts and agreements, and installation. This chapter describes how each phase is interrelated, how each phase requires knowledge of the limitations or restrictions of the other phases, and provides a general overview of project implementation.

Chapter 16 Maintenance and Monitoring

Maintenance and monitoring are actions intended to ensure the objectives of the stream restoration project are met over time. Continued performance of the project features and stream system health are dependent on appropriate maintenance and monitoring of the system. Chapter 16 provides an overview of key issues in the development of monitoring and maintenance plans. Incorporation of adaptive management as a component of operations is included as a possible approach to maintenance and operation of the project.

Chapter 17 Permitting Overview

Stream design and restoration design activities are subject to various local, state, and Federal regulatory programs. Most of these regulations are aimed at protecting natural resources and the integrity of the Nation's water resources. Chapter 17 provides a brief overview of the regulatory authorities and programs that may be applicable to stream design work. The focus is providing an awareness-level understanding of this important issue and sources to obtain more and current information. The reader should not interpret the description provided in this chapter as the only source of regulatory requirements. Local, state, and Federal regulatory authorities should be consulted as part of the planning and design efforts.

United States
Department of
Agriculture

**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 1 Introduction: Ecological and Physical Considerations for Stream Projects

Issued August 2007

Cover photos: *Top*—Restoring stream habitat is a balance between water, earth materials, plants and animals, and the goals and objectives of the restoration.

Bottom—The ecology of the stream must be characterized for the current and future conditions, with remedial measures in place.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 1 Introduction: Ecological and Physical Considerations for Stream Projects

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Chapter 1 Introduction: Ecological and Physical Considerations for Stream Projects

654.0100 Purpose

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) National Engineering Handbook (NEH), Part 654, Stream Restoration Design provides guidance for multidisciplinary teams who are planning and designing projects to improve streams and their functions. Specific project goals may include flood control, sediment control, improving drainage, stabilizing banks, improving fish habitat, and restoring the ecological functions and processes of a stream and its flood plain.

Many approaches and techniques can be used to reach these goals, but a good understanding of the living and nonliving components of the stream ecosystem, its watershed, how they interact and affect each other, and the timeframes over which stream processes occur will improve the chances of success.

This chapter provides an overview of processes important to stream corridors and their ecosystems. Stream corridors include the stream channel, riparian zone, and flood plains (level areas near the channel, formed by the stream and flooded during moderate-to-high flow events). Stream corridor features are shaped by the forces of flowing water, which depend on local topography and geological characteristics. Stream corridors are also influenced by the cumulative effects of upland and upstream activities and practices, including agricultural production, forestry, recreation, other land uses, or urban development.

The chemical and biological processes that occur within stream systems are intricate and involve numerous interactions, linkages, and feedback loops. Accordingly, this chapter presents a brief overview of current knowledge regarding stream ecosystem processes and functions important to consider when designing stream improvements. For a more comprehensive treatment of these processes, readers may wish to review one of several references, including Stream Corridor Restoration: Principles, Processes, and Practices, developed by the Federal Interagency Stream Restoration Working Group (FISRWG) (1998).

654.0101 Introduction

In 1998, water quality in at least 40 percent (by length) of assessed streams in the United States was listed as impaired by the U.S. Environmental Protection Agency (EPA) (EPA 2000). Reports of the status of freshwater species are also dismal: about a fourth of native freshwater fish species (Williams et al. 1989; Stein and Flack 1997), a half of native freshwater mussel species (Williams et al. 1993), a fourth of native amphibians (Stein and Flack 1997), and a third of native crayfish species (Taylor et al. 1996) are imperiled or extinct. Aquatic species are not only valued natural resources—they are indicators of water quality. The continued rapid decline in aquatic biodiversity (Ricciardi and Rasmussen 1999) places great responsibility on those who work in streams.

654.0102 Restoration, rehabilitation, and reclamation

Some methods of determining objectives have pitfalls. It is probably not possible to fully restore all the functions and values of a stream to a specified original condition, or, more accurately, a condition at a particular point in time. This ignores how streams form and how they maintain themselves. Taking a stream backwards is contradictory to what is known about modern ecology because it implies some static climax state that a natural system tends towards, both elastically and linearly.

Some stream work is clearly repaired in nature, which may be to fix simple erosion problems. Even simple erosion control projects on streams should be designed to maintain or improve ecological functions and values. Repairs can become little more than temporary bandages to treat what is actually a larger problem. This may result in wasted time and resources, if the problems are systemic in nature and reflect more serious imbalances between the stream, its riparian area and corridor, and its watershed.

The following terms are sometimes used interchangeably with regard to working on streams to restore specific functions and values (FISRWG 1998):

- **Restoration** is the reestablishment of the structure and function of ecosystems (National Research Council 1992). Ecological restoration is the process of returning an ecosystem as closely as possible to predisturbance conditions and functions. Implicit in this definition is that ecosystems are naturally dynamic. It is, therefore, not possible to re-create a system exactly. The restoration process reestablishes the general structure; function; and dynamic, but self-sustaining, behavior of the ecosystem.
- **Rehabilitation** is making the land useful again after a disturbance. It involves the recovery of ecosystem functions and processes in a degraded habitat (Dunster and Dunster 1996). Rehabilitation does not necessarily reestablish the predisturbance condition, but does involve establishing geological and hydrologically stable landscapes that support the natural ecosystem mosaic. Most of the stream projects that NRCS has been involved with are rehabilitations.
- **Reclamation** is a series of activities intended to change the biophysical capacity of an ecosystem. The resulting ecosystem is different from the ecosystem existing prior to recovery (Dunster and Dunster 1996). The term has implied the process of adapting wild or natural resources to serve a utilitarian human purpose, such as the conversion of riparian or wetland ecosystems to agricultural, industrial, or urban uses. Restoration differs from rehabilitation and reclamation in that restoration is a holistic process not achieved through the isolated manipulation of individual elements. While restoration aims to return an ecosystem to a former natural condition, rehabilitation and reclamation imply putting a landscape to a new or altered use to serve a particular human purpose (National Research Council 1992).

It may be difficult or impossible to restore a stream to a particular historical condition due to changes in watershed land use and human population, as well as slight to major climatic changes. It may also be difficult or impossible to adequately describe the desired historical condition, both in terms of the stream's pattern and physical characteristics, as well as its physical, biological, and chemical attributes—its ecology. For the purposes of this document, the planned stream actions, for which designs are needed, may be termed restoration, rehabilitation, or reclamation, in the context of the plan's objectives and goals. It is also possible that the plan may create or re-create some functions and values that are new to the stream, or are logical, given historical watershed changes. Most stream work done by the NRCS may be best termed rehabilitation, except where efforts are clearly focused on restoring a range of ecological functions and values to a defined historical condition.

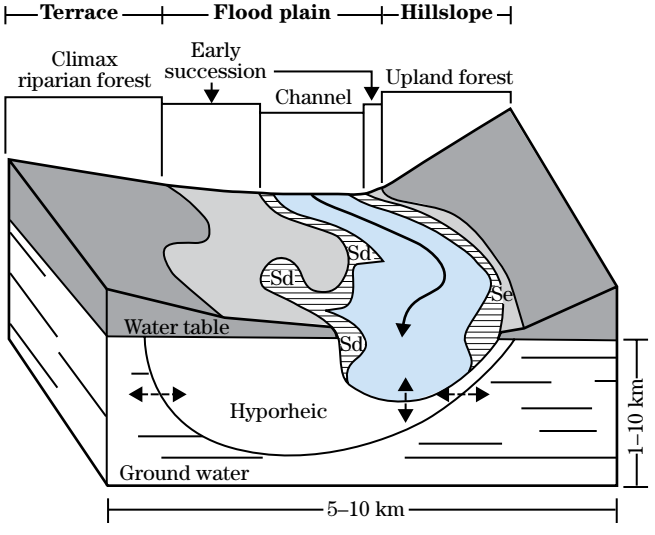
Restoration actions may be passive, simply to remove or attenuate chronic disturbances. Restoration may also be active, to intervene and install measures that are specifically designed to repair damages to the ecological structure and functions of stream corridors.

654.0103 Understanding stream corridor dynamics

Stream corridors are complex and dynamic. Natural or minimally altered stream corridors tend to be physically heterogeneous regardless of their size, with diverse patterns and types of habitats. Larger river corridors show more variation and complexity lateral to the channel, while smaller stream corridors tend to vary more longitudinally. Fluxes of energy, water, and materials throughout the stream corridor system create a dynamic three-dimensional (length, width, depth) mosaic of habitats and physical features (fig. 1–1 (modified from Stanford and Ward 1992)).

Length, width, and depth may also be identified as longitudinal, lateral, and vertical dimensions. These physical features change with time and contribute to the high level of biological diversity typical of stream corridors. The interactions occurring among the different elements of stream corridors are extensive for many of the plant and animal species that inhabit or use them. For example, bats living in the riparian zone eat aquatic insects living in the stream, while stream fishes eat both aquatic and terrestrial insects that thrive in riparian vegetation.

Figure 1–1 Cross-sectional view of generalized stream corridor showing three spatial dimensions in which stream corridor habitats are formed through time



Biota (the flora and fauna of a region) may reside in all habitats (riparian, inchannel, hyporheic, and/or ground water zone). The hyporheic zone is the saturated interstitial area beneath the streambed and in the streambanks, where surface and subsurface waters mix (fig. 1–1). Sd designates sediment deposition sites and Se is a site of bank erosion.

Human activities in stream corridors often simplify physical structure (by removing riparian vegetation). Human activities also may fragment connections, such as between the stream and its flood plain, preventing or diminishing natural processes important to many species. Projects designed to restore or maintain the inherent complexities of stream corridors, ecological linkages, and their physical connections are one solution to arrest the decline of aquatic and riparian species and to improve the Nation's water quality. Of course, projects can also address degraded/altered water quality, flow alteration, habitat enhancement, and other problems confronting stream ecosystems.

(a) Science and stream project design

The complex physical, biological, and social nature of stream corridors creates a challenge to professionals responsible for improving stream functions and conditions. Ward et al. (2001) suggested that scientists often misinterpret stream corridor processes because they usually study regulated systems—those already cleared of wood, dammed, channelized, revetmented, leveed or constrained by other types of hard structures. Systems with more intact natural flow regimes are characterized by high levels of heterogeneity, both in space and in time. From a human perspective, they are also less well behaved, less predictable, and increasingly rare in today's landscapes.

Much additional work is needed to understand the physical and biological processes typical of natural fluvial systems, or even partially altered systems, especially those with braided river channels. Scientifically validated models that help predict the physical behavior of stream systems are based mostly on single-thread meandering channels (Shields, Copeland, et al.

2003). Recent studies in stream ecology emphasize the importance of links between stream channels, riparian areas, flood plains, and hyporheic zones (Gregory et al. 1991; Naiman et al. 1992; National Research Council 1992; White 1993; Brookes, Knight, and Shields 1996; Huggenberger et al. 1998; Molles et al. 1998; National Research Council 2002).

Stream project designers rely on science and professional judgment as they develop stream improvement plans. Because professional judgment is often subjective, and applied experience may be limited, stream improvement project designs may become controversial when different disciplines are involved. Stream corridor projects that integrate the disciplines of fluvial geomorphology, hydrology, aquatic and riparian ecology, and hydraulic and geotechnical engineering are more effective at meeting multiple objectives that accommodate both economic and ecological considerations.

(b) Channel form and fluvial processes: Understanding stream corridor dynamics

Older science regarding physical aspects of streams contains many interesting observations about stream form. Earlier workers argued that form variables like the bed slope, channel width-to-depth ratio, meander wavelength, and bed-material size were related by functional relationships. Furthermore, they argued that disturbed channels (channelized streams) would not conform to these relationships, but would experience adjustment through erosion and deposition that eventually would return them to the appropriate (stable) form. Form studies naturally led to an effort to classify stream reaches based on form variables, and stream classification systems have been developed, ranging from very simple schemes with three or four categories (Shields and Milhous 1992) to those with several dozen categories (Rosgen 1996). More recent science has focused on physical, chemical, and biological processes that produce stream forms.

Since processes are driven by the dynamic variables of climate, tectonics, biological processes (plant succession, die-off, human population growth), the focus on processes has prompted reconsideration of the idea that without human intervention, fluvial systems will tend to approach an equilibrium or stable form. Although earlier workers knew the importance of processes in controlling forms and correctly identified most of the key processes (Leopold, Wolman, and Miller 1964), they often lacked the technology to monitor processes and develop mathematical descriptions. Recent advances allow scientists to collect large quantities of directly measured or remotely sensed data and use the data to build and revise their computational models. Much work remains to be done in understanding and predicting the behavior of stream ecosystems, but recent advances indicate that the best design work is usually based on general, analytical process-based approaches, rather than more subjective or site-specific empiricism.

(c) Using models to understand and manage complex systems

Models are descriptions of systems, which are collections of interrelated objects. An object is some elemental unit on which observations can be made, but whose internal structure either does not exist or is ignored (Haefner 1996). There are many types of models. Conceptual models describe the objects and relationships either with words or diagrams. Physical models, like plaster models of a watershed, are three-dimensional representations, usually at some relevant scale. Formal mathematical models represent objects and interactions quantitatively with equations and are typically implemented on computers.

Conceptual models are valuable frameworks for designing stream projects because they identify important components of the ecosystem and the processes that maintain it (Vannote et al. 1980; Schlosser 1987; Simon 1989). Project design teams can use these models to develop a common understanding of the system and to determine actions that are more likely to result in desired outcomes.

Applications of models

Model realism, precision, and generality should be considered when selecting a model (Levins 1966). Model realism and generality are important when using models as frameworks for understanding the stream (Haefner 1996). Models selected for predicting the outcome of a project (change in channel dimensions, change in fish abundance or community structure) must be precise and realistic, but they do not need to generally apply to all systems. Defense of a

model should include explanation of basic algorithms and calibration, as well as an independent validation. Models may not perform well in design of restoration or management actions at a site if they:

- fail to consider important components or processes within the system
- represent critical relationships too simplistically
- substitute professional judgment because data from the system of interest are not available
- use data from recent observations to project responses over decades to centuries
- address only part of the system or part of the life histories of the aquatic and riparian organisms
- do not account for disturbances and unpredictable processes that are important in the system
- do not account for site-specific geological conditions, or assume that they are constant in different watersheds

A good model produces results for existing data or observable conditions. Such a model may provide an expected result for a restored or altered condition, subject to model and data limitations. Remember that no perfect model exists, but models may show the relative differences or directions of changes in a stream ecosystem when alternative treatments or systems are considered.

654.0104 Fluvial systems

(a) Watersheds

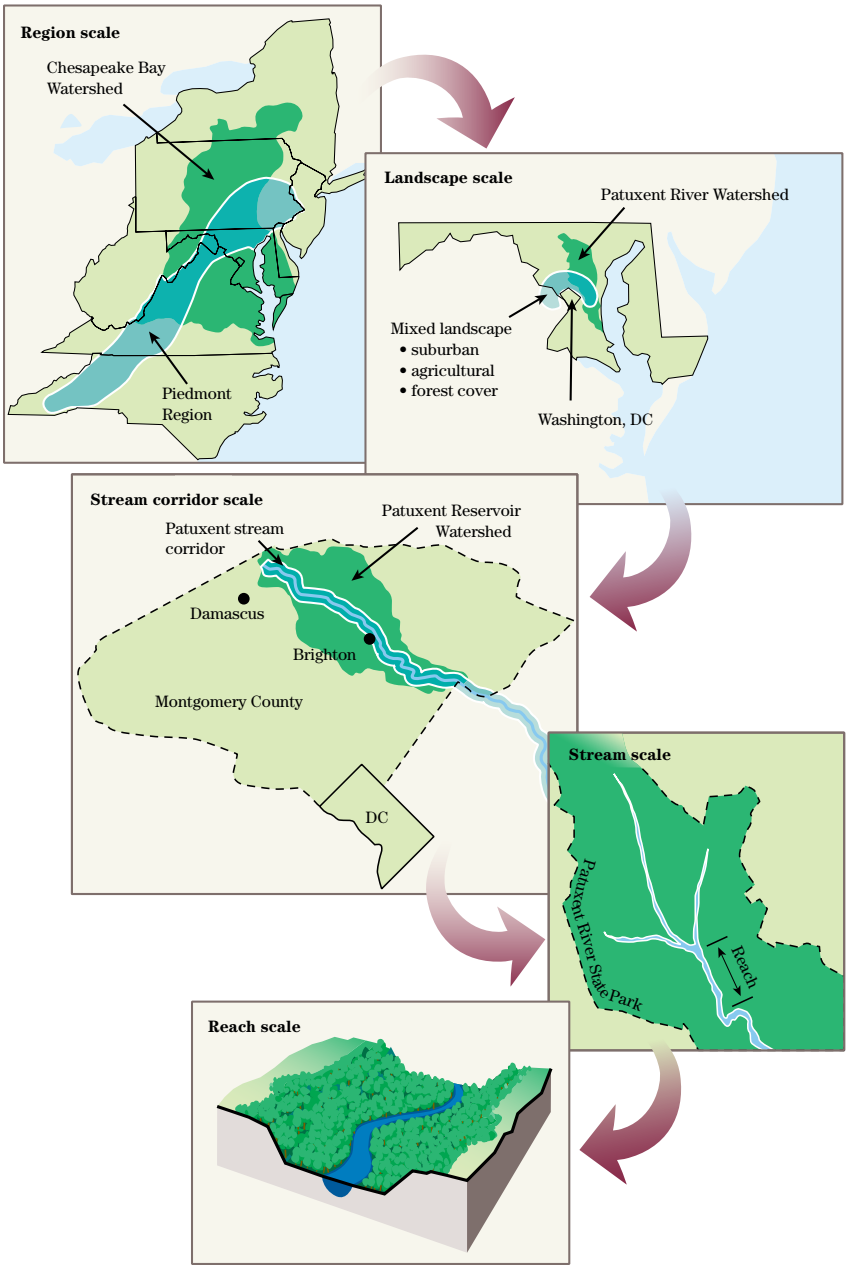
A watershed is a topographically bounded area of land that captures precipitation, filters and stores water, and regulates its release through a channel network into a lake, another watershed, or an estuary and the ocean. Watersheds are nested within one another, with larger watersheds composed of many smaller tributary watersheds, and these smaller tributaries drained by even smaller intermittent channels, ephemeral channels and rills. Watersheds are comprised of a mosaic of soil types, geomorphic features, vegetation, and land uses. If a watershed is divided into uplands and stream corridors, the uplands comprise most of its area (in most basins). Upland features control the quantity and timing of water and materials that make their way to the stream corridor. The environmental conditions of the stream corridor (such as water quantity and quality, riparian function, and fish habitat) are, therefore, linked to the entire watershed, and these linkages go both ways. For example, animals living primarily in upland habitat frequently rely on stream corridors for movement, food, cover, and water. Recent studies have also shown that marine derived nutrients carried up stream corridors in the tissues of salmon enhance the growth and survival of adjacent forest stands from which large wood in those rivers and streams originates (Helfield and Naiman 2003). Although stream project designers may have little or no control over how a watershed is managed, their plans and designs still should consider the past, present, and future status of watershed land use and historical watershed conditions.

Landscape consideration of watersheds: spatial scales

A landscape perspective is important when managing natural resources. The spatial structure of landscapes influences ecological and physical processes such as energy flow, material transport, and species dispersal within a landscape. These processes occur in all three spatial dimensions and over time within a watershed or river basin (Stanford and Ward 1992; Beechie and Bolton 1999).

Resource managers consider spatial structure of landscapes at very large scales (to analyze satellite imagery of large sectors of the Earth's surface) and at much smaller scales (to manage habitat in a stream reach), depending on the issue at hand (fig. 1–2 (FISRWG 1998)). Regardless of project scope, some consideration should be given to all scales. For instance, focusing only on the reach scale may overlook important issues that will dramatically impact the project. While many NRCS projects are applied on relatively short reaches, the stream's watershed should always be considered.

Figure 1–2 Spatial scales surrounding stream corridor ecosystems

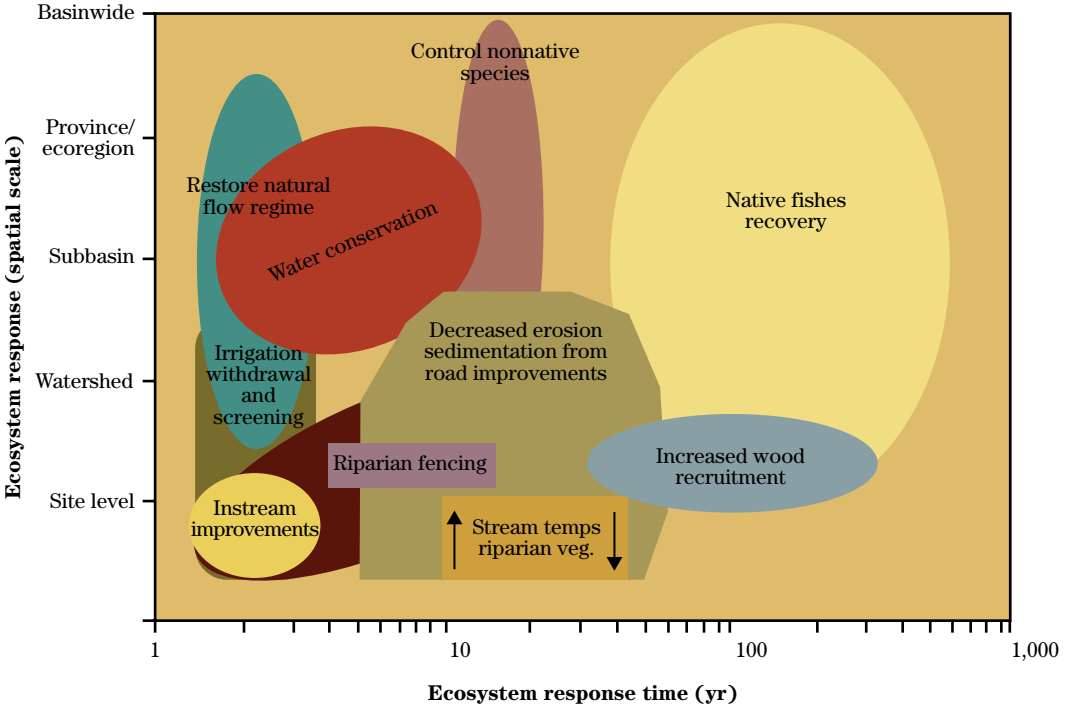


Rivers and streams, and the corridors through which they flow, may be considered long ribbons of aquatic habitat or riverscapes (Fausch et al. 2002). Riverscapes are also important to terrestrial plants and animals because stream corridors provide a transition between wet and dry habitats. Habitat is the place where an organism lives, and includes the range of environmental conditions (physical, chemical, and biological) it needs to grow and reproduce (Odum 1971). The spatial and temporal scales of a habitat are not fixed, but rather determined by the physical and biological processes that create it, the range of activities (home range) of the organism, its interactions with the biotic community, and the population dynamics of the species.

The habitat of a large or relatively mobile organism (Pacific salmon) contains the smaller scale habitats of smaller, or less mobile, organisms (aquatic insects and crayfish). This kind of organization implies a hierarchy of habitats and interactions that are formed by processes and nested in space. How long it takes for these multiple habitats to respond to stream restoration depends on the project's nature and scope and the dynamics of its landscape (fig. 1–3). Species responses to habitat modifications depend not only on the actual site work but also on the ecology of the surrounding watershed.

A hierarchical approach to stream design identifies the main spatial scale at which each ecosystem component influences the characteristics of the stream, but it does not imply that components at lower hierarchical levels are less important than those at higher levels. In fact, the connectivity of the stream environment involves feedback mechanisms by which smaller scale components may influence larger scale patterns and processes (DeAngelis, Post, and Travis 1986; Naiman 1988). Therefore, an effective stream restoration plan should consider factors affecting stream corridor processes at different spatial scales, from landscape to watershed to microhabitat. The plan should also consider factors that influence long-term population status and dynamics of aquatic species and the community of species with which they interact. This type of biological information is often available from researchers at

Figure 1–3 Spatial and temporal responses of ecosystem conditions to stream and watershed restoration actions



local universities or biologists of local fish and wildlife agencies. Focusing exclusively on maintaining local fish habitat by protecting or enhancing selected stream reaches may be ineffective in the long term because effects may be negated by changes in the stream system that occur at larger scales (Frissell and Nawa 1992).

Watersheds, stream corridors, and the dimension of time

Configurations of stream corridors change over time, as does the capacity of a channel to convey and retain water. Over geologic time all streams and their flood plains are active, often reworking entire valley floors by eroding and depositing sediments within their channels and the adjacent flood-prone areas. During smaller timeframes, pools within stream reaches are formed and maintained by erosion, and organic deposits and riffles are formed by deposition of sediments. Long-term trends in fluvial variables can be obscured by fluctuations over shorter timeframes. Stream projects are typically designed based on conditions that prevail over many decades, and they usually have projected lifetimes that do not exceed 50 to 100 years. Some geomorphologists have suggested that fluvial systems tend to reach a physical equilibrium or stability over periods that range from decades to centuries, and have termed this state "dynamic metastable equilibrium" (Schumm 1977).

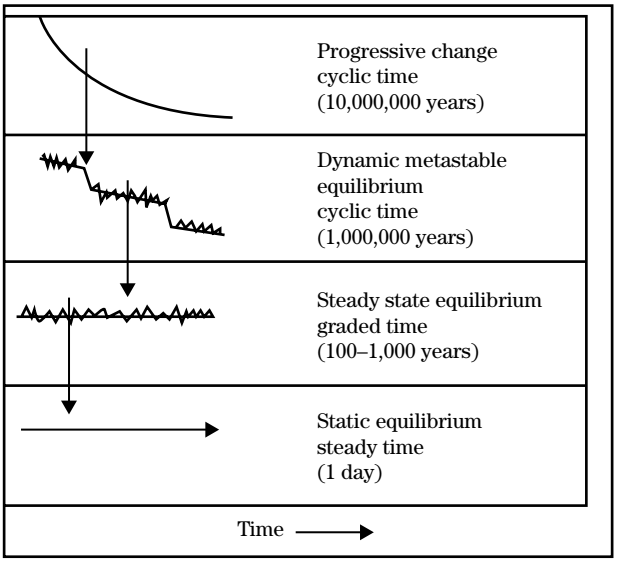
According to this theory, the physical characteristics of channels remain relatively constant during the equilibrium periods, and undergo rapid changes during short episodes that occur when the system exceeds some internal threshold (fig. 1–4). During periods of equilibrium, the channel is adjusted to inputs of water and sediment so that average channel width, depth, slope, and sediment grain size change little for any given reach. The channel transports the same amount of sediment that it receives and experiences no net erosion or deposition, although erosion or deposition may occur at smaller spatial scales, such as pool and riffle habitats. This concept of a dynamic metastable equilibrium has been useful in analyzing the response of stream channels to changes in the watershed, but it may not be valid for all streams. Nevertheless, designers attempt to select channel geometries (width, depth, slope, meander wavelength, bed and bank roughness, bed sediment size) that correspond to a stable condition defined by empirical or theoretical equations. At best, these constructed geometries will be appropriate during the periods of equilibrium. Harder, structural measures may be necessary to prevent changes across a threshold. Less intervention may be required if the stream is allowed to move in its flood plain, or if sediment production from the watershed can be managed.

Movement of water

Water that enters the watershed in the form of precipitation moves from the land into the stream channel as surface runoff or, if it infiltrates, enters as subsurface and ground water flow. Based on its pathways to the channel, streamflow is classified as stormflow or baseflow. Stormflow is the water from precipitation that reaches the channel over a short period of time (during and immediately after a storm event) through surface or subsurface routes. Baseflow is the water that percolates slowly through the ground before reaching the channel, where it maintains flow during periods of little or no precipitation.

Variability of flow is a key factor influencing the biotic and abiotic processes that determine the structure and dynamics of stream ecosystems (Poff and Ward 1990; Covich 1993). The path that water takes through a watershed determines the quantity of sediment and dissolved matter that reaches the stream. In general, the

Figure 1–4 Concepts of geomorphic equilibrium in stream corridor systems



amount of sediment suspended in the water column is greatest in small channels, gullies, and rills, intermediate in sheet flow, and lowest in ground water. Nutrient levels in water are often reduced as subsurface runoff percolates through riparian root zones. Once in the stream, nutrient concentrations are influenced by structures and processes that retard flow or promote retention, including vegetation and large wood within the channel, exchange with the hyporheic zone, or slowing of streamflow in meanders, sloughs, and flood plain depressions. In general, deposition and processing of nutrients are increased by longer flow retention time. Flood control systems designed to increase flow velocities and reduce net retention time of flood waters can greatly alter nutrient dynamics in stream corridor ecosystems.

Movement of inorganic sediments

Watersheds transport sediment, as well as water, but sediment transport usually varies as a function of discharge. As a result, changes in discharge magnitude and duration (downstream of a flood control dam) throughout the watershed magnify changes in sediment discharge. Alluvial channels (those with beds and banks made of materials readily transported by the stream, in contrast to threshold channels that are controlled by bedrock outcrops or materials too large for the stream to frequently transport) constantly adjust their geometry in response to the sediment load they receive.

Sediments range in size from clay particles, only a few microns in diameter, to large boulders, but a given stream is typically dominated by a smaller range of sediment sizes (sand to gravel or just fine sand). The types of stream organisms reflect the quantity and size distribution of sediments that move along or lie on the bed of the stream corridor. Aquatic organisms are quite sensitive to the size distribution of sediments (Shields and Milhous 1992). The frequency of bed-material movement and sediment-size distribution controls the size of microhabitats provided by interstitial spaces of bed substrates. Those streambeds dominated by uniform and small size particles (fine sediments) naturally sustain fewer species of aquatic insects (Benke et al. 1984).

Typically, species-rich stream substrates have particles of a wide range of sizes coarser than sand. The resulting high porosity of the streambed allows exchange of well-oxygenated water in the channel and within the hyporheic zone. This component of stream corridors is yet another zone of complex gradients and transitory boundaries over space and time.

Movement of organic material

Movement of organic material within a stream corridor system also occurs in four dimensions. The timing, quality, and quantity of organic matter transport through a stream system are related to streamside vegetation, channel complexity, aquatic food web dynamics, light intensity (from the sun), seasonal fluctuations in flow, and all of the aforementioned physical processes that influence the movement of water and sediments. Organic material includes parts of trees and shrubs, insects, nutrients in surface runoff, and aquatic organisms.

In forested landscapes, trees in upland areas become structural elements of stream corridors when carried to channels by landslides. In most landscapes, trees and/or shrubs border stream channels, even if the rest of the watershed is too arid or too developed to support woody species. Riparian trees fall into streams and flood plains during windstorms, floods, or bank sloughing and mass failures. Trees and other woody material are critical elements of aquatic ecosystems, affecting both the physical and ecological structure and function of stream corridors (Gregory, Boyer, and Gurnell 2003). The mobility of wood in streams and rivers is highly variable from site to site, depending on the size, slope, and configuration of the channel, as well as the characters of the wood (especially its size, morphology, density, decay rate, and extent to which it is lodged in the channel). Wood accumulations or single logs in unaltered, low-order streams may stay in place for decades or centuries, creating stable step-pool habitats.

Wood in large river channels often shifts with seasonal flows. This can cause considerable concern for river managers who are responsible for ensuring the safety of recreational users or for minimizing risks to infrastructure such as bridges. Still, the growing recognition that large wood is an important component in stream systems has led researchers and managers worldwide to develop innovative techniques for adding large wood to streams and rivers (NEH654 TS14J; Reich, Kershner, and Wildman 2003). Because woody debris can alter the flow path and shape of stream systems, programs exist to both remove and to retain wood in the stream.

654.0105 Channels

(a) Describing channels

The cross section of an average stream channel can vary greatly depending on water flow, amount of sediment carried by the water, and the geology of the terrain. The dimensions of a channel cross section between the sloped banks define the active channel and determine the amount of water that can pass through without spilling over the banks (fig. 1–5). The deepest part of the channel is referred to as the thalweg.

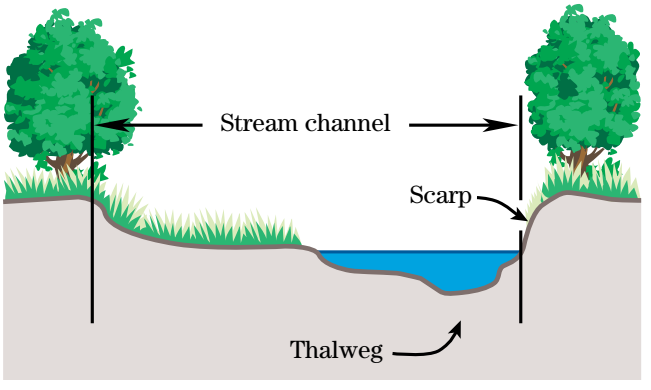
Channel slope is the average slope of the longitudinal thalweg profile. Flow velocity and stream power are proportional to the slope. Because these variables determine the rates of erosion, sediment transport and deposition, channel slope is an important controlling factor in channel form and pattern.

The form of a channel changes from its headwaters to lower elevations. In the steeper terrain of the headwaters, stream channels tend to be single and relatively straight. Channels of intermediate slope tend to maintain a single channel, but with increased sinuosity (curvature). Once in the depositional, flat slope of a watershed's lowlands, channels tend to develop multiple threads (or channels) and very high sinuosity. Multiple thread streams are further divided into braided and anastomosed streams. While static, braided streams are not often observed, they usually are formed in response to erodible banks, high bed-material sediment load, and rapid and frequent variations in stream discharge. The multiple channels of braided streams tend to be shallow and wide. In contrast, the multiple channels of anastomosed streams tend to be narrow and deep, because their banks are typically cohesive sediments. Anastomosed channels are often found on alluvial fans. While the description is a generalization, it should be noted that large parts of the country such as the Midwest have very flat channels, and these channels may either steepen or remain flat with distance downstream.

Natural stream channels are typically never totally straight and display different amounts of curvature or sinuosity. The sinuosity of a stream reach is calculated by dividing channel length by valley length. Sinuosity can also be calculated by dividing valley slope by stream slope. The degree of meandering is low if the sinuosity is less than 1.2, appreciable for sinuosities of 1.2 to 1.5, and high for sinuosities above 1.5. Sinuosity is related to both streamflow and gradient. In general, low to moderate levels of sinuosity are found in the headwaters, and these levels increase as the stream enters the flat and broad valleys downstream.

Independent of their form, stream channels are rarely uniform in depth and tend to have alternating, regularly spaced, deep (pool) and shallow (riffle) areas. Pools typically form where the thalweg approaches the outside bank of the channel at bends, whereas, riffles usually form between channel bends in the zone where the thalweg migrates from one side of the channel to the other. Streambed composition affects the pool and riffle characteristics, as well. Streams with coarse substrates, gravel to cobble-size particles, tend to have evenly spaced pools and riffles, to the extent that pool-to-pool distance is approximately five to seven times the width of the channel at bankfull discharge (the discharge that fills a stable alluvial channel up to the elevation of the active flood plain). In such systems, cobbles and large gravels accumulate in the riffle areas, while smaller particles tend to deposit in the pools. On the other end of the spectrum, streams with sand and silt-dominated substrates do not form true riffles due to the absence of coarse grain sizes. However, they still have evenly spaced pools connected by shallower runs or glides.

Figure 1–5 Cross section of stream channel



For assessment and design, it is useful to categorize stream channels as threshold channels or alluvial channels. Threshold channels have beds and banks that are not easily mobilized by the stream or river flow, while alluvial channels are continuously or frequently reshaped by erosion and deposition. Alluvial channels are shaped constantly by their streamflow. Differentiating between these two types of channels is subjective, since almost all channels have mobile boundaries under extremely high flows. Alluvial channels may be preliminarily assessed with one dominant discharge, but are assessed under a range of expected flow conditions. Threshold channels are so called because the flow forces during a given discharge are at or below the level (threshold) needed to move particles on the channel bed or banks. Typically, threshold channel boundaries are assessed for mobility under design flow conditions. Threshold channels occur when there are bedrock outcrops, or when coarse boundary materials are remnants of earlier fluvial processes, such as glacial outwash. Threshold channels have beds and banks that are mobilized slowly by the streamflow or riverflow, provided there are no human-induced changes in the watershed and stream system.

In alluvial channels, there is a frequent exchange of channel boundary material with the flow. Meander migration in stable threshold channels might be a few feet or less annually, while in alluvial channels it could be many feet of movement in response to a single stormflow. The distinction between alluvial and threshold streams is addressed in more detail in NEH654.07.

(b) Key physical variables

Stream channels require up to 13 variables for a complete physical description, but only three governing equations are known, and only about six of the variables are fixed by site conditions (table 1–1 (Hey 1982, 1988)). See FISRWG 1998 for a fuller description of the meaning of each of the variables. Since there are more unknowns than equations, channels are indeterminate systems. For threshold channels, most variables are fixed by site conditions or by the choice of the designer, but alluvial channels can adjust their geometry in several dimensions. Existing models are not capable of accurately predicting long-term behavior of channels even when water and sediment in flows are specified.

Table 1–1 Degrees of freedom and governing equations

Type of channel	Governing equations	Fixed variables	Independent variables	Degrees of freedom	Dependent variables
Threshold, fixed bed, no sediment transport	Continuity, flow resistance	S, W, d_m, λ, Δ, p, z	Q, D, D_r, D_l	2	V, d
Planform and width are fixed, bed is mobile	Above, plus sediment transport	W, d_m, λ, Δ, p, z	Q, Q_s, D, D_r, D_l	3	V, d, S
Fully alluvial	Above equations, plus six additional process equations needed to render the system determinate, but these are generally not available		Q, Q_s, D, D_r, D_l, S_v	>3	All but independent variables

V = mean flow velocity; d = mean depth; S = bed slope; W = width; d_m = maximum flow depth; λ = bedform wavelength; Δ = bedform amplitude; p = sinuosity; z = meander arc length; Q = water discharge; Q_s = sediment discharge; D, D_r, D_l = characteristic sizes of bed, right and left bank sediments, respectively; S_v = valley slope.

Predicting the type of planform a channel will develop (straight, braided, or meandering) or the rate of lateral erosion of streambanks (meander migration) is difficult at best.

(c) Using conceptual models to understand stream channel dynamics

Conceptual models often link structural properties of stream channels with critical processes that operate within them in a qualitative fashion. An example of a process based conceptual model is the incised channel evolution model (CEM) (Schumm, Harvey, and Watson 1984; Simon 1989), which describes changes in straightened channels that are undergoing headward incision. Presented in greater detail in NEH654.03, this conceptual model is based on observations of channels in watersheds undergoing systemwide disturbance.

Although the model does not allow prediction of the magnitude or rate of channel changes, it does link processes and allow qualitative prediction of outcomes (channel widening, progression of incision, and incision control options). The CEM is idealized, and any processes or conditions that are different than the fundamental conditions assumed in the model may alter the outcomes. For example, channels may change the trajectory or location of incision if boundary conditions are changed or pulses of sediment are supplied by incising tributaries. Another important feature of the CEM is that it allows resource managers to differentiate between local instabilities (erosion of the outside of a particular bend due to impinging flow) and systemwide instabilities (increased peak flows related to increases in impervious surfaces), which are much more complicated to control.

Conceptual models are widely used by stream ecologists, as well. Major conceptual models in stream ecology include the river continuum concept (Vannote et al. 1980), flood pulse concept (Junk, Bayley, and Sparks 1989), nutrient spiraling concept (Newbold et al. 1981), natural flow regime (Poff et al. 1997), patch dynamics concept (Townsend 1989), serial discontinuity concept (Ward and Stanford 1995b), and ecosystem perspectives of riparian zones (Gregory et al. 1991). Such models help organize ecological thinking about streams and rivers, much the same as the CEM helps hydrologists and geomorphologists understand the process of incision in stream channels.

As an example, the river continuum concept (RCC) suggests that the physical form of streams and rivers is generally predictable from headwaters to large flood plain rivers. In all these cases, the RCC provides a conceptual model that helps people think about how they expect the stream ecosystem to be structured and what processes are most likely to occur along the network from headwater streams to large rivers. Such conceptual models are useful in assessing stream degradation and setting restoration goals because they describe how physical habitats are related to aquatic community structure and the ecological processes that are important to them.

(d) Using classification systems to describe channels

Although every stream is a unique combination of watershed characteristics, channel boundary conditions, and hydrologic and climatic regimes, people have long attempted to generalize their knowledge about streams by developing classification systems (Hawkes 1975; Bryce and Clarke 1996; Rosgen 1994; Frissell et al. 1986; Montgomery and Buffington 1993a, 1993b; Thorne 1997). Environmental classification systems are thoroughly reviewed by Zonneveld (1994) and stream classification systems by Kondolf (1995), Kondolf and Downs (1996), and USDA NRCS (2001c). Classification systems generalize field observations, facilitate communication, and identify dominant groups of processes. Classification systems are useful tools for communicating descriptive information since it saves time to simply state that a stream is type X, rather than specifying values for all of its component variables.

Overly simplistic use of categories can lead to misunderstandings and cookbook approaches, rather than an understanding of how a stream reach is functioning. Some workers have suggested that stream classification may be used to develop restoration prescriptions or to predict changes in morphology or ecology. Extremely simple classification systems include those based on flow habit (ephemeral, intermittent, perennial), planform (straight, braided, or meandering), or boundary mobility (threshold or alluvial). Others include physical variables (bed-material size, slope, sinuosity, channel width, valley shape) or biological

characteristics (riparian vegetation, insect communities, or fish communities). Most stream classification systems are based on morphological or form variables (how streams look), rather than process variables (how streams behave, for example, widening or degrading). It is always easier to determine form, rather than process because processes act over time. Therefore, process determinations require sequential observations, historical data, or some surrogate, such as a space-for-time substitution. Unfortunately, fluvial systems are complex (threshold responses, variable responses, biological adaptation) and frequently changing (climate, streamflow, tectonic events, land use changes). It is difficult to accurately predict future stream behavior from current morphology.

Accordingly, a classification system alone should not be used for determining the type, location, and purpose of restoration activities (FISRWG 1998). Some have proposed the idea of a diagnostic or weight of evidence approach as an alternative to process models, evolution or conceptual models and classification systems (Ward and Trimble 2004).

(e) Using mathematical models to predict channel responses

Quantitative predictions usually require a series of numerical calculations. Like most environmental systems, stream and watershed systems are complex, so the series of numerical calculations needed to make a prediction have been incorporated into a wide range of different models. The components of mathematical models are described in measurable units, and the relationships and processes within the models are represented by explicit mathematical formulas. Most complex mathematical models require specialized training in the scientific discipline that is being modeled (phytoplankton or sediment transport). Some complex mathematical models include user interfaces that ask specific questions and make it possible for an informed resource specialist to apply the model. Even in such cases, users of complex mathematical models should be aware of the context for which the model was developed, assumptions within the model, and data required to run the model. As a result, mathematical models are less popular for general application in stream project designs than broader conceptual models. However, the quantitative projections and predictions of mathematical models can greatly enhance the design of a project if the appropriate expertise is available to the design team.

Quantitative models of streams have become quite complex, and normally require specialized academic training for successful application. Models may be classified based on the characteristics they simulate and the way they handle temporal and spatial variations. Models of streamflow that predict the depth and velocity in a stream channel for a given geometry and discharge are most common, but models that include sediment movement, water quality, and some index of physical habitat quality are also widely used.

Stream ecosystem models simulate changes in habitat, biological populations, community structure, and ecological processes for stream ecosystems. Water quality (dissolved oxygen, temperature, suspended sediment, nitrogen and phosphorus) for streams and rivers are widely modeled for different regions and land use practices (Brown and Barnwell 1987; Lisle and Lewis 1992). Phytoplankton and benthic algae abundance along streams have been modeled for both streams and rivers (Brown and Barnwell 1987; McIntire 1973; McIntire et al. 1996; Stevenson and Smol 2002). Macroinvertebrate community structure and relationships to water quality and habitat have been modeled (Reynoldson et al. 1997; Karr et al. 1986; Hawkins et al. 2000). Hundreds of models are used around the world to relate the abundance of fish populations to physical habitat availability (Armour, Fisher, and Terrell 1984; Fausch, Hawkes, and Parsons 1988; Lee 1991). A recent review of models of large wood in streams identified 14 simulation models developed for streams and rivers (Gregory, Meleason, and Sobota 2003). In addition, several models simulate entire stream ecosystems (McIntire and Colby 1978; Newbold et al. 1983). Regional resource agencies often provide expertise to cooperating agencies and public groups to allow the application of more complex models in stream project design. A major limitation, particularly in models, is the poor linkage between ecology, water quality, and geomorphology. Multiple stressors are at work impairing health of many stream ecosystems, and it is often difficult to establish which are the most important.

A parametric model has parameters that must be estimated in some fashion. An empirical model contains any empirical relationship, one that is based on data. An empirical model is based, at least in part, on observed data, rather than a thorough understanding

of the underlying physical principles. A lumped model describes processes on a scale larger than a point, while a distributed model describes all processes at a point then integrates processes over space and time to produce a total system response (Haan, Barfield, and Hayes 1994). A stochastic model is one whose outputs are predictable only in a statistical sense. Repeated use of a given set of model inputs produces outputs that are not the same, but follow certain statistical patterns (Haan 1986).

Model quality and capability vary widely. Several fundamental types of mathematical models are:

- Steady-state models predict conditions that occur for a given set of boundary conditions. For example, a flow model might predict the water surface elevation, given a fixed channel geometry and a constant flow.
- Unsteady models predict variations that occur with time such as during the passage of a storm hydrograph by dividing such an event into a series of steady-state time steps. Complex unsteady models have feedback loops that allow channel boundaries or other key variables to respond to inputs and change between time steps.

From a spatial perspective, models may be one-, two-, or three-dimensional:

- One-dimensional models only consider forces that occur in one (usually the streamwise) direction. Velocity and other stream properties may vary upstream and downstream, but not from bank to bank and not from the bed to the water surface. A common example is HEC–RAS.
- Two-dimensional models are usually depth averaged. They simulate variation in the horizontal plane, but assume no variation in the vertical.
- Three-dimensional models simulate variation in all three directions. Model cost, size, and complexity increase by roughly an order of magnitude with each added dimension.

654.0106 Key processes affecting stream corridor ecosystems

Stream channels are dynamic. Therefore, stream project planners and designers must be able to identify and understand key processes. Physical processes include hydrologic and geomorphic processes. Both biological and physical processes occur longitudinally, laterally (across the corridor), and vertically (above and underneath the corridor) over time. Abrupt changes in stream channels and their riparian areas by natural features (geologic differences along the river, vegetative changes related to geology, soils, or regional climate) and human activity (land conversion, urbanization, agriculture, forestry) often disrupt ecological processes.

(a) Physical processes

Longitudinal adjustment

The longitudinal profile of a stream typically displays the effect of headwater erosion and downstream deposition over long periods of time. In the shorter timeframe, bed profiles may become locally steeper or more gradual, or they may exhibit aggradation (deposition of sediments) or degradation (channel deepening), as supplies of sediment and stream power fluctuate in response to changes in discharge. Since the energy gradient that drives the fluvial system is normally equal to the bed slope, other channel variables are quite sensitive to slope changes. Ecological impacts of slope change are generally related to changes in water velocity or sediment transport. For example, degradation of stream channels can lead to a lowering of the water table and consequent dessication and loss of riparian vegetation. Severe aggradation of stream channels decreases water depth and flow and can result in excessive temperatures or decreased dissolved oxygen during summer. Aggradation can also cause a lack of cover and smothering of coarse-grained substrates. During periods of low flow, aggraded stream channels may be too shallow to allow movement of fish. Excess sediment is most damaging where aquatic life is not adapted to these conditions.

Lateral adjustment and bank erosion

Mean bank erosion rates vary from a few millimeters per year to 800 meters per year (Lawler 1993). Bank erosion is the result of about 10 processes, several of which are usually operating on a given site. Processes may be loosely grouped into hydraulic processes (removal of sediment by flowing water) or geotechnical processes (collapse, slumping, or sliding of sections of bank due to gravitational forces exceeding resisting forces). Hydraulic processes include scour of particles or aggregates of bank material. Fluid shear forces tend to be greatest for the bank toe, but erosion can occur anywhere on the bank, especially if not well vegetated. When the bank toe is eroded, often the upper bank is undermined, cantilever-type geometries result, and banks ultimately fall into the channel. Hydraulic processes also include erosion of the bank face by overbank flows that concentrate into rills and gullies, sometimes called valley trenches.

Geotechnical failures usually occur when large blocks of bank material fall into the channel from high, steep banks. These failures are often observed when erosion has lowered the channel bed, increasing bank height and angle. Shallow ground water flow toward the channel often facilitates failure by increasing soil weight, decreasing soil strength, creating voids by piping erosion, and lubricating planes of weakness. Geotechnical failure requires that banks be high and steep enough to create gravitational forces that exceed soil strength, which varies with soil type, soil moisture, vegetation, and other site-specific factors. A high bank may be only a few feet for noncohesive soils and more than 20 feet for cohesive soils.

Ideally, threshold channels (those for which hydraulic forces are at or below the threshold needed to initiate motion of boundary sediments) resist hydraulic erosion processes. Since alluvial channels are constantly shaped by streamflow, their banks are more mobile than threshold channel banks. However, rates of bank retreat vary widely from point to point along the bank and through time. For example, as an alluvial channel meanders freely across the flood plain, the current direction may shift, forcing the flow onto a section of bank that has been stable for years. A period of rapid bank erosion ensues. All channels experience some degree of bank erosion. Most sediment inputs are relatively small and are incorporated into stream corridor processes, such as flood plain development. Human activities can accelerate or decelerate bank erosion rates by orders of magnitude. In both cases, ecological impacts may be significant. Increased bank erosion can lead to deposition of clay and silts that is especially damaging to fish spawning habitats and habitats of benthic macroinvertebrates that live in the interstitial spaces of cobble and gravel substrates.

Channel avulsion and flood plain construction

When bank erosion and longitudinal adjustment occur at a large scale, rapid changes in channel planform (avulsions or cutoffs) occur. These events typically occur during floods or high flows and trigger an episode of rapid local change in the region surrounding the avulsion. Typically, such events produce shorter, steeper channels in the short term, with erosion of upstream reaches and deposition in former channels and downstream. Channelization of streams often proceeds by construction of a series of artificial cutoffs to straighten the channel, with extreme impacts on channel stability if control structures or erosion resistant lining are not provided.

The impact of natural cutoffs is less than that of channelization because natural cutoffs normally occur one at a time, so that the overall length (and average bed slope) of a long reach does not change much. Since avulsions often trigger periods of large-scale, unpredictable instability, erosion control structures are often designed and placed to prevent them. However, in unmanaged stream corridors, major avulsions provide habitat complexity and diversity for aquatic species. Sloughs and oxbows that are abandoned channels provide low-energy habitats and refugia from the sporadic or seasonal fast water in the main channels. Newly deposited sediments in these areas and on the outside of meander bends provide substrate for pioneering plant species, while erosion topples older riparian forest communities and induces recruitment of wood to the stream. Flood plains that are periodically reworked by avulsions tend to be rich mosaics of plant communities of several successional stages. Over long periods of time, an unmanaged stream corridor will migrate back and forth across the entire valley, generally increasing the elevation of the flood plain through depositional processes. This generalization has exceptions such as deeply incised headwater streams or streams experiencing a drop in base level.

Sediment transport

Sediments are transported and sorted during high flows, so flow regimes are critically important to

aquatic species. Unaltered streams receive sediments from their watersheds, beds, and banks and subsequently sort these sediments by size into well-defined spatial patterns. Coarse sediments (larger gravel and cobble) occur along the axis of highest velocity and greatest depth, and finer sediments are deposited along the margins of stream channels or in the velocity shadow of larger inchannel obstructions (logjams, large boulders). Channel beds often feature a surface layer of coarse particles (armor) that is only one- or two-grain diameters thick, with a more heterogeneous mixture of sediment sizes underneath. Bed sediment size distribution or sediment texture is one of the most dynamic aspects of a fluvial system, changing rapidly in response to changes in other variables (channel bed slope, discharge, or amount of large wood). In turn, sediment transport is very sensitive to bed sediment size. Benthic organisms such as insects and small plants (periphyton) that live on the surface of coarse sediments are sensitive to changes in sediment size, sediment porosity, and the frequency of bed sediment movement. Biota from regions with naturally occurring fine-grained substrates are less sensitive to sediment than biota from regions with coarser-grained substrates. Fish that reproduce by laying eggs in gravel are particularly sensitive to changes in particle size, as they must rearrange stones to create redds. Also, well-aerated, intragravel flow is important for egg survival and larval growth.

Sediment sorting processes are less evident in fine sediment, where deposited in flood plain depressions, sloughs, and oxbows and within eddies along channel margins. These silty and clayey deposits provide media for colonization by terrestrial macrophytes when they are exposed by falling stages, or if they are low enough to remain under water, provide substrate for burrowing types of macroinvertebrates not found in the sandy main channel bed.

(b) Ecological processes

Energy flow and nutrient cycling

The flow of energy and nutrient dynamics in aquatic ecosystems occur in all dimensions and is influenced greatly by the physical dimensions of the stream channel. In turn, these processes strongly influence the community structure of stream ecosystems and the ecological processes along their longitudinal network. In small headwater streams, channels are narrow and shallow. In forested landscapes, inputs of solar radiation to the channel are, therefore, generally very small, and inputs of organic matter from the terrestrial ecosystem are relatively large.

Aquatic invertebrate communities are dominated by organisms that shred the larger terrestrial inputs (leaves, twigs) or by collectors that feed on the fine particles transported in from the terrestrial ecosystem or created by the shredding of large particles into smaller particles in the stream ecosystem. Since streams get larger as they flow downstream, channels generally become wider and deeper. Openings in the riparian canopy over the stream increase the inputs of solar radiation, causing increased production of algae and vascular aquatic plants, reducing the relative inputs of terrestrially derived organic matter. As a result, aquatic invertebrates are dominated by organisms that scrape algae off the streambed and collectors that feed on small particles of organic matter. The change in the longitudinal gradient of streams is also the primary factor driving hyporheic exchange flows (Harvey and Bencala 1993). This change creates unique physical, chemical, and hydrologic environments in streams and riparian zones, providing a diversity of habitats for many specially adapted macroinvertebrates (Stanford and Ward 2001).

The lateral exchange of water between a river and its flood plain is the driving force for nutrient cycling and the dynamics of the flood plain biotic community. Primary productivity of flood plain habitats is closely tied to hydroperiod, the periodic or regular occurrence of flooding or saturated soil conditions (Marble 1992), or the ratio of flood duration divided by flood frequency over a given period of time (Mitsch and Gosselink 1986). Productivity is greatest in wetlands with pulsed flooding (periodic inundation and drying) and high nutrient input, and lower in drained or permanently flooded conditions and low-nutrient water. Riparian wetlands may also influence stream channel morphology and flows, buffering the stream channel against the physical effects of high flows by dissipating energy as waters spread out onto the flood plain. Alternately, as streamflows recede, riparian wetlands provide water storage, slowly releasing water back to the stream through subsurface transport, thereby influencing stream baseflows.

Recruitment of large wood

Wood is important from headwater streams to large rivers and estuaries (Maser and Sedell 1994). Wood

in the stream provides structure and organic matter that creates and enhances habitat diversity, and is a food source for many riparian and aquatic organisms (Boyer, Berg, and Gregory 2003).
Wood in streams also increases channel roughness and habitat complexity, triggers the formation of islands, and forms dams that trap leaves, twigs, and fine sediments. Fine particulate organic matter (particles smaller than 1 mm in diameter) retained by large wood pieces provides food for insects and other aquatic invertebrates.

Small, steep headwater streams with wood input often contain a series of step pools formed by fallen logs that cross the channel and trap smaller pieces of woody material and leaves. At the other end of the spectrum, some large rivers have been completely blocked by natural accumulations or rafts of large wood that dominate stream corridor processes (Triska 1984; Collins, Montgomery, and Sheikh 2002). Natural channel widening and bar formation associated with wood obstructions allow development of the short, braided reaches and secondary channels that are important spawning grounds for salmon and trout in the rivers of the Pacific Northwest. In the sand-bed coastal plain rivers of the southeastern United States, wood also provides important habitat for invertebrates and provides fish with a source of food (Wallace and Benke 1984). Therefore, in many streams and rivers throughout the world, fish abundance and diversity depend on accumulations of large wood.

Wood recruitment processes are complex since they involve site-specific variables (size, species, density, and condition of riparian trees, bank geometry, and erosion) and stochastic events (tree death, tree blow-down, high flows, bank failures). Continuously submerged wood resists decay for centuries, but wood subject to alternate wetting and drying may disintegrate and decay in less than a decade, with exact rates dependent on species and regional climatic factors (NEH654.14 and NEH654 TS14J). Transport of fallen wood is inversely related to the ratio of wood length to channel width; logs with lengths greater than channel width may lodge in place for a lengthy time period.

Removal of wood is perhaps the most widely practiced type of stream channel management, and the practice of removal (de-snagging or clearing and snagging) along with deforestation and removal of beaver have left many streams with only a trace of the large wood that existed previously. For a full description of the effects of wood in streams and rivers, see Gregory, Boyer, and Gurnell (2003).

654.0107 Stream corridor habitats

Stream channels are usually the focus of stream restorations, but how these channels are ecologically linked with other parts of the landscape, watershed, and corridor should be considered and addressed. The dynamic nature of streams and their response to floods and other disturbances create many diverse habitat types and conditions, both in the stream and along its corridor. These habitats and the processes that occur among them affect each other dramatically, adding to the habitat complexity and species interactions in the stream and riparian area. Stream corridors support a disproportionately rich biological community, relative to the rest of the landscape.

Confounding this ecologically valuable richness, however, are the challenges that river and stream processes such as flooding present to humans. Add to these the many human demands on streams as water supplies, and as agricultural, recreational, and urban development sites, and managers feel compelled to take actions that compromise the ability of watersheds to sustain important ecological functions of habitats. Stream corridors provide filtering, buffering, retention, and conduit functions for water, sediment, wood, chemical compounds, seeds, and habitat for aquatic and riparian organisms. Therefore, maintaining multidimensional connectivity along a stream corridor is important to maintaining the species and habitats within them.

(a) Stream channel habitats

Instream habitats are as diverse as the systems that form them. High quality stream habitats are a mosaic of great spatial diversity created by various combinations of water quality and quantity, water depth, velocity, large wood substrates, mineral substrates, riparian vegetation, and the organisms that inhabit stream corridors. For example, shallow, swift flow over coarse bed material occurs in riffles that are often found at the inflection points of meanders. These habitats are important for stream invertebrates and as spawning sites. Generally speaking, aquatic organisms need what most organisms need to survive: clean water, oxygen, a steady food source, a place to hide or find refuge, and a place to successfully reproduce and grow to adulthood. Some aquatic organisms such as microscopic zooplankton live almost entirely in the water column; others, such as fish, use the water column and bottom substrates. Still others rely on the interstitial spaces of hyporheic habitats in and below the streambed.

Considerable research over the last several decades has described the importance of hyporheic zones to many alluvial stream corridor systems. These functions include:

- regulation of stream temperature by ground water upwelling
- water retention and storage which can reduce peak flows during floods and sustain baseflows during dry periods
- habitat creation, especially for aquatic invertebrates such as crustaceans, and vertebrates such as larval fishes
- buffering and filtering nutrients from streamflows and ground water
- aquifer recharge
- nutrient enrichment

Most species use a variety of habitats during the course of their lives, some moving upstream or downstream, others into and out of the flood plain, a few into or out of the substrate, and still others to and from the ocean, all depending on the season, their age and physiology, and the conditions they face in their habitats. The complexity of their life cycles requires comparable complexity in their habitats and connections among them to allow movement at the appropriate time. To sustain aquatic communities, stream corridor project designers should consider the habitat needs of aquatic organisms throughout their life stages and the physical and ecological processes that provide them.

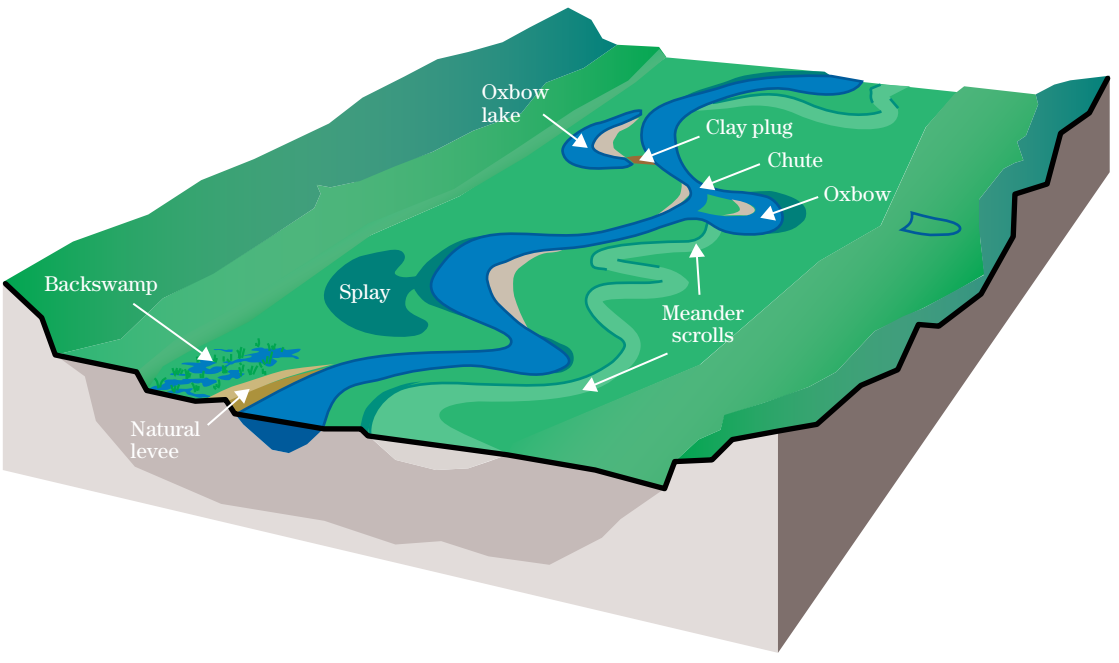
(b) Riparian and flood plain habitats

A stream corridor is comprised of the stream channel and its riparian zone. The riparian zone forms an ecotone or transitional zone between the stream and uplands and provides value, both in productivity and biotic diversity, far greater than its relatively small

area would indicate. Riparian zones may or may not include flood plains, depending on the valley form of the stream corridor. In relatively wide stream corridors, flood plains are prominent components of the riparian zone. Whereas stream channels have often been the focus of stream restoration projects over the past few decades, project designers today recognize the links between the stream channel and its riparian areas and flood plain or riparian wetlands. Projects that consider these linkages are becoming more common (Middleton 2002). Flood plain/riparian wetlands, which include swamps, oxbows, sloughs, ponds, backwaters, abandoned channels, and flood plain lakes, usually are remnants of historic river channels or shallower depressions created by scouring and sediment delivery associated with flooding (fig. 1–6). Riparian wetlands receive water from the stream during overbank flow events; however, runoff from adjacent uplands, ground water seepage, and precipitation can be significant or dominant contributors to wetlands depending on regional climate, soils, and other variables. During overbank flows, these wetland habitats are connected to the river by surface water, but as a stream recedes, water is trapped in low lying areas forming seasonal, isolated wetlands varying in size, shape, permanence, and significance for aquatic species.

The occurrence and relative importance of riparian wetlands in a stream ecosystem changes with stream gradient. High gradient streams are steep with small riparian zones, and few developed riparian wetlands. In contrast, lower gradient streams have broader riparian zones and flood plains characterized by more predictable hydroperiods and more extensive riparian wetlands systems. In these systems, flood plain wetlands can contribute significantly to stream ecosystem productivity and function.

Figure 1–6 Flood plain features important for aquatic species



The hydrological characteristics of wetlands vary from permanently flooded backwaters to wetlands that have overland sheet flow during floods, to ephemeral, isolated pools. In lower gradient streams, plants, invertebrates, and vertebrates have evolved survival strategies that depend on occasional or seasonal flooding or ponding. Some macroinvertebrates complete their entire life cycle in these habitats, persisting in seasonal wetlands in a drought resistant form, such as an egg. Vertebrates (fish, amphibians, mammals, and birds) frequently make seasonal movements into flood plain wetlands (from the stream, wetlands outside the flood plain, or surrounding uplands) and time key periods of their life cycle (breeding, rearing young, or migration) to riparian zone ponding and flooding. Riparian wetlands are also important habitats in stream corridors as they provide low velocity refugia for organisms that benefit from stream processes, but cannot survive for long periods in moving water, such as frogs. Temporary and seasonal flood plain wetlands provide vernal pool habitat for amphibians and other organisms. Importantly, simply returning water to a stream's flood plain is not adequate for reestablishing function for all organisms, because each may be dependent on a specific timing, depth, duration, or frequency of flooding.

Just as riparian wetlands can influence stream function, anthropogenic changes in stream channel morphometry can influence the function of a flood plain wetland. Riparian wetlands are often filled or isolated from the stream by constructed levees, channel incision, or channel straightening projects. Physical isolation changes the hydroperiod and precludes access to the flood plain by many stream obligate organisms (fish). Channelization can result in streambed incision that changes the frequency of overbank flows, and therefore, the hydroperiod of flood plain wetlands. In urban areas, stream incision causes loss of riparian wetlands by lowering the flood plain water table. Similarly, channel stabilization usually precludes avulsive processes (a sudden change of course of a stream) that can form new flood plain wetlands and create complex mosaics of different successional stages. This latter point is critical to maintaining habitat diversity in the riparian zone. Therefore, stream restoration projects that produce normal overbank flooding regimes can be more successful at restoring stream ecosystem function and the species that depend on them.

654.0108 Disturbance and response in aquatic ecosystems

(a) Definitions of disturbance

Fluvial systems can experience abrupt changes in environmental conditions that are often considered to be disturbances. However, simple variation in physical (discharge, sediments) or environmental (temperature, dissolved oxygen) conditions are inherent in any system and should not be considered disturbances without the context of their effects on ecosystems. The most widely accepted definition of ecological disturbance is: "… *any relatively discrete event in time that disrupts ecosystem, community, or population structure and changes resources, substrate availability, and the physical environment*" (White and Pickett 1985). Fluvial ecosystems are inherently variable and can be naturally subject to wide ranges in flow conditions.

Stream ecologists have limited this general ecological definition of disturbance to include only those events characterized by frequency and intensity that are outside a predictable range (Resh et al. 1988). These definitions separate disturbances from inherent variation in terms of (1) the disruption of a biological system, (2) the change in resources or physical environment, and (3) rarity or unpredictability outside a range of commonly observed variation. It is important to recognize that disturbances are not just events that cause decreases in abundance of organisms. In these definitions, any event that disrupts—either increasing or decreasing—the structure of the ecosystem, community, or populations of species is considered a disturbance. For example, abrupt releases of fertilizers that cause an increase in algae would be considered a disturbance.

The biological communities and physical form of a stream, river, riparian corridor, or watershed exhibit the influences of small- and large-scale disturbances that have occurred (fig. 1–7 (FISRWG 1998)). Natural disturbances include floods, fire, drought, or storms. Disturbances induced by land management actions are more aptly called perturbations and include such activities as timber harvest, urban development, dam construction, and agricultural production. The inten-

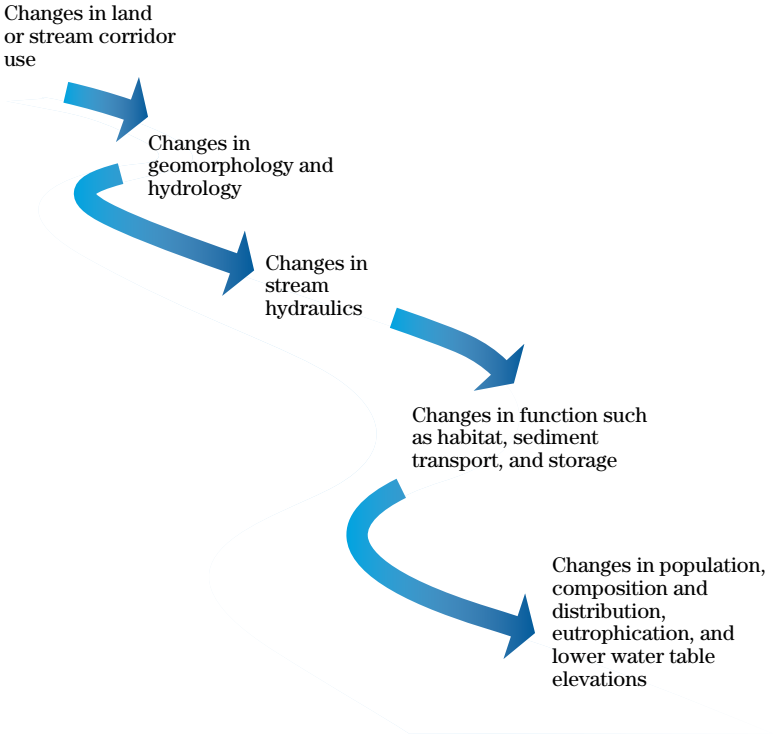
sity, magnitude, duration, recurrence intervals, and interactions of a disturbance or perturbation affect the manner in which fluvial systems respond to them.

(b) Physical responses to disturbances

The U.S. Army Corps of Engineers (USACE) (USACE 1994d; FISRWG 1998) describe disturbances in rivers and the physical responses of channel form and bed composition. Over some timeframe, stream systems tend to seek a condition of equilibrium. However, the behavior of fluvial systems is nonlinear due to time lags in response and the existence of thresholds (Schumm 1977). An illustration of a threshold in a fluvial system is the response of a hypothetical channel to urbanization. Initial deforestation and construction within the watershed produces little change in channel morphology, but when the impervious area of the watershed exceeds a threshold, for example 10 percent, rapid bed and bank erosion occur.

These types of nonlinear behavior often result in a complex response (Schumm 1977), defined as a response to disturbance that is *not* progressive and systematic. Another example of a complex response is provided by the changes in bed elevation that occur downstream from a hydraulic control structure such as a culvert, bridge crossing, weir, or dam. A dam placed on a hypothetical stream reduces sediment supply downstream, leading to bed scour and degradation. In addition, the lower flood stages affect base levels for tributaries, triggering incision and headward progressing bed erosion within the tributary watersheds, contributing sediments to the main channel below the dam. However, since flood flows are reduced by operation of the dam, the main channel is no longer capable of moving larger size particles, leading to long-term main channel bed aggradation. For many small watershed projects, changes in land cover and channelization have triggered instabilities that resulted in incision of upstream tributaries and aggradation along the main stem. The main stem aggradation reduces

Figure 1–7 Disturbance to a stream corridor system typically results in a causal chain of alterations to stream corridor structure and function.



channel capacity and increases the frequency of flooding. Small dams can stop the migration of headcuts and reduce flood peaks along the main stem.

Often macroscale stream corridor features are created or destroyed by the influence of large scale disturbances such as glaciation, earthquakes, tectonic movements, volcanic eruptions, large forest fires, and climate change. These processes and events affect watersheds on a regional or even continental scale. At a smaller scale, floods can alter a stream corridor. Disturbances can gradually or suddenly transform the bed type, planform, or cross section of a stream reach and its flood plain and riparian area. Stream project planners and designers usually have no way to influence natural disturbances or upstream human perturbations on the landscape, but they should be aware of their impact on the stream system of interest. Inadequate consideration of disturbances can rapidly diminish the sustainability and benefits of stream restoration and protection projects. The responses of aquatic species to disturbances depend on the scale of the disturbance, the population structure of the species, and the connectivity of the watershed both before and after the disturbance.

Streambeds within the active stream channel experience the greatest frequency of geomorphic disturbance that may be on the order of every year or two (sediment transporting events). Side channel and backwater areas are not as frequently disturbed, but are affected by higher flow events and channel avulsions (perhaps 5- to 10-year flows). Generally, flood plains have even less frequent disturbances than the main and side channel; it may require a 10-year or larger flood event before a flood plain can be significantly altered. Terraces and hill slopes typically have the lowest frequency disturbance regime when placed in context of stream processes (slope failures and mass movements). Common to all of these disturbances is the episode of disturbance followed by a period of recovery. If the disturbances become so frequent that the system cannot recover before the next disturbance event, then the stream is held in a constant state of disequilibrium or instability (National Oceanic and Atmospheric Association National Marine Fisheries Service (NOAA Fisheries Service) et al. 2006). In these situations, the concept of dominant discharge (channel-forming discharge) is not applicable.

Change in discharge

Long-term changes in discharge magnitude or duration have important implications for channel form and process. Urbanization, deforestation, and fires destroy vegetative cover, increase peak discharges, and lead to channel erosion, while flood control impoundments dampen peak discharges, and smaller, simpler channel forms develop downstream. Sharp increases in peak discharge and resulting decreases in baseflow are often observed in smaller watersheds undergoing development. Conversely, urban stormwater management activities may significantly increase the time that the flow is at bankfull stage, causing an increase in channel erosion. Changes in the discharge may also produce changes in water quality, sediment yield, bed sediment texture, pool habitat availability, and flood plain ecosystems that depend on lateral channel migration processes. Impacts for certain threshold-type channels may be particularly severe if flow forces required for bed movement occur much more frequently under the new discharge regime.

Changing sediment loads

Watershed developments and agricultural practices often generate higher sediment loads. Sediment can be a major concern for water supply reservoirs and navigation channels. Elevated sediment loads may cause real or perceived detrimental impacts on the stream and receiving water ecosystems. Impacts tend to be most severe in coarse-bed threshold systems with low turbidity and normally stable bed conditions. Elevated loads of sand and finer material may blanket gravel or cobble riffles, filling interstitial spaces that are key habitats for invertebrates and gravel-spawning fish. Elevated sediment loads in alluvial systems can result in filled or plugged channels that overflow many times a year and provide little deep water habitat. In other cases, elevated sediment loads have triggered accelerated channel widening or even a shift in channel form from single-thread to braided, with consequent changes in the riparian and aquatic community structure.

Changing water and sediment discharge

When both water and sediment discharge regimes change, fluvial response may be more complex. In the absence of complicating factors, a decrease in bed-material load and water discharge might produce a narrower or shallower channel. If bed-material and water discharge both increase, but water discharge increases more, the alluvial channel will become wider and deeper. For example, in long-term urbanization, the

frequency and magnitude of discharges increase, triggering channel erosion. If sediment supply and water discharge both increase but sediment supply increases more, channels will become wider and shallower.

Changing bed sediment size

Any of the mentioned changes can cause shifts in bed sediment size. Bed sediment size and frequency of movement is a fundamental characteristic of stream habitats that is used to classify or organize stream habitats at the reach scale (Shields and Milhous 1992). However, bed sediment size can change rapidly in disturbed watersheds, in response to changing hydraulic conditions and changes in sediment supply (Doyle and Shields 2000). Formation and destruction of armor layers (layers of coarse sediments on the surface of more heterogeneous deposits) may control the frequency of bed movement and stability. Feedback loops occur in fluvial systems since bedforms, flow resistance, depth, and velocity are governed by bed sediment size.

Changing channel geometry

Erosion that produces channel widening or deepening over a long reach usually signals a change in inputs (increasing discharge) or boundary conditions (lowering a water table leading to death of riparian vegetation and accelerated bank erosion). Changes in channel geometry are also symptomatic of systemic erosion and deposition that accompany channel incision. The most direct result of changes in channel cross-sectional areas is shifts in water depth and velocity at flows that do not overflow the channel banks and the loss of flood plain wetlands and other habitats.

These changes have major implications for aquatic organisms. As larger channels convey higher flows without overflow, more of the erosional forces are focused on the channel bed and banks, rather than dissipated across the flood plain. This can result in loss of productive lands adjacent to the river, loss of riparian vegetation, and discontinuity of stream corridor processes.

(c) Responses of stream corridors to flooding

Physical responses

Unaltered streams usually overflow their banks regularly. Although current thinking among designers is that stream geometry (width, depth, slope) reflects a channel-forming discharge (Copeland et al. 2001), debate continues about the relative influence of rare, extremely large floods. Regional factors such as relief, geology, vegetation, and weather patterns govern the geomorphic significance of large floods relative to smaller ones (Werrity 1997). Clearly, major changes in channels and flood plains occur during high flows. Perhaps less obvious are important ecological functions that occur due to exchanges of water, sediment, nutrients, and organisms between the main channel and the flood plain during floods. The fact that flood plains along large rivers owe their fertility to seasonal floods that deposit thin layers of silt has been recognized for millennia, but the key role that low-velocity regions on flood plains play as refugia and nurseries for aquatic organisms has not. Flooding and associated erosion are often managed or eliminated by water resources projects due to their perceived and real deleterious effects on riparian land uses such as crop production and recreation.

Ecological effects of floods on stream ecosystems

Floods are the most common type of natural disturbance in streams (Resh et al. 1988; Fisher 1990). These high-flow events erode, transport and deposit sediments on flood plains, move large wood, add trees into the channel, flush fine sediments and silts out of streambeds, and transport nutrients and organic matter into streams from the surrounding terrestrial ecosystems (Junk, Bayley, and Sparks 1989; Gregory et al. 1991). The effects of disturbances on stream ecosystems have been reviewed extensively (Ward and Stanford 1983; Niemi et al. 1990; Steinman and McIntire 1990; Wallace 1990; Yount and Niemi 1990; Lake 2000). Aquatic organisms have evolved to not only withstand the potential impacts of floods, but actually benefit from these events (Kimmerer and Allen 1982; Meffe 1984; Matthews 1986; Remillard, Gruendling, and Bogucki 1987; Bayley 1991; Allan and Flecker 1993). For example, trout and salmon deposit their eggs in gravel nests or redds. Silt and fine sediments can smother the eggs and prevent emerging alevins from reaching the stream surface. Floods flush the fine sediments from gravel deposits in streams and create a variety of areas for spawning and clean gravel environments and habitats for rearing fry and juvenile trout and salmon. However, the ecology of trout and salmon is synchronized with these seasonal high flows or floods, so that sensitive life stages (eggs and alevin)

are usually absent or physiologically capable of surviving channel flushing events.

Aquatic organisms differ greatly in their life histories, their vulnerability, and their ability to recover from disturbances (Resh et al. 1988; Yount and Niemi 1990; Lake 2000). The recovery of stream and river ecosystems following disturbances was the focus of a special issue of Environmental Management in 1990. A review of field studies of responses to flooding reveal that, in general, algae and microbes recover in days to weeks, macroinvertebrates recover in less than a year, and fish recover in 1 to 2 years, with a few species requiring decades (Yount and Niemi 1990). The conditions of the ecosystem and riparian corridors are critical factors in determining resistance to the disturbance and the subsequent rate of recovery (Reeves et al. 1995). Refugia from disturbances are important factors in recovery and the design of stream restoration projects (Sedell et al. 1990). Flood plain rivers are larger and more complex than small streams, but the enormous power and frequency of flooding create natural processes for restoring large rivers and their flood plains (Bayley 1991; Sparks et al. 1990).

Disturbances as restoration processes
Disturbance processes, such as floods, fire, and droughts are natural processes of restoration (Gregory et al. 1991; Sedell et al. 1990; Sparks et al. 1990; Reeves et al. 1995). Design of restoration projects or changes in stream management should consider the frequency and location of disturbance events and make certain that their beneficial effects of floods and other disturbances are not negated by the rush to harden streambanks, prevent channel change, and remove habitat features that provide complexity and heterogeneity (large wood, gravel bars, islands, sloughs). In some areas, past projects that were originally designed to minimize the effects of disturbances (levees, riprap, tidal gates) are being removed to restore streams, rivers, and estuaries (CALFED 2003). Restoration projects also should consider natural processes of riparian regeneration (Boyer, Berg, and Gregory 2003). River channels may reoccupy old or abandoned side channels, if revetments and other barriers are removed. Careful design and analysis can achieve a balance between taking advantage of the restorative processes of natural disturbances and the need to protect property and communities from them.

654.0109 Human land uses and their effects on stream corridors

The ecological integrity of stream corridors is intrinsically related to the pattern of streamflow (Poff et al. 1997). The magnitude and timing of water and sediment inputs reflect watershed land use. Their effects on physical habitat and biological communities follow (Wang et al. 1997). Refer to table 1–2 for a list of physical responses of stream corridors to human activities (Gregory and Walling 1973).

(a) Agricultural land use

Typically, both water and sediment runoff increase, and infiltration decreases when forests or grasslands are cultivated or grazed. Irrigation return flows to streams can diminish water quality, but generally do not increase sedimentation and erosion to the extent cultivation and grazing do. Impacts of livestock grazing on stream corridors include destruction of riparian vegetation, soil compaction, bank erosion, water pollution, and degradation of fish habitat and riparian habitat quality. Destruction of vegetation by livestock or by farm equipment may be more damaging adjacent to channels with relatively erosion-resistant beds; if banks are more erodible than the bed, flow energy directed against the banks may produce channel widening and loss of productive land. However, the severity of impacts diminishes when grazing management practices are designed to accommodate seasonal conditions, watershed soils, slopes, climate, and other factors. Similarly, effects of cultivation on stream corridors can be mediated by using conservation practices such as conservation tillage, grassed waterways, and riparian buffers.

(b) Woodland and timber management

Forest management activities affect stream corridors. Regional changes in precipitation runoff relationships have been attributed to development (afforestation or reforestation) or clearing of woodlands. Clearing is usually associated with reduced infiltration and increased runoff and sediment loading. Forestry practices also affect large wood recruitment to streams. Although forests often regenerate rapidly following

Table 1–2 Types of human activities that produce physical changes in stream corridors

Change in stream corridors	Human modifications	Form affected		
Direct changes				
Drainage changes	Irrigation networks		N	
	Drainage schemes		N	
	Agricultural drains		N	
	Ditches		N	
	Road drains		N	
	Stormwater sewers		N	
Channel changes	River regulation	G	P	
	Bank stabilization	G	P	
Water and sediment balance	Abstraction of water	G		
	Return of water	G		
	Waste disposal	G		
Indirect changes				
Land use	Cropland	N	P	G
	Building construction		P	G
	Urbanization	N	P	G
	Afforestation	N	P	G
	Reservoir construction		P	G
Soil character	Drainage			
	Plowing	N		
	Fertilizers	N	P	

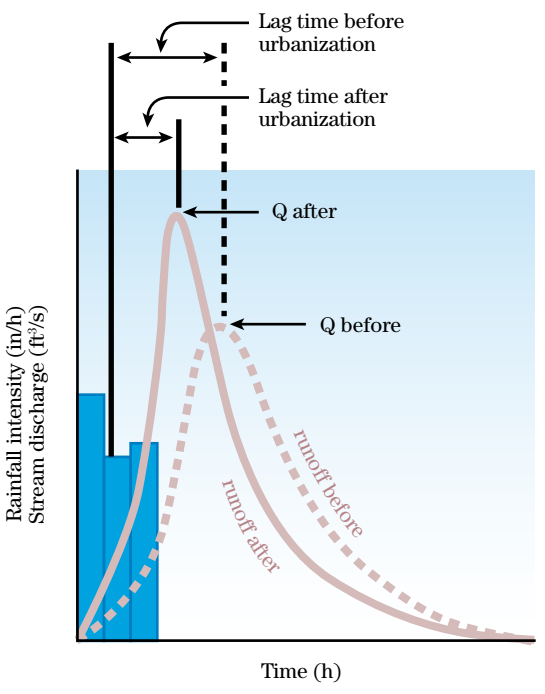
N=modifications of drainage network
G=channel geometry
P=channel planform

harvest (either due to natural succession or replanting), roads and stream crossings may have severe, long-term impacts on stream habitats if not properly designed and maintained. Best management practices such as riparian buffers of minimum widths mitigate the environmental effects of timber harvesting. There are local and regional variations in regulations, and therefore, variable success at protecting stream corridor resources.

(c) Urban development

The primary effects of urbanization are increased surface runoff and reduced baseflows (fig. 1–8). High-flow events of a given magnitude become more frequent (Moscrip and Montgomery 1997). During initial development, sediment yield may increase by an order of magnitude or more, but usually declines as construction projects are completed (Wolman and Schick 1967).

Figure 1–8 Typical effects of urbanization on flow event hydrograph



Over a longer term, urbanization increases the area of impervious surfaces (parking lots, roads, and roofs) which increase runoff and peak flows by eliminating undeveloped land where infiltration can occur. Impacts of urbanization on stream ecosystems occur due to shifts in hydrology that alter stream habitats, such as fine sediment deposition, depletion of large wood, destruction of riparian vegetation, and significant water quality degradation from point and nonpoint pollution (Moscrip and Montgomery 1997). Baseflow in urban streams may be comprised primarily of wastewater discharges and irrigation return flows. Even low levels of urbanization (8% to 12% connected impervious area) impair stream ecosystems (Wang et al. 2001; Wang and Lyons 2003). However, effects may be mitigated by interspersing vegetated plots with impervious zones and maintaining riparian buffers along streams. An extensive review of literature about the effects of urbanization on stream ecosystems is provided by Paul and Meyer (2001).

(d) Mining activities

Mining activities have perhaps greater potential for damaging stream corridor resources than any other human endeavor (Macklin and Lewin 1997). Mines may be constructed above or below ground. Subsurface mines change hydrologic relationships, and in some cases, long reaches or entire streams may be diverted into abandoned underground mines. Drainage from subsurface mines often can be acidic and can contain elevated concentrations of heavy metals. Surface mines are sometimes constructed within channels or on flood plains immediately adjacent to channels, and changes in surface topography and channel volume are great enough to trigger large-scale channel instability or to transform lotic habitats into lentic habitats. Gravel removal from streams may result in changes in streambed type and morphology for long distances and times due to the diversion of coarse bed load from the stream corridor, complicating rehabilitation efforts (Brown 1998), and rendering spawning habitats unusable.

Many stream corridors continue to respond to disturbances created by hydraulic or dredge mining over a century ago. In other cases, watersheds have sustained drastic changes in topography, drainage networks, and vegetative cover due to extremely acidic or infertile soils that have been exposed by mining or disposal of

mining wastes. Dispersal of heavy metals and radionuclides derived from mining or smelting is particularly detrimental (Macklin 1996).

(e) Exotic or invasive plants and animals

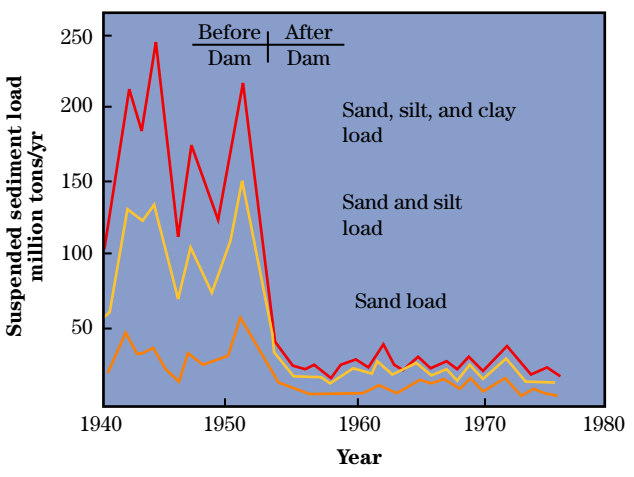
Exotic, or nonnative species, occur in many stream corridors, and management of these organisms is often a necessary component of rehabilitation or restoration projects. Invasive alien species are defined as nonnative organisms that cause, or have the potential to cause, harm to the environment, economy, or human health (Pimentel et al. 2000). Examples of invasive animals are zebra mussels and stocked game fish that supplant native species. These species compete with native species for niche resources, often leading to declining habitat quality and biodiversity. For example, the exotic vine, kudzu (*Pueraria lobata*), was imported from Asia in the nineteenth century and planted along stream corridors in the Southeast for erosion control. In recent decades, kudzu has hindered recovery of native riparian woody plants in stream corridors (Shields, Bowie, and Cooper 1995), and is viewed as a nuisance pest by forest managers. The exotic salt cedar (*Tamarix chinensis*) thrives in dammed rivers and stream corridors of the arid West and Midwest, excluding cottonwood, willow, and many other native riparian species (FISRWG 1998).

(f) Dams and diversions

Dam construction has affected all of the watersheds larger than about 2,000 square kilometers within the continental United States (Graf 1999). Dams typically moderate peak flows and trap sediments (fig. 1–9 (adapted from USACE 1994d)), but additional perturbations also occur depending on the operating conditions and site-specific variables. Grant, Schmidt, and Lewis (2003) reviewed existing information regarding downstream physical effects of dams and proposed a conceptual model based on sediment supply and the change in the frequency of sediment-transporting flows produced by the dam. Dams typically increase water depth and decrease velocity upstream, transforming lotic habitats to lentic conditions. Dams reduce peak flows downstream, resulting in narrower channels with more uniform flood plain vegetation. In some cases, braided channels may be transformed to single-thread forms. Bed material becomes more stable and interstitial voids fill with fines since flows high enough to flush gravels are less frequent. Water quality impacts include major changes in water temperature, turbidity, and nutrient levels. Dams also are a barrier to migration for aquatic species, as well as the flow of energy and materials, leading to fragmentation of habitat and ecological processes critical for sustaining aquatic species (Poff and Hart 2002). Dams can also block coarse sediment transport which, in some cases, results in channel incision downstream. Dams also reduce the delivery of large wood to downstream reaches.

Removing dams is an increasingly common practice, particularly where the dam is no longer needed, costs of maintenance and repair do not warrant continued operation, or environmental values upon removal exceed those provided by the dam. Dam removal projects create technical and political challenges, and the environmental effects may be negative, as well as positive. Of particular concern is the management of sediments stored by the dam, as this sediment may contain contaminants from the watershed. A review of information related to dam removal is provided by the Aspen Institute (2002), and the base of knowledge in this area is rapidly expanding. Other references addressing aspects of dam removal include Schuman (1995); Doyle and Shields (2000); Bednarek (2001); Grant (2001); Pizzuto (2002); and Doyle, Stanley, and Harbor (2003).

Figure 1–9 Effect of storage reservoir on downstream sediment transport (Missouri River average annual suspended sediment load at Omaha, NE)



By their very nature, water development projects such as dams and diversions alter the timing, duration, magnitude, and frequency of streamflow in a river system (Ward and Stanford 1979; Lillehammer, Brittain, and Saltveit 1984; Petts 1984; Gore and Petts 1989; Calow and Petts 1994; Church 1995; Ligon, Dietrich, and Trush 1995; Ward and Stanford 1995a, 1995b; and Stanford et al. 1996 for extensive treatments of this subject). Importantly, dams and diversions can substantially alter fisheries and riparian habitat along regulated stream reaches (Lane 1955a; Williams and Wolman 1984; Ligon, Dietrich, and Trush 1995; Montgomery and Buffington 1998; Buffington and Montgomery 1999; Shields, Knight, and Cooper 2000).

(g) Channel modification projects

Channel modifications are frequently implemented for flood control, drainage, erosion control, or to relocate channels for construction of various types of infrastructure. Changes in channel geometry can trigger significant fluvial response and usually require erosion control structures like weirs or revetments. Many stream management projects address physical or ecological damages produced by channelization projects constructed between 1950 and 1970 (Brookes 1988; Bolton and Shellberg 2001).

The USACE (1994d) ranks changes in the channel cross section by their potential for creating channel stability problems (from lowest to highest) as shown in table 1–3.

Generally speaking, the more dynamic a channel reach is before alteration, the more likely that changes in channel cross section will cause stability problems (USACE 1994d) (tables 1–4 and 1–5).

(h) Recreation

Stream corridors provide recreational opportunities such as swimming, boating, fishing, hiking, hunting, bird watching, and photography. The sensitivity of stream corridors to recreational use varies with soils, climate, topography, and intensity of use (FISRWG 1998). Intense foot or vehicle traffic may compact soils, destroy vegetation, and trigger flow concentration and erosion. Power boating can cause bank erosion due to wave wash, and accidental spills or waste discharges can degrade water quality. Fish and wildlife may be impacted by over harvesting or disturbance. Littering, noise, erosion, and vandalism degrade stream corridor aesthetics.

Table 1–3 Stability rankings for various channel cross-sectional changes

Stability Ranking (1 = low, 10 = high)	Stream cross-sectional change
1	Nonstructural flood control measures (flood-proofing structures, warning systems)
2	Levees set back clear of the meander belt
3	Levees within the meander belt
4	Off-channel detention basins
5	Upstream flood retention basins or reservoirs
6	Flood bypass channels
7	Clearing and snagging (removal of large woody debris or bank vegetation)
8	Enlarged cross section with the existing low-flow channel left intact
9	Channel widening
10	Channel deepening

Table 1–4 Typical features and stability problems associated with streams

Channel type	Typical features	Stability problems
Mountain torrents	Steep slopes Boulders Drops and chutes	Bed scour and degradation Potential for debris flows
Alluvial fans	Multiple channels Coarse deposits	Sudden channel shifts Deposition Degradation
Braided rivers	Interlacing channels Coarse sediments (usually) High bed load	Frequent shifts of main channel Scour and deposition
Arroyos	Infrequent flows Wide flat channels Flash floods High sediment loads	Potential for rapid changes in platform, profile, and cross section
Meandering rivers	Alternating bends Flat slopes Wide flood plains	Bank erosion Meander migration Scour and deposition
Modified streams	Previously channelized Altered base levels	Reduced activity Degradation and aggradation Bank erosion
Regulated rivers	Upstream reservoirs Irrigation diversions	Reduced activity Degradation below dams Lowered base level for tributaries Aggradation at tributary mouths
Deltas	Multiple channels Fine deposits	Channel shifts Deposition and extension
Underfit streams	Sinuous planform Low slope	Meander migration
Cohesive channels	Irregular or unusual planform	Variable

Table 1–5 Rating of channel modifications for effects on channel stability

Type of channel modification	Mountain torrent	Alluvial fan	Braided, multiple channel stream	Arroyo	Meandering stream	Modified stream	Regulated stream	Delta	Underfit stream	Cohesive stream
Nonstructural flood proofing, flood warning, evacuation	0	0	0	0	0	0	0	0	0	0
Levees set beyond stream meander belt	1	2	2	1	1	1	1	2	1	1
Levees set within stream meander belt or along bankline	2	5	5	4	3	3	2	4	2	2
Off-channel flood detention basin	2	3	3	3	2	2	2	2	1	1
Within-channel flood detention basin	4	5	5	5	4	4	3	4	2	2
Major flood storage reservoirs	3	4	4	4	3	3	2	3	1	1
Floodway, diversion, or bypass channels	4	5	5	5	4	4	4	5	3	3
Removal of bank vegetation or large wood (clearing and snagging)	6	6	5	5	7	7	5	5	5	5
Compound channel, low-flow pilot plus flooding berms	5	8	8	7	7	6	6	7	4	4
Significant channel widening	6	9	9	8	8	6	7	7	5	5
Significant channel widening and deepening	7	9	9	9	9	9	8	8	8	7
Significant channel widening, deepening, and straightening	8	10	10	10	10	8	9	9	7	8

Note: Qualitative rating of 1 (low) to 10 (high impact on stability)

654.0110 Summary of ecological principles to guide stream designs

Fluvial systems are dynamic. They change over time and in space in response to their hydrology and geomorphology, and the interactions of these physical processes with biotic communities (bacteria, plants, animals). To protect species, habitats, and water resources, managers must incorporate environmental features into stream project designs (Shields et al. 2003). Historically, engineered solutions to stream channel problems featured constrained physical systems. Today, resource managers and stream design engineers are seeking ways to modify tried and true designs to allow minimally constrained natural ecological processes to be restored. The following principles of stream restoration design incorporate ecological considerations to facilitate such modifications:

- Base designs on ecological principles, as well as physical ones. To the extent possible, restore or maintain the inherent complexities of stream corridors, ecological linkages, and their physical connections. For example:
  - Incorporate native vegetation into design of flood control structures, revetments, levees, and other hard structures.
  - Incorporate silvicultural treatments to maximize generation of trees, specifically for large wood recruitment.
  - Incorporate livestock and/or recreational management regimes into stream design projects to protect restoration or conservation investments in riparian zones and sustain their functions.
  - Remove hard structures no longer deemed necessary or functional in the watershed due to changes in the physical and ecological conditions.
  - Work with partners such as USACE and the U.S. Bureau of Reclamation (USBR) to restore natural hydrologic regimes to the extent possible.
  - Protect life and property.
- During the design process, integrate the disciplines of fluvial geomorphology, geology, hydrology, aquatic and riparian ecology, sedimentation engineering, and hydraulic and geotechnical engineering. If possible, collect baseline and post-implementation data to validate successful designs of innovative approaches to stream corridor restoration. Publish and distribute the information so that it can be used by other designers in the future.
- Design for site-specific response in a watershed-scale context. Consider factors affecting stream corridor processes at different spatial scales, from landscape to watershed to microhabitat, as well as factors that influence the long-term population status and dynamics of aquatic species and the community of species with which they interact. Seek technical advice regarding aquatic species from local experts and state fish and wildlife agencies.
- Consider ecological costs and values, as well as project and long-term maintenance costs of engineered solutions to channel problems. Projects that are compatible with the inherent tendencies of stream corridor systems tend to be more stable, require less maintenance, and are more ecologically productive than traditional engineered approaches (Brookes 1989). These advantages should be emphasized when determining design options.

United States
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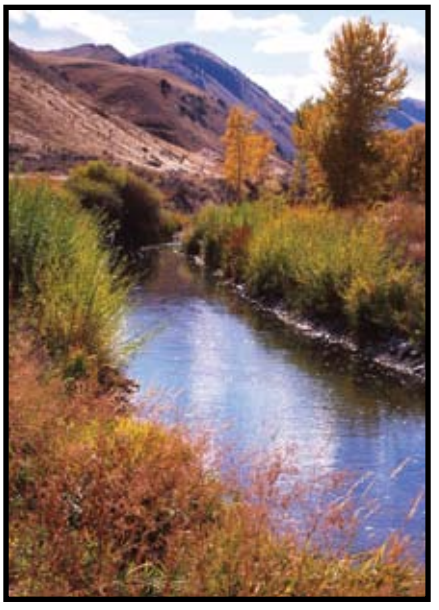
**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 2 Goals, Objectives, and Risk



Issued August 2007

Cover photo: Setting quantifiable, realistic, and achievable goals and objectives is a critical early step in planning successful stream restorations.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 2 Goals, Objectives, and Risk

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Chapter 2 Goals, Objectives, and Risk

654.0200 Purpose

The purpose of this chapter is to emphasize the need for the clear identification of the desired outcome or result of any action to restore or protect streams. Identification of the true nature and causes of stream problems is a critical step in the overall planning process and one which has been abbreviated or overlooked on many failed or poorly performing restorations.

The selection and evaluation of goals, as well as any design approach or treatment alternative must address risk or consequences of failure. This should be examined from both an ecological perspective, as well as a life and property standpoint. While risk is described at several points in this handbook, it is introduced in this chapter. Designing solutions is also an integral part of the overall planning process. The procedure for designing solutions is described in NEH654.04.

654.0201 Introduction

Conservationists are frequently faced with conditions along and in streams that are characterized as problems because certain functions are not being provided or simply because the overall character of the stream system has changed. It may be that the system is damaged and needs to be repaired or that a shift in perception of stream functions and values has occurred, spurring the need for some sort of action.

Understanding the true nature of stream problems is challenging because of the dynamic nature of streams, their seasonal changes, responses to disturbances, and their ability to recover. Recognizing the current condition of a stream, comparing it to historical conditions, and projecting its future conditions are, therefore, challenging; but, nonetheless, need to be documented and clearly understood to determine appropriate and achievable goals and objectives.

The goal of a stream restoration planning process is to formulate a plan that is feasible and effectively addresses the identified problems and goals of the restoration project without adversely affecting adjacent stream reaches or riparian areas.

The term stream restoration can be used to describe many different activities. Actions that support or lead to designed solutions are a critical part of the stream restoration process to assure that what is designed and implemented fits the goals and objectives of the job or project.

(a) Goals and objectives

The perceived success or failure of many stream restoration projects can be as much a function of the criteria selected as the design. Therefore, the importance of establishing achievable project objectives is critical. Once established, these objectives will delineate the data collection effort, methodologies for assessments, and finally the design itself. An interdisciplinary team is required since few people have all the skills necessary to conduct a successful stream restoration study and design. While the exact makeup of the team can vary, it should include engineering, geomorphological, and ecological expertise.

The team should also include the stakeholders. Stakeholders are the groups who may fund the project, affect the stream directly, or be affected by actions taken on the stream. A trained facilitator and interdisciplinary involvement may be needed to guide the development of goals and objectives and to assure that all stakeholders, problem identification issues, other opportunities, and constraints are fully recognized. Once agreement is reached on the alternatives to be pursued, the design process can proceed.

Generalities in objectives, such as *fixing the stream*, can lead to problems. Narrowing the objectives reduces ambiguity for the study team members. Objectives should be:

- specific
- realistic
- achievable
- measurable

Restoring streams to a given historical condition may be an objective. If this is the approach, care must be taken to ensure that physical or biological changes in the watershed have not prohibited a return to that historical condition. For example, the objective for an incised and widening stream in an urban watershed could be to restore it to support a sensitive fish species that was present before development. Changes in water quality and runoff patterns could make this an unattainable objective. Many restoration projects are actually environmental enhancement projects or rehabilitations, since it may not be feasible to return a system to an historical condition. Another of the principal reasons for this is that good, quantitative data on watershed and stream historical conditions is normally lacking. Restoration, therefore, becomes rehabilitation, since not all ecologically self-sustaining functions and values can be restored to the stream.

Clear objectives that are reachable, within the constraints and capabilities of the stream and its riparian area, will lead to better designs that perform as intended. Some objectives may, at first glance, appear to be realistic, but may need to be reformulated if preliminary design information indicates that either the costs will be too high, the intended results may not be achievable, or that boundary constraints may significantly alter or preclude the implementation of the final design.

Typical goals and objectives

Some typical goals for urban stream restoration and recovery are to:

- prevent streambank erosion on residential properties and protect infrastructure
- prevent flooding of residential properties caused by debris or sediment in the channel
- protect bridge abutments, bridges, and road crossings
- protect valuable agricultural land
- protect a municipal water supply (main source works and water quality)
- maintain or restore fish habitat
- maintain or restore water quality

Residential homeowners may be primarily interested in repairing eroded banks and removing debris or woody material blocking the channel to protect their yards, drainage pipes, septic systems, retaining walls, barns, and houses. A municipal water company may need to have a water main protected. Channel erosion may be causing headcutting of the channel, threatening bridge abutments or a road (fig. 2–1). Other stakeholders, including state and Federal agencies, may have primary interests in maintaining or improving aquatic habitat.

Further refinement of stakeholders' interests may produce more goals and better defined objectives such as:

- Maintain or rehabilitate environmental quality by designing and constructing stream restoration projects that:
  - look natural
  - function naturally with channels connected to flood plains
  - provide desirable stream and riparian habitat, including overhanging root cover and large woody material
  - reduce bank erosion
  - maintain water quality
  - are economical to design and build

- Protect infrastructure in channels and flood plains by designing and constructing stream restoration projects that:
  - do not increase flood profiles
  - do not migrate across flood plains
  - protect valuable riparian infrastructure
  - have a low risk of failure
  - do not send debris or woody material downstream to plug bridges and culverts
  - maintain water quality
  - are economical to design and build

In some cases, a compromise needs to be reached between goals for infrastructure protection and aquatic habitat. Sometimes these goals are incompatible, and sometimes they are mutually supportive. Some instances of incompatibilities are:

- An interest in having a project that can naturally evolve over time or rapidly change in response to large flow events, where the stability of riparian infrastructure requires a fixed and static bankline.
- Woody material can provide valuable habitat benefits, but can also increase flood profiles by plugging bridge openings.

Some instances of mutually supportive goals are:

- Large woody material is valuable for aquatic habitat and on some streams can help achieve channel stability.
- Natural streams with channels connected to flood plains can reduce tractive forces in the channel by dispersing and attenuating high velocity flows, thereby increasing channel stability.

Figure 2–1 Township road threatened by severe degradation of channel bed (Calhoun County, IL) (*Photos courtesy of Michael Hollow*)

(a) March 2003—Original concern about bank failure threatening road expanded to include rock riffle grade control structures to stabilize bed, reduce bank height, and improve aquatic habitat

(b) June 2003—2 months after treatment using rock riffle grade control structures to stabilize bed and gabion baskets to stabilize failing bank near road. Note water impounded in pool.

654.0202 NRCS Conservation Planning Process and stream restoration

A plan is a sequence of logical steps to reach a goal or objective. Most stream restoration projects consist of complex issues involving a number of people and ecological components. Using a multi-disciplinary planning team helps to identify and address many of the issues in a timely manner. The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Conservation Planning Process (CPP) follows policy written in the National Engineering Manual (NEM), Part 510, Planning.

The NRCS CPP is referenced because of the need for NRCS field conservationists to recognize how stream work fits into the overall CPP.

Prescribing stream corridor restoration design elements requires progression through and iteration of NRCS CPP steps (fig. 2–2 (USDA NRCS National Planning Procedures Handbook (NPPH), 2003b)). As part of this process, alternative resource management systems (RMS) are developed for the conservation management unit (CMU) or, in this case, the stream reach or stream corridor, and an RMS is selected by the client and then implemented. The nine-step process is listed in detail in table 2–1, with relevance to stream restoration. Although sequential in steps, iterations and cycling back to a previous step commonly occur in the planning process. Plans may result in complex solutions involving a balance of watershed, riparian, and instream actions. The actions may be combinations of management, as well as designed and implemented practices and techniques. The planning process may be rapid for simple projects and may require extensive time for complex projects involving many people and resource issues.

Stream solutions start with landowners or stakeholders requesting assistance with a stream-related problem. The problem may be streambank erosion, which may be controlled and simultaneously protect or enhance ecological functions and values of the stream and riparian area. However, the problem may be a much more serious and widespread condition of multiple reach or systemwide instability, requiring detailed planning and coordination with many landowners and stakeholders. The area of streambank of concern to a landowner is also part of the stream system and its watershed. The focus of the planning team must be on the whole system to determine the cause of the problem, formulate alternatives, and evaluate the effects alternatives may have on the rest of the stream system.

Although these steps are listed in sequential order, the process may require an interactive or sometimes iterative approach. For example, the preliminary design for a planned alternative may not fit the site or may otherwise result in unacceptable construction requirements or unintended or poor overall performance. Recycling back through some steps of the planning process may be required to develop a more suitable alternative for which a new design can be developed.

The formulation and selection of an alternative solution should give consideration to the potential problems and human resource availability. Information must be identified that could affect installation such as construction access, safety concerns, material availability, pollution control requirements, and local ordinances. Some of the potential problems a planner may identify are:

- permitting requirements (surveying, clearing, earth-moving, dredging, cultural resources)
- ownership/land rights
- site access (season, timing, and physical limitations)
- material availability (earth materials, plant materials)
- construction scheduling (season, environmental windows flow conditions)
- local ordinances
- tolerance for risk and uncertainty
- utilities (underground, overhead)
- pollution control (instream, parking areas, sediment control, chemical control)
- safety concerns (working on slopes, in water, around heavy equipment, using hand tools)
- threatened or endangered species

Figure 2–2 NRCS CPP showing the dynamic interaction between the steps

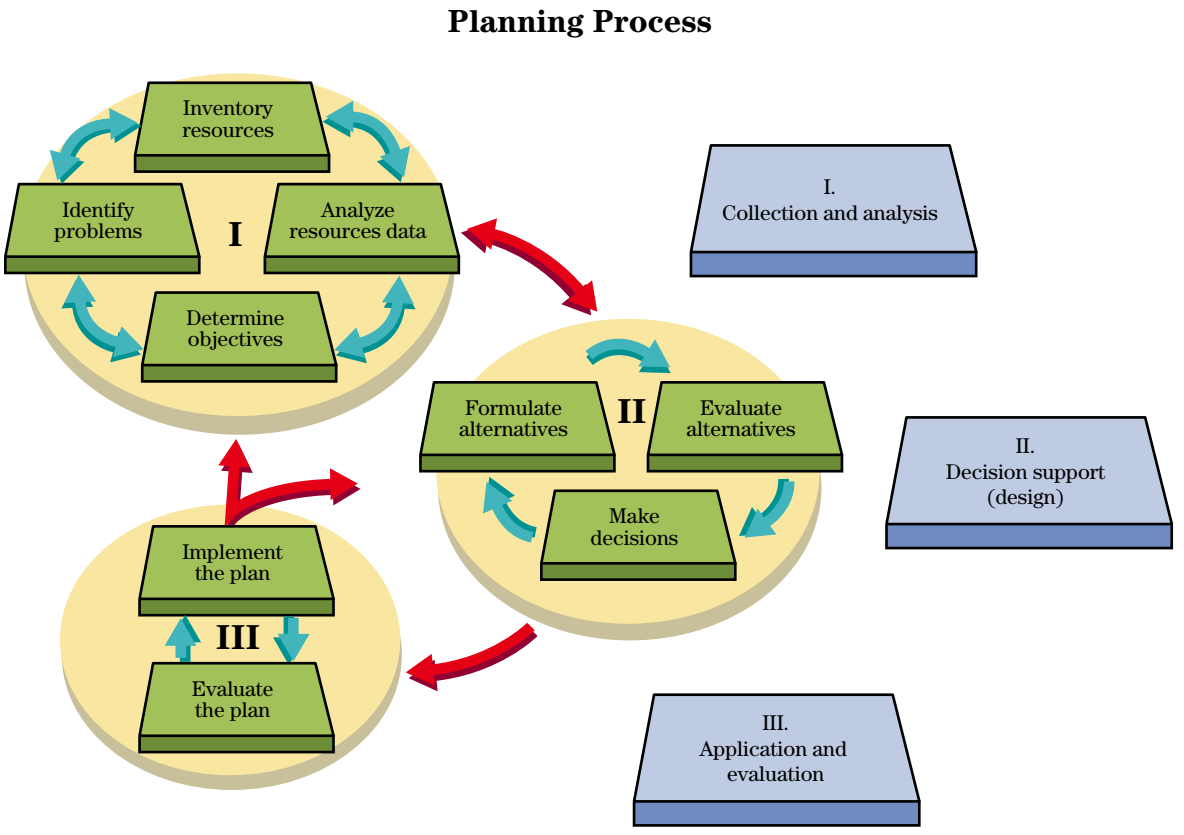


Table 2–1 Stream restoration planning process

Step no.*	Description	Generalized stream restoration planning step	NEH 654 chapter	Detailed stream restoration planning steps	Potential iteration of steps
Phase I—Collection and analysis (understanding the problems and opportunities)					
1	Identify problems and opportunities	Decide what stream characteristics need to be changed	1 2 4 17	Project identification: identify all • Stakeholders • Goals and objectives • Risks • Local vs. systemwide instabilities	Rigor of the assessment depends on the acceptable risk. As each element is addressed, use the information to confirm the initial assessments. It may be necessary to revisit the goals and objectives (may need to revisit step 2).
2	Determine objectives	Describe the desired physical, chemical, and biological changes in the stream			
3	Inventory resources	Study the stream to understand its primary physical processes, dominant impacts on water quality, and abundance and distribution of different biological populations	3 5 6 13 16 17	Assessment: assess the following at the watershed scale and at the site or reach scale: • Geomorphic condition (stream type) • Existing ecological conditions (riparian and instream) • Ecological and physical thresholds • Dominant physical and biological processes and constraints • Sediment budget and stability of existing conditions Acquire hydrologic data (watershed scale) Acquire hydraulic data (stream reach scale) Determine: • Why is the stream in its current condition • What is the ideal condition • What keeps it from naturally adjusting to the ideal condition	
4	Analyze resource data	Examine the collected information and decide what are the most important factors or processes that impact and influence the desired conditions in the stream			
Phase II—Decision support (understanding the solutions)					
5	Formulate alternatives	Determine which processes and factors can be changed, and decide if those changes are sustainable and self-reinforcing	4 5 6 7 8 9 10 11 12 13	Conduct the stability design Select practices or techniques for RMSs Select and design appropriate stabilization techniques • Cross section • Planform • Stabilization, soil bioengineering, integrated techniques • Profile, grade Conduct a sediment budget and stability assessment on the selected design, appropriate to design the practice, so it can be implemented	
6	Evaluate alternatives				
7	Make decisions				
Phase III—Application and evaluation (understanding the results)					
8	Implement the plan	Implement the selected changes to the stream system	15 16 17	Identify construction issues and impacts on design to fine-tune design and implementation Document maintenance and monitoring requirements: • Perform ongoing maintenance • Evaluate success and practice adaptive management	
9	Evaluate the plan	Modify the course of action as new information is collected and analyzed			

*NRCS Planning Procedures Handbook, Amendment No. 4, 180–VI–NPPH, March 2003

During the stream restoration planning process, information is gathered and decisions are made that will direct the design, determine the type of contract or agreement to use, and identify installation concerns. Decisions such as the extent of design needed are determined based on the complexity of the alternative selected, type of contract or agreement, availability of experienced staff to direct construction, and contractor experience.

An understanding of the different types of contracts and agreements is imperative during planning. Contract issues are described in more detail in NEH654.15. Once the planners know the available resources, they can select the type of contract or agreement. Project cost can determine the type of contracting procedure selected such as formal or informal (simplified) acquisition procedures. Funding may also dictate the selection of a particular type of contract. For example, labor may be provided by volunteer groups and the equipment acquired with an equipment rental contract, if funds are limited. A local sponsor may be able to do part or all of the work if they have the equipment, workforce, and experience.

During the planning process, installation must be considered when selecting alternative solutions. For example, complex solutions may require either experienced construction oversight to direct the work or a very detailed design package.

654.0203 Historic approaches for determining goals for stream restoration designs

Knowledge of the behavior of streams in relation to conditions in their watersheds before and during the historical period gives insights to effective watershed management. The design and restoration of streams is often guided by a desire to recover a lost condition. This historic basis requires asking to what standard or for what historical period we are designing. For example:

- What did a stream and its watershed look like at the time of European settlement?
- What did a stream and its watershed look like before the land use became what it is today?
- What did a stream and its watershed look like before the last big storm?
- What did the stream and its watershed look like before its condition became a concern?

The historical approach is not new. Some important earlier studies are by Gilbert (1914); Happ, Rittenhouse, and Dobson (1940); and Vita-Finzi (1969). A more recent, but classic, study using a large assortment of historical techniques for landscape reconstruction is that of Whitney (1995).

(a) Limitations of historical approaches

Goals for a stream restoration project are often determined by picking a point in the past from a photograph, writing, oral history, or from interpretation of landforms and attempt to put the stream back to that condition, or a desired point in time. However, things are not always as they seem. For example, a large Georgia swamp pronounced by authorities as primeval was shown to have been prime agricultural land in the 19th century that had been transformed to swamp by human action (Trimble 1970a). On the other hand, some Australian lakes and rivers commonly thought to have been radically transformed by human action were shown to have changed relatively little, and those changes may have had more natural than human causation (Finlayson and Brizga 1995).

Selecting a stream shape from a photograph and trying to replicate that shape ignores other factors that control the planform and other attributes of the stream and its corridor, including the riparian area. Photos of streams typically focus on crossings, easily accessible points, and cross sections. In many cases, usually little can be learned about the historical pattern and diversity of riparian vegetation from photographs at such locations.

Dynamic changes in timing, frequency and magnitude of flows, and sediment load and transport are also not revealed in photographs. The size, shape, and other physical characteristics of alluvial streams are a function of the types and quantities of sediment in the water and comprising the bed and banks, as well as the nature of the flow conditions. A photograph could easily show a transition phase between two relatively stable states, but may provide little understanding about the direction or magnitude of that change. Refer to NEH654 TS2 for an expanded description on the use of historic information for stream restoration design.

In a physical and possibly biological sense, streams are disturbance-driven systems. The current processes that can be observed in a stream channel were shaped by prior floods, sediment input and transport events, channel changes, vegetation changes, and species interactions. Although it is useful to think of a stream as having a most probable form, each of these extreme events resets or alters that form.

654.0204 Geomorphic approaches for determining goals for stream design

The geomorphic approach to stream restoration work encompasses a number of different activities including stabilizing unstable streambanks and channels, reconfiguring the planform of channelized or aggraded streams, restoring natural substrates and other habitat features, and even daylighting piped streams. Figure 2–3 illustrates a daylighting stream project showing a stream that formerly flowed through a pipe underground and was restored to a more natural condition. This work can be undertaken on a single stream reach or comprehensively over an entire watershed. The geomorphic approach to stream restoration work provides a way to meet management objectives of:

- protecting streamside property or structures from erosion or reducing sedimentation rates in a downstream reservoir or navigable waterway
- improving ecological conditions for aquatic or riparian life

Work undertaken as compensatory mitigation is included in this latter management objective. Regardless of the management objective, stream geomorphic restoration design and construction techniques strive

Figure 2–3 Daylighting stream project

to produce a stable stream that is natural in appearance to the untrained eye, with minimal detrimental environmental impacts.

A structured planning process is needed for stream work that:

- examines the physical, biological, and chemical processes in and around a stream to determine their hierarchy and interaction
- describes in what historic range of variability those processes functioned
- determines which processes could be modified to bring about desired results
- describes desired results and how long it would take to achieve them
- monitors the results of a modification to a stream to determine the level of success
- adapts future actions according to monitoring and evaluation results

Many stream management and modification practices fail because of oversimplification, application of approaches that are not designed for dynamic fluctuations in site conditions, and a general lack of understanding about how streams function, physically, biologically, and chemically. A goal might be that the number of adult salmon returning to a stream will be increased tenfold in the next 20 years. Until the amount of habitat in the stream and its utilization are described, there may be no way of knowing if these fisheries goals can be achieved.

In addition, physical processes of sediment delivery and transport and streamflow fluctuations create physical habitat units. The amount of flooding and interactions between floodwaters, riparian vegetation, and the shallow alluvial aquifer and hyporheic corridor often play a major role in nutrient redistribution in a stream. This can impact primary food sources and productivity. Until these issues are understood in relative importance to one another, determining if the goal is realistic or sustainable may not be possible.

Ideally, environmental investigations should be conducted in the planning stage, prior to formulating a stream restoration plan. Work proposed to control erosion or sedimentation should be substantially different in scope from work proposed to benefit aquatic life. For the former, environmental planning investigations should be focused on collecting information necessary to develop the optimal design that will meet the erosion and sedimentation control objectives. Designs should keep conditions as natural as possible, and construction practices should be used that minimize adverse environmental impacts to stream life during construction. In contrast, when the management objective is to improve ecological conditions for aquatic life, it is important for restoration planners to determine that a stream is biologically impaired and that degraded geomorphic conditions are, indeed, a principal stressor to aquatic life.

(a) Geomorphic analog or reference reach

An analog section of stream, sometimes called a reference reach, can also be used in establishing goals. In this technique, a section of the project stream or a neighboring stream is identified that is thought to function in a desired manner. The reference reach is measured, vegetation is analyzed, and biologic conditions are characterized, and these become the goals for the reach of stream that is deemed to be not functioning properly.

In cases where there have not been substantial changes in sediment supply and hydrologic character, stream reaches up or downstream of the degraded reach could provide an appropriate template for restoration design. This situation is of greatest potential applicability when the cause of channel degradation is from direct channel disturbance or riparian vegetation changes.

More insight is gained by this reference reach approach than the desired point-in-time method, but the technique has some limitations. Directly transferring the properties of one stream to another makes the assumption that the recent disturbance regimes have been similar. Also implicit in this technique is that analog sections are in the same geologic materials and have similar size watersheds, chemical budgets, sediment budgets and sediment particle size distributions, and biologic food chains and predator-prey relationships. The lack of similarity between reference reaches and the restoration stream reach may induce more uncertainty into the process for setting objectives.

Geologic conditions may be controlling stream behavior in the reference reach. These larger scale geologic controls often create stable stream conditions. Unfortunately, this stability is not necessarily transferable to the restoration stream section that is under the influence of different geologic conditions. The limitations of this approach are addressed in more detail in NEH654.09.

654.0205 Ecosystem approaches for determining goals for stream design

Prioritization of stream restoration work should first characterize the existing ecosystem condition, identify stressors, and then prioritize among these stressors. Stream restoration plans should be formulated to focus effort on correcting major stressors. To restore aquatic life, degraded stream conditions should be restored only if these conditions are a priority stressor for aquatic life and will not likely self-correct in a timely manner without intervention.

Several degraded conditions may be harmful to aquatic life. These include constructed fish blockages, upstream migrating headcuts, streams confined in underground pipes, streams confined by concrete, and recently maintained or channelized streams in earthen channels. These stream conditions should generally be considered priority candidates for stream restoration work, since remediation of the condition would likely benefit aquatic life.

The ecologic approach to stream restoration work may provide the greatest benefit to aquatic life in a short reach, but the results could benefit aquatic life over a much greater length of the stream system. When degraded conditions are widespread, the restoration work should be strategically targeted at local reaches that can eventually produce widespread improvement to benefit aquatic life, or work would need to be undertaken on a large scale. Table 2–2 shows likely impact scales for various stream problems.

Two opportunities where localized restoration work benefits aquatic life over a much greater length of stream are where a structure obstructs the upstream passage of aquatic life (fig. 2–4) and when a downstream change in base level causes a rapid upstream migrating headcut (fig. 2–5).

Fish blockages prevent upstream movement of fish and other aquatic organisms that are unable to pass through or over them. Following natural or human-caused events that result in depletion of aquatic species upstream of the blockage, populations occurring downstream may be unable to reoccupy upstream

habitat when conditions improve. Also, following downstream migration, migratory aquatic species may be unable to return upstream of the blockage and cannot survive otherwise suitable habitat. However, it should be noted that fish blockages may be desirable if they are preventing the upstream movement of an unwanted invasive aquatic organism.

Diversion of water flow for irrigation, municipal and industrial water supplies, and recreation can have extreme consequences for aquatic habitat and riparian vegetation along the stream where water is diverted. The degree of impact from these diversions depends on state laws and regulations on instream flow conditions and water rights. In the past, some streams have been totally dewatered due to diversions, resulting in total loss of aquatic habitat. In the past 20 years, many irrigation diversions have installed fish screens with return flows that prevent fish from being diverted into ditches or irrigation fields.

Table 2–2 Situations in which ecologic restoration projects in a stream reach would have a high likelihood of benefiting aquatic life

Stream reach problem	Likely scale of impact
Constructed fish blockage in stream system naturally lacking fish blockages	Watershed
Rapidly upstream migrating headcut	Watershed
Piped stream	Stream reach
Concrete stream channel	Stream reach
Earthen stream channel recently channelized or maintained	Stream reach
Water diversions causing flows too low for fish passage or rearing	Stream reach

Figure 2–4 Fish blockage in stream

Figure 2–5 Upstream migrating headcut; smaller tributaries will also cut into fields, triggering gully erosion

Headcuts proceeding upstream can destabilize streams over a very large area, altering the relationship between the stream and its flood plain, drying out flood plain wetlands, and generating large volumes of sediment that can be harmful to aquatic life. Headcuts are also often fish blockages.

Two degraded geomorphic conditions that present restoration opportunities to improve conditions locally are piped streams and streams with concrete channels (fig. 2–6). When streams are piped or lined with concrete, habitat complexity is completely lost, and flow conditions are often severely altered. Water velocities are greatly increased during high-flow events, while the channels may run nearly dry at other times. Additionally, flow between the stream and ground water underlying the stream (the hyporheic habitat) is prevented, severely restricting the nutrient processing functions that the stream and its aquatic life would otherwise perform. Daylighting piped streams is the restoration of a stream's planform and normally involves substantial design efforts, especially in built-up areas. Removing concrete channel boundaries and restoring a stable planform may be the only way to restore functions to these streams. In either case, a first step is to begin to reconnect riparian areas and people to the streams. In the case of piped streams, the starting point is to gain awareness of what the stream once was and what it can be with daylighting. For concrete-lined channels, reconnecting can start with establishment of green areas and managed riparian areas along the channel.

Channelized streams with earthen channels (fig. 2–7) present unique challenges for restoration. The simplified substrate and depth conditions of the channelized stream constitute a loss in habitat quality for stream life.

Stream channelization is common in regions of the country where large areas of wetlands have been lost (fig. 2–8 (U.S. Fish and Wildlife Service (USFWS))). In these areas, opportunities to restore flood plain wetlands should be investigated as a way to contribute to stream ecosystem restoration. Generally, the self-restoration potential of lost wetlands in absence of intervention is low.

Although excessive sediment in streams is the principal stressor to aquatic life nationwide, restoration projects may not always benefit aquatic life. Excessive sediment, while not desirable, is not typically damaging to all stream aquatic life, as are some other stressors, such as highly degraded water quality and severe

Figure 2–6 Stream encased by concrete channel

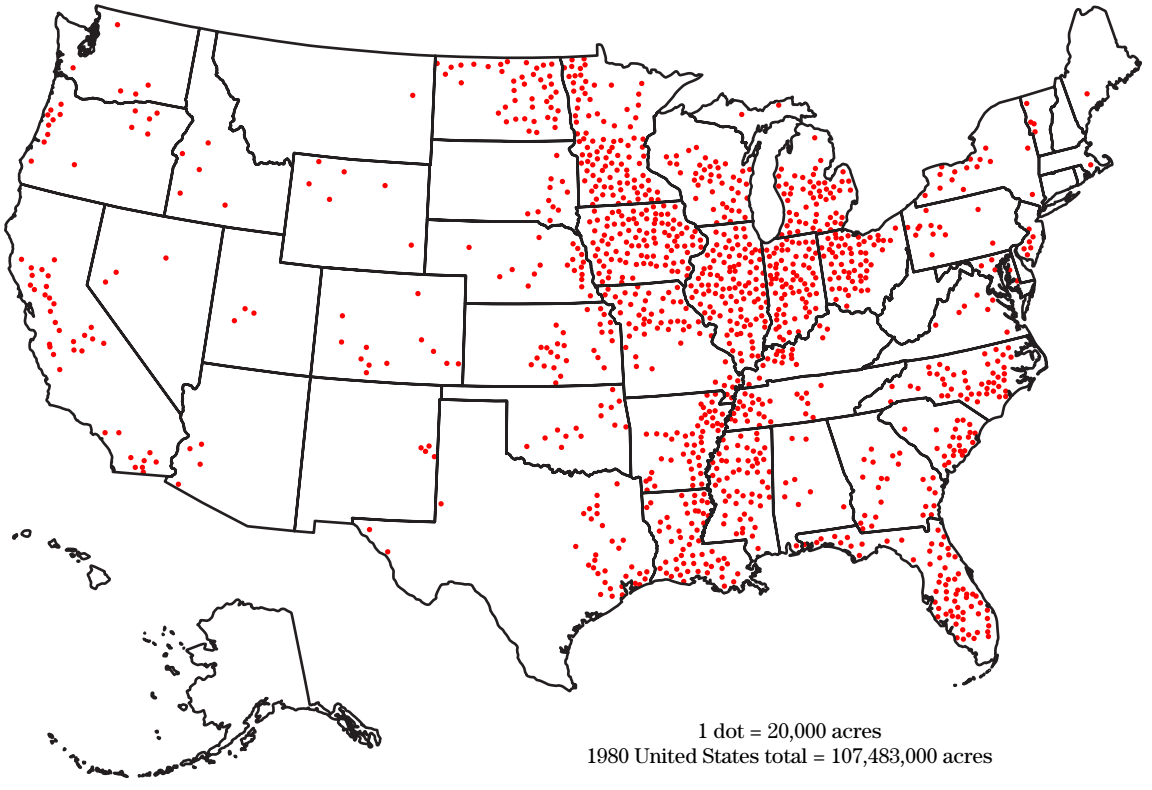
Figure 2–7 Channelized stream (lower left); former natural stream has been assimilated into the regional artificial drainage network

alterations in flows. The impacts of excess erosion and sedimentation impact primarily sediment-intolerant species such as:

- aquatic insect larvae in riffles
- fish that spawn on coarse substrates
- fish that eat insects of coarse substrate bottom habitat
- aquatic organisms that eat submerged aquatic plants

Excessive sediment damages some highly valued aquatic organisms such as many species of trout. Sediment-tolerant organisms, however, may thrive if no other stressors are present. Systemwide strategies may be needed to reduce watershed sediment production. The USDA Agriculture Research Service (ARS), NRCS, and U.S. Army Corps of Engineers (USACE) have undertaken projects to demonstrate such systemwide sedimentation/erosion control strategies in northern Mississippi (Demonstration Erosion Control project).

Figure 2–8 Regions of the country where channelized streams would likely be associated with historic lost wetlands



654.0206 Rural stream restoration

The primary task in most rural situations is to protect an identified resource. Stream restoration in rural areas is often undertaken as a result of an individual landowner request at a specific site where there is no organized effort to restore a larger stream segment. While it may be legitimately questioned whether stream restoration can be accomplished on such a small scale, there are many opportunities to address local conditions and begin the process of education with a long-term goal of restoration on a larger scale. The problems or symptoms leading to the request can be analyzed and documented to determine the feasibility and probable effects of a local solution. The analysis will then conclude whether appropriate action can be taken to offset negative treatment effects and then assess the risk of action or inaction. The time and expense of large-scale studies and data collection may not be justified by a single request from an individual or a small group of individuals. However, in many cases, individual goals and objectives can be achieved by careful problem identification, root cause analysis, and appropriate application of restoration techniques. At the very least, a determination of no feasible action at the individual scale is far superior to an inappropriate attempt at a solution that may have negative impacts at the larger scale.

(a) Issues

Typical rural requests fall into two broad categories: protecting property or restoring and maintaining channel capacity. Both types of requests normally relate to one or more specific problems centered on the loss of tangible property due to bank erosion, excess bed-load deposits, excess woody material, or increased runoff exceeding channel capacity and, therefore, resulting in increased flooding or channel adjustments. The desired condition in these instances is simply to protect what is being damaged: crops, cropland, public roads, utilities, private roads, bridges, and levees. Unfortunately, the problem is seldom as isolated as the landowner's goal of protecting a resource.

The landowner objectives or goals must first be related to an immediate cause and a root cause before a treatment recommendation can be determined. Table 2–3 shows how the most common primary goals relate to problems, immediate causes, root causes, and solutions.

Where possible, it is preferable to address the root cause of the problem. Realistic goals must take into account the accurate assessment of the root cause of the problem. The first task is to broaden the landowner's concept of stream dynamics from merely patching a problem to understanding why the problem exists. Often asking about other current or past stream related problems will lead to a productive discussion about the landowner's longer term goals and objectives. And just as important, it will give the designer insight into the overall stream's behavior and state of equilibrium.

As an example, slope failure affecting an access road may be the problem, but there may also be a problem maintaining a stream crossing or keeping the large logjams out of the channel. Investigation may lead to the conclusion that the channel is degrading, causing the stream crossing to be undermined. The same incision process is then causing excessive slope failure as the bank height increases, resulting in channel widening and large mature trees being undercut and falling into the channel. The landowner may now understand that to patch the slope failure threatening the access road may be futile unless the incision problem is first addressed. The goal of protecting the access road has been broadened to address the cause of the problem. By halting the channel incision on this reach of stream, the landowner's access road can be protected. The stream then can be improved by moving it towards equilibrium, and the aquatic value and aesthetic qualities enhanced.

The task of addressing the immediate problem will remain the landowner's objective, but the method of attaining the goal must address the larger issue of channel instability by treating the root cause of the problem. A decision will then need to be made regarding the scope, risk, and cost analysis of all the proposed treatment alternatives. Before discussing alternatives, explore the secondary goals and objectives of the landowner. The requests are almost always generated by one of the primary objectives, but some landowners will also be interested in such secondary benefits such as aesthetics, aquatic habitat, wildlife habitat, or water quality.

Table 2–3 Common streambank problems, causes, and solutions

Primary goal	Problem	Immediate cause	Root cause	Solution
Protect property: cropland, forestland, residential land Infrastructure: roads, bridges, utilities, levees	Lateral migration	Excess energy/ increased velocity	Steepened gradient or increased flow	Reduce energy gradient by reducing slope with grade control or re-meandering stream. Increases in flow regime will require watershed treatment and/or temporary storage to reduce discharge
		Inadequate riparian vegetation	Clearing and/or removal of mature vegetation	Restore riparian vegetation and buffer area. Additional treatment (toe protection) may be needed during establishment period
		Channel obstruction	Woody material, landslide has reduced channel capacity at site forcing flow around obstruction	Remove obstruction to restore channel capacity
		Unstable channel planform	Normal lateral migration, channelization or modifications have created small radius bend(s)	Modify channel geometry to conform to natural channel geometry relationships of stable channels. Typically with radius of curvature/bankfull width ratio greater than 2.0
		Excessive bed-load deposition	Excessive erosion upstream generating more bed load than channel can transport. May be result of channel incision and widening upstream of problem. May be aggravated by channel widening, resulting in excessive width depth ratios. May also be depositional area created at delta above confluence with larger stream or reservoir	Find and treat sources generating excessive bed load. Channel may then need to have stable cross section and planform reestablished at problem reach. Attempts to modify channel to transport bed load through the problem reach are only successful in moving the problem downstream
	Slope failure	Critical bank height exceeded	Channel incision has created bank height that exceeds soil strength to resist failure	Stabilize bed to prevent additional incision, and raise bed elevation to restore bank heights that are less than critical height. An alternative after halting incision is to slope banks to an angle that is stable for the materials and heights
			Banks are over steepened by lateral erosion at the toe of the bank resulting in slope failure	Stop lateral erosion at the toe. Refer to causes of lateral migration to insure root cause is addressed
		Geotechnical problems	Banks have internal geotechnical problems resulting in bank failure only indirectly effected by streamflow (seeps, springs, weeps, differing soil materials)	Address the geotechnical problem before attempting any other solution. Consult with appropriate technical personnel for assistance

Table 2–3 Common streambank problems, causes, and solutions—Continued

Primary goal	Problem	Immediate cause	Root cause	Solution
Restore or maintain channel capacity	Bed-load accumulation	Excessive upstream sources	Large bank failures/escarpments or bed degradation contributing excessive bed load	Identify and make appropriate treatment to reduce bed-load contributions
		Reduced velocity in reach resulting in deposition of bed-load material	Change in slope or backwater effects from channel obstruction downstream reservoir or confluence with another stream	May be no effective practical solution without detailed project analysis and major project activity to reduce bed load
	Multiple or frequent logjams	Logjams restrict flow, resulting in loss of channel capacity and increased flooding or bank scour near obstruction	Introduction of woody material from logging, clearing ,or high mortality rate of mature trees upstream of problem, resulting in logjams at site	Locate source, and address problem by removing potential for excessive woody material in channel
			Excessive slope failure upstream causing large woody material from riparian zone to enter channel	Address problem of slope failure upstream of problem. Refer to causes of slope failure to ensure root cause is addressed
	Increased runoff/ flooding	Land use changes in watershed such as urbanization or intensified agricultural use	Change in flow regime resulting in increased peaks or extended durations initiating changes in channel morphology	Make watershed modifications to restore natural flow regime. Alternative is to allow channel morphology to adjust naturally, or make carefully planned adjustments to changes in flow regime

Fortunately, effective treatment to address the immediate problem will usually have positive impacts on these secondary goals if the root causes of the problems are addressed and the stream segment is brought back to a state of near equilibrium. However, by first identifying the secondary concerns, the level of improvement can be enhanced with appropriate design, construction, and operation and maintenance of treatment measures.

(b) Scale

After the root cause has been identified, the scale or scope of the solution must be determined. The question is, "Is this a local instability problem or a systemic problem?" If the problem is local, an individual landowner or cooperation between two or more landowners can implement the needed solutions. However, if it is a systemwide failure, rarely can the rural stream restoration project expand to the watershed level without a local organization to sponsor the project. Figures 2–9, 2–10, and 2–11 illustrate a systemwide stream stability problem, and figure 2–12 shows an example of a local stability problem treated with a grade control structure and stream barbs.

The question then becomes, "Is there a solution that can be implemented by the landowner?" If not, the only answers may either be to expand landowner involvement or abandon the project until the required area of treatment can be addressed.

Fortunately, many areas of the country have a grid of roads, culverts, and bridges that effectively confine many of the channel instability problems to segments between road crossings. Many times, even a systemwide failure may have some solutions or treatments available by working complete segments between these manmade stable points. The root cause again will indicate the extent or scale required to implement a satisfactory solution.

654.0207 Developing watersheds

Public officials are faced with ever-increasing liability pertaining to public safety, public infrastructure, property, and other forms of investment. As rural watersheds transform to urban, municipal governments must accommodate growth by annexing and zoning additional land parcels. Preparation for subsequent development of subdivisions and other construction may include an inventory of streams and other sensitive sites to assess the impact of additional runoff from impervious cover. Other planning measures include updating or revising the comprehensive plan, development codes, ordinances, and other protective measures. Rural communities and areas in the urban fringe undergoing transformation may not have technical or human resources to develop comprehensive plans, ordinances, or to carry out special studies. Others, however, play an active role in planning and guiding development.

In these newly urbanizing areas, as well as areas already urbanized, stream restoration can be viewed as a capital improvement because of the amount of public expenditure involved with working in and around streams. Measures are available to municipal and county governments to minimize future impacts on streams, as well as to protect improvements made along the stream. State legislation grants municipal home rule authority, enabling local jurisdictions to enact and codify ordinances. These legal instruments are used to further protect community assets, which include streams.

The U.S. Environmental Protection Service (EPA) Office of Water compiled a collection of municipal ordinances from various local governments throughout the country. These ordinances were collected as part of a larger partnership effort with organizations, such as the International City Municipal Association (*http://www.icma.org*), American Water Works Association, and others, as a template for those charged with making decisions concerning growth and environmental protection. These ordinances also address aquatic buffers, erosion and sediment control, open space development, stormwater control operations and maintenance, illicit discharges, and post construction controls.

Figure 2–9 Systemwide instability. Heavy bed load from upstream erosion exceeds this stream's capacity to carry bed load. The root cause is channelization and urbanization, resulting in loss of channel capacity as midchannel bars form. (Sugar Creek, McLean County, IL)

Figure 2–10 Systemwide instability. Very heavy bed-load deposits have filled original channel, forcing stream to move laterally into finer grained bank materials. This is an example of an alluvial or bed-load-driven stream. (Sexton Creek, Alexander County, IL)

Figure 2–11 Systemwide downcutting induced by channelization project downstream. Additional landowners must become involved to address the root cause of channel incision to stabilize the entire degrading reach. This is an example of a threshold or flow-driven stream. (Hurricane Creek, Jefferson County, IL)

Figure 2–12 Local instability problem above a township bridge. This channel became misaligned with the bridge opening due to lateral migration. The treatment includes stream barbs and a rock riffle grade control structure to protect against possible degradation as a result of shortening the channel during realignment. (Bay Creek, Pike County, IL)

654.0208 Urban stream restoration

The challenges of working to restore physical and biological functions and values in urban or developed streams and their watersheds focus on hydrologic characteristics that no longer fit a natural stream, as well as the obvious limitations provided by physical and legal boundary constraints. To accurately understand the objectives and risks of stream restoration in a developed watershed both the social complexity, as well as the biophysical complexity of the landscape, must be understood (fig. 2–13). Stakeholder goals and objectives must also be clearly defined and the community's interests prioritized. Implementing any successful project also requires that risks be understood mutually by the community, as well as the planners and designers.

Understanding the temporal and spatial scales of stream processes, channel evolution process, and linkages between flow and sediment movement and channel dynamics is essential in any stream restoration project. Understanding these interrelationships will be incomplete, however, without a dynamic watershed context. Recognizing that many developed watersheds are, in fact, actively developing is essential to implementing a successful stream restoration project.

Figure 2–13 Developed area (urban or suburban)

How streams and their watersheds change over time must be clearly understood. It is important to recognize, at the time of observation, where the channel exits in the space-time continuum of its dynamic equilibrium with the water and sediment of its watershed. Failure to do so can result in the implementation of a stream restoration project which is neither in harmony with the land management objectives of the community nor meets the biophysical needs of the resource.

(a) Issues

The issues and interests of landowners within developed watersheds often are similar to those in rural watersheds. These issues and interests often include loss of property, fish and wildlife habitat, recreational opportunities, risk of flooding, and aesthetics. However, this difference in residence time, so to speak, significantly affects all steps in planning a stream restoration project in an urban area.

The human community affects ecological processes and is also affected by the implementation of a stream restoration project. Fully engaging the community in the planning process to identify issues and interests encourages people to look beyond their own backyards and to identify ways to integrate the complex facets of a given project.

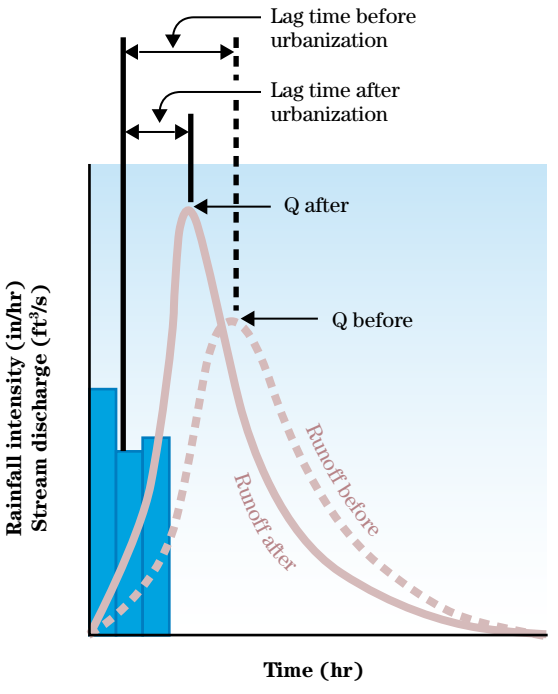
The scale of the project, degree to which the stakeholders wish to participate, and in some cases, the resource issues being evaluated will determine the amount of public participation. An issues and interests meeting has two principal objectives:

- All stakeholders can identify the issues and interests that they feel are important, both as related to the specific project resources and to the area as a whole. These include the natural resources of the area, as well as the social and economic resources of the local community. This allows all members of the community who choose to participate to have a voice in the resource conservation decisionmaking process. By doing so, it creates a way for stakeholders to communicate, explore different perspectives, and see the project in a larger context than might otherwise be possible.
- Stakeholders attending the meeting(s) can participate equally in a collaborative process

to identify the project objectives and focus. The goal is to design and implement a technically sound stream restoration plan that meets the needs of the ecosystem and is in harmony with the resource management objectives of the community and the respective local, state, and Federal agencies. This meeting establishes common threads and common ground for stakeholders and creates a way for their dialogue to be translated into action by implementing an achievable plan to conserve, protect, manage, or rehabilitate the stream corridor resources.

It is of paramount importance to recognize how changes in land use affect watershed hydrology and sediment regime. Urban development produces more impervious surface area, subsurface drains, land grading, and stormwater conveyance systems. The effects of increased imperviousness and the subsequent disconnect of the water infiltration and water storage capacity of the watershed soils and ground water result in a distinct shift of the streamflow hydrograph to the left, as shown in figure 2–14 (Federal Interagency Stream Restoration Working Group (FISRWG) 1998). Both the rising limb and recessional limb of the hydrograph have an increase in slope with a higher peak discharge and a decreased lag time between the onset of a particular storm event and peak streamflows. How this changed and changing hydrology affects the morphology and stability of urban streams and channels must be understood, recognizing that regional curves of typical stream dimensions for various drainage area sizes may not be usable at all.

Figure 2–14 Comparison of hydrographs before and after urbanization



Increased flows in urban watersheds often result in channel incision. In addition, the clear-water discharge associated with present day storm drainage systems results not only in increased streamflows, but also results in streamflows with a higher capacity to transport sediment. The process of incision often results in the simplification of the streambed topography. The pools shorten in length, become shallower, and pool slope is steepened. Riffles become more extensive and steeper.

The process of incision and resulting change in stream morphology operate in a negative feedback loop, perpetuating instability and loss of habitat within the stream. Consider the equation for stream power:

$$\phi = \gamma Q S \qquad \text{(eq. 2–1)}$$

where:

ϕ = stream power (ft-lb/s-ft)
γ = specific weight of water (lb/ft^3)
Q = discharge (ft^3/s)
S = slope (ft/ft)

As shown in figure 2–15, development within a watershed results in an increase in stream Q during a storm event. An increase in Q results in a direct increase in stream power. The increase in stream discharge and, thus, in stream power translates to an increased ability to transport sediment. The channel must adjust (incise) to accommodate the increased flows now generated by its watershed.

Incision tends to decrease bed topography, thereby increasing channel slope. An increase in channel slope results in a direct increase in stream power. Again, the increase in stream power translates to an increased capacity to transport sediment, which is expressed as incision. Figure 2–15 illustrates the relationship between changes within a developed or a developing

watershed, relative to incision and loss of habitat, with respect to the variables of the stream power equation.

An often overlooked and misunderstood risk associated with stream restoration in urbanizing or developed watersheds is the acceptance of the project by the community. It is important for the resource professional, both the planner and designer, to recognize that the community is not only one of the resources affected by the project but also one of the resources which affects the project. A stream restoration project, which is technically sound from a biophysical perspective, but not in harmony with the resource management objectives of the community, may also be considered a failure.

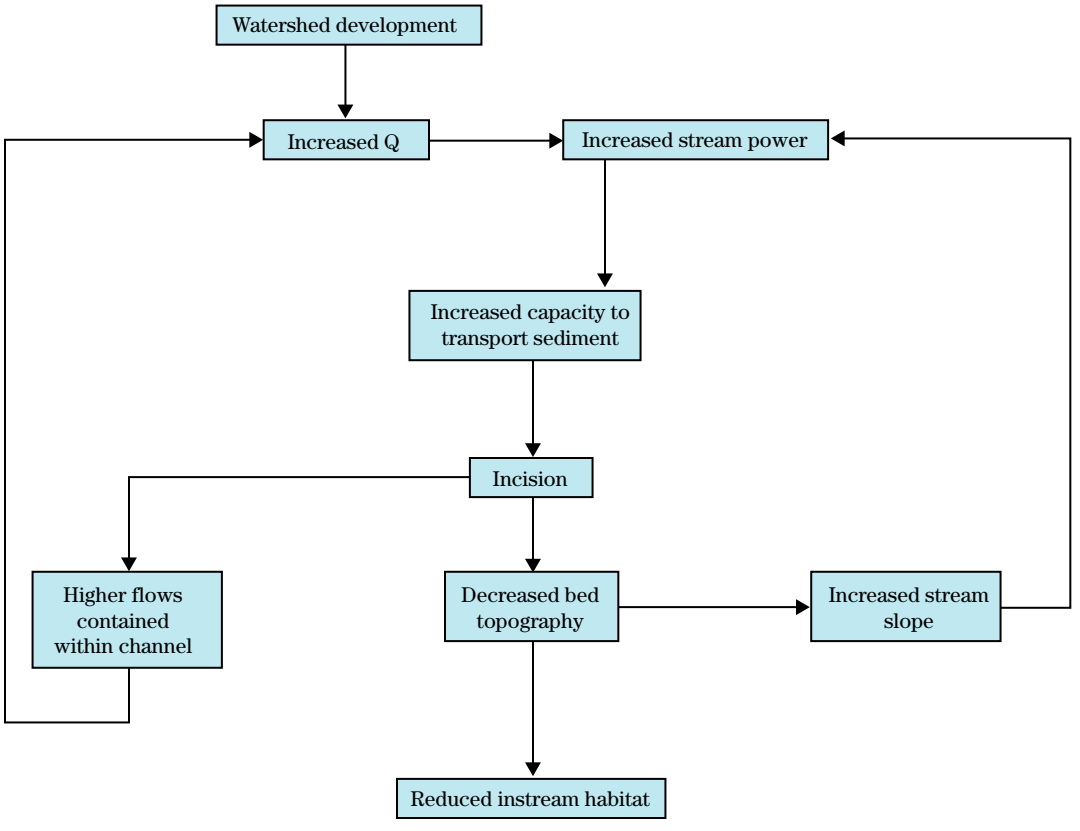
Case study 8 of this handbook, Copper Mine Brook, provides some limited risk analysis for an urban stream restoration project involving concerns about infrastructure, as well as biological and physical stream processes.

(b) Scale

In a rural watershed, the entire stream reach (say, 12 meander wavelengths) may be located on the property of a single landowner who has resided on the property for the past 25 years. The description of the issues and interests of the landowner, relative to the temporal and spatial scales of the channel instability, is comprehensible for the landowner. The landowner has witnessed the evolution of the channel and has a stake in its entire reach.

Conversely, in a developed watershed, that same reach of stream may be home to 30 different property owners who have an average residence time of approximately 5 years. The discussion of issues and interests

Figure 2–15 Potential effects of urban development in a watershed



expands accordingly, and the description of the spatial and temporal scales of the channel process may not be as relevant to these landowners. The perspective of each landowner rarely extends beyond the adjoining properties if it extends beyond their individual property. In addition, their perspective of the channel and its associated processes, on average, do not extend beyond 5 years. They own only a portion of the channel and have been witness to its evolution for only a short period of time.

654.0209 Constraints

Constraints limit the possible actions. Determining project constraints is just as important as establishing objectives. There is a feedback loop between constraints and project goals and objectives. Constraints can be natural anthropogenic. Examples of natural constraints include:

- mountains that limit channel planform
- bedrock outcrops that limit or control channel grade
- water quantity that limits the aquatic species that can use a channel

Examples of anthropogenic constraints include:

- flood plain development or land use that limits channel planform
- tolerance for risk of project failure
- endangered species or regulatory concerns that helps defines acceptable treatment practices

Anthropogenic constraints are particularly common in urban flood plains and include rights-of-way, highways and bridges, utility crossings, buildings, archeological and historical sites, and cemeteries.

Another common concern is contaminated sediment in the streambed or banks. To ensure that these polluted sediments stay in place, it may be necessary to stabilize the banks, preventing the natural channel migration process.

Technical and nontechnical issues affect the feasibility of any stream restoration project. Technical constraints are generally reasons why a particular treatment recommendation cannot function or meet the landowner objective. Nontechnical constraints are generally reasons why the treatment recommendation will not be implemented.

(a) Technical constraints

Data availability—In most rural situations, the existing data are sparse and general in nature. Typically, information is limited to existing aerial photography,

topographic maps, soils maps, and local knowledge. The information from these sources is invaluable, especially historical photography that can be used to determine changes in planform, land use changes, lateral migration, and some bed features such as point bars and central bars.

Additional data collection at these rural sites is usually limited, as the scale of the project will not justify large data collection expenses. If more data are needed than can be collected locally, the technical constraint may then be the lack of sufficient data to make a recommendation or to design a treatment. This constraint must be balanced with the experience and judgment of the designer, as it is unlikely that any project will have all the data the designer would like to have available.

Number of landowners—Another technical constraint enters when the scale of the project requirements exceeds the level of interest. In other words, effective treatment requires work on several properties and there is not the interest or the resources available to implement a solution. The technical decision will then quickly be reduced to answering questions about long- and short-term feasibility and risks. Questions to be asked include:

- Is there a treatment that can be effectively applied within the scope of the project area?
- Would the proposed solution have negative impacts on stream stability on a larger scale?
- Will the effect of upstream or downstream instability threaten the implementation or planned life of the treatment?

If these questions cannot be answered satisfactorily, the treatment is not technically sound and should not be implemented.

Experienced designer(s)—The lack of sufficient data and the lack of justification to devote resources to data collection make experience and professional judgment extremely critical in these rural settings. It becomes essential that the designer investigating these sites has the knowledge, time, and experience to gather basic field information and make sound observations of stream characteristics and behavior both at the site, as well as upstream and downstream, for a considerable distance, before making any treatment recommendation. The investigation must be thorough enough to make sound judgments about the stage of channel evolution in the project reach, sediment transport efficiency, bed-load transport capability, bank materials, presence of geotechnical concerns, planform geometry, geomorphic bankfull dimensions, and incision. Local data are not widespread in the form of reference reach data or localized regional curve information to determine the normal or expected size, shape, and slope of a stable channel in the local physiohydrological region. Therefore, until and unless these resources are developed locally, the designer will need to rely on professional judgment to apply appropriate technical information from other regions and base recommendations on experience gained from similar applications.

Availability of materials, equipment, and labor—For any solution to be implemented, it must be feasible to construct with materials and equipment readily available. Many stream restoration projects are in areas where access is difficult. These types of questions should be asked before finalizing a recommendation:

- Is there access for the necessary equipment to get to the site?
- Is there room for the equipment to operate safely at the site?
- Is the right kind of equipment available locally?
- Will construction be done from the land or bank side or the streamside?
- What kind of environmental damage is likely?
- Will there be damage to roads, lawns, or fences that must be considered?
- Is there access to get materials to the site?
- Are required materials readily available?
- Will access be available for repair or maintenance?
- Are skilled and experienced contractors available?
- Is the labor pool locally restricted during the time of installation?
- Are volunteers available, and can they perform the work?

(b) Nontechnical constraints

Costs—Economic constraints are often the most obvious constraints. In rural areas, the cost may easily exceed the value of the resource to be protected. In many circumstances, protecting rural land may not have a favorable cost/benefit analysis unless other factors, such as improvement to water quality, aesthetics, and habitat enhancements, make the project viable. Landowners may not value these secondary benefits enough to make the project economically attractive. Therefore, a large portion of rural projects often include protection of roads, bridges, utilities, and access points. For this reason, some areas or projects may qualify for financial assistance from Federal, state, or local funding sources to provide landowners an incentive to apply stream restoration practices that would not be economically feasible if the landowner were to bear all costs.

Regulations—Regulatory constraints may also impact the project design and feasibility. All projects are subject to review by regulatory authorities under Section 404 of the Clean Water Act (33 U.S.C. 1344), Section 10 of the Rivers and Harbors Act (33 U.S.C. 403), State Section 401 Water Quality Certification, and Section 106 of the National Historic Preservation Act. Most areas also have state and local regulations that must be met. Become familiar with all the regulatory guidelines in your project area before completing final designs to be submitted to permitting agencies. NEH 654.17 provides additional information and consideration regarding permitting requirements.

Aesthetics—Aesthetic or societal constraints may also affect planning in rural settings, although usually to a lesser degree than in an urban project. By addressing the root cause of the identified problem, the stream segment can be stabilized, and the damage caused by previous erosion or construction activities will be restored through natural regeneration. In settings and locations where natural regeneration is permissible, substantial cost savings can make a project economically viable. In areas with adequate seed supply and fertile soils, sites can naturally revegetate during the first growing season. Figure 2–16 shows a project site on Kickapoo Creek in Illinois, where the banks were revegetated naturally. Some locations will require the restoration of all disturbed or eroded areas with vegetation due to aesthetic, societal, or regulatory requirements.

654.0210 Risk, consequences, and uncertainty

Evaluating risk, consequences, and uncertainty help designers and stakeholders make decisions on what design choices to make. Such measures of probability are described in many texts and handbooks (Fripp, Fripp, and Fripp 2003). Risk is the probability of some event happening. Uncertainty describes the level of error in estimates of risk and consequences. Examples of these are:

- *Risk*—There is a 50 percent chance a 2-year storm will occur each year.
- *Consequences*—If the 2-year storm occurs, the following series of consequences could happen:
  - The streambank could erode 5 feet.
  - Part of a state highway will slide into the river.
  - Motorists could be killed and highway repairs would be expensive.
- *Uncertainty*—Tools to predict the discharge and velocities from various frequency storms are commonly used. Given a certain frequency storm, present tools to evaluate the certainty of the bank eroding with resultant damages are not that accurate or precise.

The analysis of both short- and long-term benefits must consider the risk factor of the proposed treatment alternative. The concept of risk is mentioned here because of its relevance in defining realistic goals for stream restoration.

In rural settings, the risk factor is normally somewhat lower than in an urban setting. If the stream restoration project fails, the consequences are often much greater in a heavily developed area than in an undeveloped area. At the same time, a rural setting can have a high risk factor when infrastructure, such as roads, bridges and buildings, is involved. Generally, the more risk involved in a potential failure, the more caution should be taken in the recommendation and design. This risk assessment should always be considered and discussed with the landowner so that all parties are aware of the level of risk taken. In a low-risk location where only moderate damage may occur, many

Figure 2–16 Project site where banks were vegetated naturally (Kickapoo Creek, IL)

(a) December 2000—lateral bank affecting adjacent cropland

(b) April, 2001—5 months after installation of stream barbs. No shaping or seeding of banks was included in project. Eroding banks will be allowed to vegetate naturally.

(c) September 2001—10 months after installation of stream barbs. Eroding banks have sloughed to stable angle and revegetated.

landowners are willing to accept possible damage that would need some repair, rather than accept substantial cost increases to lower the potential damage. As the riparian corridor matures, a well-designed stream restoration project becomes more stable over time. The greatest risk of damage normally occurs in the period immediately after installation.

More often than not, as a result of increased infrastructure, as well as compromised ecosystem health, the risks of action or inaction tend to be higher in a developed watershed than in a rural watershed. The risks associated with any one particular project vary based on the scope and scale of the subject stream reach and watershed. Although the risks associated with stream restoration are often interrelated, they can be related to the objectives for the social and biological communities.

Different approaches to achieving a given objective may involve varying degrees of risk to public safety, natural resources, property, or infrastructure. They may also offer varying certainties for success. These risks and the probability for success must be weighed against other project considerations when selecting and prioritizing projects. Table 2–4 shows an interpreted range of qualified risks for selected instream treatment techniques.

In any stream project, the "do nothing" alternative should be evaluated. This is also referred to as the "future without action" alternative. However, even this apparently simple approach should not be considered casually. Allowing an unstable condition to continue can have significant detrimental consequences from both a physical, as well as an ecological perspective.

Table 2–4 Potential range of qualified risks for selected instream treatment techniques*

Technique	Risk to habitat	Risk of channel change	Risk to infrastructure, property, or public safety	Uncertainty of technique	Probability of success
Boulder clusters	Low	Low to moderate	Low	High	Moderate
Channel modification	High	High	Low to high	High	Low to high
Drop structures	Low to moderate	Moderate	Low to high	Low	Moderate to high
Fish passage restoration	Low to high	Low	Low to moderate	Low	High
Instream sediment detention basins	Moderate to high	Low to moderate	Moderate to high	High	Low to high
Large wood and logjams	Low	Moderate	Moderate to high	Moderate to high	Moderate
Side channel/off-channel habitat restoration	Low	Low to moderate	Low	Low	High

* Derived from Stream Habitat Restoration Guidelines, September 2004; Washington Department of Fish and Wildlife, U.S. Fish and Wildlife Service, and Washington Department of Ecology: *http://wdfw.wa.gov/hab/ahg/shrg/*

654.0211 Conclusion

The accurate identification and prioritization of the issues and interests of the land user or community is crucial in planning and designing a stream restoration project. Objectives or goals that are preconceived or defined unilaterally for a restoration project often result in failed projects or projects that do not perform properly or meet expectations. Detailed designs, based on poorly formulated goals and objectives, will not normally meet expectations of the restoration. Time and resources should only be expended on detailed designs if the objectives are specific, realistic, achievable, and measurable.

Objectives of a restoration should be as specific as possible, with the resulting conditions clearly described in terms that stakeholders understand. Improving the environment would be a poorly stated objective, without any other description of what will be different with the project in place.

Objectives should be realistic and achievable. Early optimism during project planning should be tempered by what can actually be done. For example, restoration of a cold-water fishery in a stream that has been severely altered by urbanization and watershed changes may not be achievable, even though it is a noble goal. The temperature regime of the stream, both before and after restoration, should be thoroughly understood. Another example might be the desire to restore a stream to an historical condition, but the current watershed conditions differ significantly. It may not be possible to restore all of those historical functions and values to the system, but a few could actually be restored.

Objectives should be measurable. Subjective goals, such as improve water quality, may seem to be good, but should be further refined to state exactly what changes in water quality parameters are the desired outcomes of the restoration. Monitoring of the before and after conditions will reveal exactly how much change has been achieved or to what degree the desired functions and values have been restored to the stream.

The selection of goals and objectives must take into consideration the risk associated with the current, as well as the proposed project condition. This risk must be evaluated from both an ecologic, as well as a life and property prospective. In addition to the risk of the project, the uncertainty associated with the design approach and the probability of success should be taken into account. The evaluation of risk and uncertainty may force a revision of the goals and objectives.

The restoration design should include a balanced approach between structural and management elements. For example, stabilizing streambanks should include not only bank stabilization practices, but also riparian practices to manage cattle crossing (fencing), access to water (designed stream crossing), and grazing management. The final plan and design for the restoration should consider ways to meet the goals and objectives of the stakeholder(s), as well as to benefit or improve water quality, fish habitat, and riparian habitat.

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**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 3 Site Assessment and Investigation

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Cover photo: Characterizing the physical, chemical, and biological conditions of a stream involves sampling and the use of assessment tools.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 3 Site Assessment and Investigation

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Chapter 3 Site Assessment and Investigation

654.0300 Purpose

This chapter describes procedures for assessing watershed and site conditions. Stream system inventory and assessment techniques are identified and compared. Information is provided on stream stability, as well as geological and biological assessments. A description of the uses, advantages, and disadvantages of various geomorphic stream classification systems is also provided. Finally, this chapter addresses fluvial processes and geologic issues related to ecological function, as well as stream design.

The description in this chapter of assessment requirements and methods focuses on stream systems. A stream system consists of a watershed and ground water component that contributes discharge to the system and a flood plain area that is directly connected to a fluvial channel. In a natural setting, a channel is sized by nature and associated with discharge and sediment loading from upland areas, as well as earth materials in the channel. Other upland influences include anthropogenic changes in rainfall runoff characteristics such as occur with land use change and change in sediment supply. Sediment changes can be associated with land use change and, also, with dam construction. In addition, the system might be influenced by downstream factors such as a bridge, dam, or the confluence of another stream or river.

654.0301 Introduction

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) is increasingly providing technical guidance to organizations and individuals who are actively restoring rivers and streams degraded by extreme storm events, as well as human activities. Stream restoration is an interdisciplinary, comprehensive effort that focuses on reversing past damages and assisting nature to restore partial or complete functioning of a stream system.

Watershed hydrologic, hydraulic, geomorphic, and biological processes affect streams, and designers should have an understanding of these basic principles to work in streams. NEH 653 (Stream Corridor Restoration: Principles, Processes, and Practices) provides fundamental information on streams and their corridors, as well as the basics of how to plan stream corridor restorations (Federal Interagency Stream Restoration Working Group (FISRWG) 1998).

A stream inventory and assessment is needed to provide the process-based framework to define past and present watershed dynamics, develop integrated solutions, and assess the consequences and success of restoration activities. This assessment generally includes data collection, field investigations, and a determination of the equilibrium stage of the system or portions of the system. A channel is considered in dynamic equilibrium when the prevailing flow and sediment regimes do not lead to aggradation or degradation or to changes in the channel cross-sectional geometry over the medium to long term. Data collection and assessment forms the foundation for analysis and design and is an essential first step in the design process, whether planning the treatment of a single reach or attempting to develop a comprehensive plan for an entire watershed (FISRWG 1998). Refer also to the National Water Quality Monitoring Handbook, NEH 600 (part 1) and NEH 651 (part 2, draft).

A multidisciplinary investigation is typically performed to assess prior, existing, and future stream system conditions; to better understand the dominant processes acting in a watershed; to identify information and resource needs; and to aid in the selection and design of project alternatives. Key factors that should always be considered are spatial and temporal influences on the

system or portion of the system of interest. Numerous methods are available to investigate and assess stream systems. None of these methods are perfect and vary considerably in the information they provide, information they require, the spatial and temporal scales they consider, and the complexity, expertise, and resources required to use each method. Many of these methods, together with factors associated with their use, are briefly described in this chapter. A compilation of numerous inventory and assessment techniques (USDA NRCS 2001c) is presented in NEH654 TS3A, along with a table that describes the principal features and applicability of each method.

(a) Stream system assessment

Planning for stream system projects includes a systematic investigation of past, existing, and future conditions in the system. A complete analysis requires a team experienced in stream geomorphology, geology, hydrology, ecology, and stream hydraulics. The purpose of this investigation is to:

- identify the dominant fluvial processes in the stream system
- identify the equilibrium state of the system or portion of the system of interest
- determine if there is a problem. If so, is it an anthropogenic problem, a problem associated with the equilibrium state of the system, an existing or potential problem associated with past, current, or future land use, flood plain or riparian zone changes, or a combination of factors
- identify the factors that influence the issues of concern, as well as potential mitigation strategies

Knowledge of dominant processes allows prediction of the proposed project's impact on stream geomorphology, potential changes in the equilibrium of the system, and the impact the natural processes will have on the functionality of the project. The equilibrium state of various stream reaches and the changes occurring in the stream system should be accurately assessed. This assessment is the foundation for understanding future changes in the system and how alternative management, design, or mitigation strategies will work. Solutions are developed to address the goals and objectives of the project. These solutions might be self-sustaining, or require periodic maintenance, or the solutions are meant only to be temporary. In some cases, the best solution might be a river rules concept that simply provides adequate space for the stream to adjust to change.

Many perceived or actual stream problems are associated with a change in sediment supply within the system; change in sediment transport capacity or competency; change in bank erodibility, usually resulting from vegetation removal; or a combination of these factors. Potential causes of these changes are many. They might be due to localized stream modification such as a new culvert or bridge crossing, flood plain modification, or a more systemwide change. They might be due to urbanization, increased impermeable surface area, altered drainage, increased runoff, more discharge, larger peaks, and more frequent high flows. Biological and ecological impacts are sometimes associated with other factors such as changes in water chemistry, changes in low flow regimes, or changes in vegetation on the banks, flood plain and riparian zones. Assessment of these factors is presented later in this chapter.

Bank and meander migration, scour, and deposition are natural stream processes that might be exhibited by high quality streams that are in dynamic equilibrium. Natural meander migration rates vary across hydrophysiographic areas, so that a particular rate may or may not constitute a problem. Major events or significant perturbations may cause a stream to make rapid adjustments to move toward or depart from a state of equilibrium. In some areas, very small rates, perhaps a fraction of a foot per year, might signal a problem, while in other areas many feet of movement in a single event might be normal.

Often, any adjustment is viewed as a problem because it causes an unwanted impact on anthropogenic land use or structures. In these cases, the bank is often hardened. This treatment creates a temporary solution for the human concern, but, in some situations, actually makes the stream more prone to moving out of equilibrium because an additional constraint has been added to the system. Therefore, it is important to recognize that short-term changes in sediment storage, channel shape, and planform are both inevitable and acceptable in natural channels with unprotected banks. A key to preventing problems or to develop-

ing self-sustaining solutions is to provide the channel system with adequate space and time for adjustment.

A range of conditions exists for a stable channel, and some stable processes may appear unstable. Specifically, many large river systems have a stable state characterized by low gradient alluvial channels with active channel migration zones. Mistakes have been made in the past due to the lack of recognition of this key process (Wohl 2000; Reid and Dunne 1996).

Stream evaluations can be performed at various levels. The appropriate level of detail depends on the status of the study, the perceived significance of potential problems, the scale of the project, risks, and the resources available. A unique approach of using aerial videography and Geographic Information System (GIS) technology to assess stream stability is described in NEH654 TS3B.

Basic information requirements

Comprehensive evaluations of stream systems can require both extensive resources and extensive expertise across a wide range of disciplines. It is important to have adequate expertise and to identify and address the most important issues. For example, it is not uncommon for assessments to focus on hydrology and hydraulics. While both might be vitally important in developing an appropriate solution, the most critical basic information is first-hand knowledge of the stream system and an assessment of the past, current, and future equilibrium state of the stream system. This often requires an assessment of sediment supply and transport.

(b) Initial stream characterization: flow duration

U.S. Geological Survey (USGS) topographic contour maps may be a first source of information on some important flow characteristics of streams, but the blueline streams may lack the detail to decide which streams need protection (Leopold, Wolman, and Miller 1964; Hansen 2001). Delineating stream networks using the contour crenulations (indentations) with some field verification can improve the identification and location of streams on maps, resulting in better awareness and management of small streams (Strahler 1957).

Perennial and intermittent stream types flow for extended periods beyond storm events. Under normal circumstances, perennial streams typically flow all year. Intermittent streams cease flow during parts of the year. Ephemeral channels primarily flow in response to storm events, but normally do not flow for extended periods afterward. Physical and biological indicators of flow duration and channel response to flow are also useful to help characterize a stream when flow data are not available.

Streams may be classified according to their flow conditions (table 3–1 (Hansen 2001)). The presence of a defined channel may be the best indicator to separate perennial and intermittent streams from ephemeral channels.

Small streams are seldom identified on contour maps beyond indentations in the contours, so they may go unnoticed if field evaluations do not follow office plan-

Table 3–1 Field criteria used for characterizing streamflow conditions

Criteria	Streams classified by flow duration characteristics		
	Perennial	**Intermittent**	**Ephemeral**
Channel	Defined	Defined	Not defined
Flow duration (est.)	Almost always	Extended, but interrupted	Stormflow only
Bed water level	Above channel	Near channel surface	Below channel
Aquatic insects	Present	Few, if any	None
Material movement	Present	Present, less obvious	Lacking or limited
Channel materials	Scoured, flow sorted	Scoured or flow sorted	Mostly soil materials
Organic material	No organic buildup	Lacks organic buildup	Organic buildup

ning. Stream and water quality protection goals may be difficult to achieve if the watershed streams and their connection and impact on the project area are not well defined.

For larger areas, stream detail can be digitized from topographic maps. Flow networks can also be estimated from digital elevation models (DEM) or triangulated integrated networks (TIN), which can be developed from digital topographic contours with flow routing methods using GIS software. If detailed digital elevation data are available, 10-square-mile data are preferred over 30-square-mile data, and noninteger elevations are preferred. Light Detection and Ranging (LIDAR) imagery, if available, may also be used to identify stream systems in great detail. Substantial editing of computer-generated stream networks may be needed to verify streams according to the contour crenulations.

Unusual flow patterns and paths can complicate stream type identification. Perennial streams may be interrupted as surface flow travels underground in coarse substrates, crevices, or through debris deposited in landslides. In karst topography, perennial streams may appear from underground flow networks, and substantial surface runoff may enter ground water directly through sinkholes and other solutioned features in karst limestone.

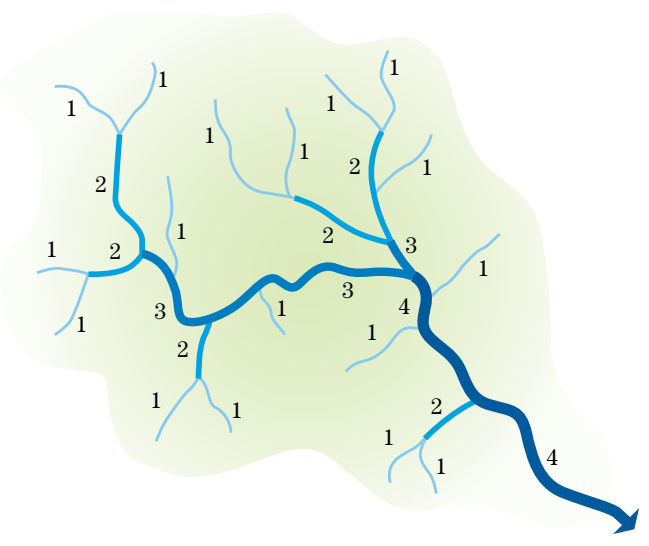
Soil types and plant species are not listed specifically for determining stream types, but the presence of hydric soils, hydrophytic plant species, or associated hydrologic indicators may be important in determining stream types (U.S. Army Corps of Engineers (USACE) 1987). Hydric soil indicators adjacent to streams include gleyed color and mottling and can be used to help estimate depth to the permanent water table or saturation zone. Plant species and rooting adaptations common to high moisture conditions may provide additional information.

(c) Initial stream characterization: stream orders

Identification of the stream network and stream orders (Strahler 1957) can be done through analysis of 1:24,000 scale topographic quadrangle maps (Leopold, Wolman, and Miller 1964), digital orthophotoquads (DOQ), or DEM. First-order streams are identified as the unbranched channels that drain from headwater areas and develop in the uppermost topographic depressions, where two or more contour crenulations (notches or indentations) align and point upslope. These first-order streams may, in fact, be field ditches, gullies, or ephemeral gullies. The combination of two streams of the same order forms the next higher order (fig. 3–1). The density and pattern of the streams may vary with drainage size, geology, landform, and type of stream channel.

The use of stream orders is a valuable quick reference and has been used to correlate information. However, it does have limitations. Since the intersection of a channel with a lower order does not raise the order of a stream, a long, skinny basin may be classified with much lower order streams than a wider, but shorter drainage basin. As a result, stream order comparisons work best when the comparison is within a single drainage basin.

Figure 3–1 Stream orders in a watershed may be useful in understanding flow conditions, setting goals and objectives, and provide preliminary information for design of solutions.



654.0302 Preliminary investigation

Ward and Trimble (2004) recommend that a preliminary investigation be conducted to provide sufficient information to design the study; select analytical methods, models, and procedures; and prepare an estimate of fund requirements to conduct the assessment. The purpose of the study is to:

- assemble and evaluate existing data
- obtain as much information as necessary
- develop a scope of work
- identify data requirements, data deficiencies, and cost resource requirements
- identify system boundaries and boundary conditions
- prepare a preliminary diagram of the physical system
- identify issues that restrict or inhibit the ability to conduct the study or need further study
- use USGS gage data or other records, where available, to assist in analysis of high flows and channel-forming events
- identify biological and ecological assets and concerns

654.0303 Reconnaissance

A reconnaissance of the site should always be done. Careful planning and advance preparation is critical. Practitioners should obtain background information by reading reports or previous studies. It is also valuable to obtain available topographic maps and aerial photographs and collect climate, soils, geology, and land use information. Especially useful will be maps, photographs, and surveys from different years in the same location, to indicate changes in the watershed and stream. It is important to talk to people familiar with the location and communicate with local, state, and Federal agencies to determine if there are ongoing or recently completed studies in the region.

Much of the data can be assembled in the office by reviewing old reports, maps, and aerial photos. Historical data are used to identify trends, provide information on rates of landform change in the watershed, and help determine land use impacts on current conditions. The examination and review of geologic information, local historical accounts, historic channel thalweg and cross section information, FEMA maps, biological monitoring, hydrologic models, watershed development and land use patterns, and aerial photographs can be useful in this assessment. Recent gage data should be reviewed to determine if current conditions might be the result of a recent extreme event, rather than long-term or systematic instabilities.

Prior to a site visit, it is recommended that the field team prepare a checklist of needed equipment and materials. The team should prepare written descriptions of each task to be performed, and make sure each reconnaissance team member is aware of the objectives, as well as their assigned tasks. It is useful to consider things that might go wrong and prepare contingency plans before going to the field. This is particularly important if electronic equipment is being used to document findings or take measurements.

For safety and logistical reasons, field work is best accomplished by teams of at least two people. Field work, particularly in urban areas, may raise significant health and safety issues, including crime, needles, and exposure to raw sewage and waterborne pathogens such as hepatitis.

654.0304 Detailed field investigation

Following the preliminary investigation, a detailed field investigation is performed to describe the geomorphological landforms of study reaches and to identify potentially destabilizing factors. This effort is often coupled with an identification of potential treatments or projects. This is based on field-gathered evidence of erosion, sediment storage, and deposition in the individual reaches. It is critical that experienced personnel conduct this effort. It is recommended that as a minimum the team consist of a biologist who is familiar with characteristics of aquatic and riparian habitat of the study area; a scientist or engineer who is experienced in stream geomorphology and sediment transport; and engineer(s) experienced in hydraulics, hydrology, design, and construction practices.

Inspections at bridge crossings should be treated with caution, since bridges are frequently placed at constrictions or at bedrock outcrops. These locations may not be characteristic of the stream as a whole. However, valuable indicators of stream stability can be observed at bridges and other points where infrastructure crosses the stream. Field assessments are best made during low-water conditions and during the dormant season when banks are not covered with vegetation and can be more readily examined. However, it is important to recognize that conditions may be different at high flows. In assessing streams in the field, it is important to keep in mind that a channel typically has four degrees of freedom: width, depth, slope, and planform.

Basic information on how to conduct field investigations to collect data for a channel stability assessment is contained in the following publications: EM 1110–2–4000 (USACE 1995c); EM 1110–2–1418 (USACE 1994d); and Thorne (1998). Biedenharn, Elliott, and Watson (1997) contains a detailed description of field equipment and features to look for in the field. The collection of field data can be aided with the use of field assessment data sheets, which should be adapted to the specific study needs. Guidance for carrying out detailed reconnaissance surveys is given in Downs and Thorne (1996); Thorne, Simon, and Allen (1996); and Thorne (1998). Example field assessment data sheets are provided in appendices B and C of Copeland et al. (2001).

Generally, the following basic information should be collected:

- descriptions of the watershed development and land use, flood plain characteristics, channel planform, and stream gradient
- assessment of historical conditions—this can be obtained with interviews of knowledgeable landowners. Anecdotal testimony, however, may result in some exaggeration of historical conditions, but multiple sources will help to provide accuracy
- measurements of low-flow and bankfull channel dimensions and channel slope in critical reaches and identification of terraces and active flood plains
- characterization of the channel bed—determine if it is bedrock, erodible cohesive material, armored, or unconsolidated alluvium. Determine the gradation of any armor layer and collect bed material samples of the substrate.
- descriptions of river bank profiles, bank materials, and evidence of bank instability
- descriptions and locations of point bars, pools, riffles, bed instability and evidence of sedimentation processes
- observations of response to channel alterations, and evidence of stream recovery
- descriptions of channel debris, woody material, and bed and bank vegetation
- preliminary stream restoration alternatives should be identified so information can be gathered on possible constraints such as access, utilities, and staging areas.
- photographic records of critical stream and watershed characteristics

There are many possible indicators of the equilibrium state of a stream system. A range of field indicators within a watershed is shown in table 3–2, reproduced from Copeland et al. (2001). These indicators are not absolutes, and items listed as possible indicators of instability may occur in natural or stable streams. Usually, no single indicator will accurately identify the cause

of a problem or the equilibrium state of the system. A weight of evidence approach should be used, and it is important that those conducting the field assessment be experienced in the accurate interpretation of stream reconnaissance results.

It is also important to recognize the possible pitfalls of field assessments. These include observer bias, temporal limitations, and spatial limitations. Issues related to observer bias can be partially overcome with the consistent use of trained personnel. This practice will minimize relative differences between observations. Temporal bias can be minimized by examination of historical records, but these may be incomplete. Having the field team walk a continuous reach of stream can reduce spatial bias. Field investigation should extend both upstream and downstream of the project reach and, ideally, should be conducted during different seasons.

Table 3–2 Possible field indicators of river stability/instability

Evidence of degradation	Terraces (abandoned flood plains)
	Perched channels or tributaries
	Headcuts and nickpoints
	Exposed pipe crossings
	Suspended culvert outfalls and ditches
	Undercut bridge piers
	Exposed or “air” tree roots
	Leaning trees (hockey stick trunks)
	Narrow/deep channel
	Banks undercut, both sides
	Armored bed
	Hydrophytic vegetation located high on bank
	Points of diversion for irrigation have been moved upstream
	Failed revetments due to undercutting
Evidence of aggradation	Buried structures such as culverts and outfalls
	Reduced bridge clearance
	Presence of midchannel bars
	Outlet of tributaries buried in sediment
	Sediment deposition in flood plain
	Buried vegetation
	Channel bed above the flood plain elevation (perched)
	Significant backwater in tributaries
	Uniform sediment deposition across the channel
	Hydrophobic vegetation located low on bank or dead in flood plain
Evidence of stability	Vegetated bars and banks
	Limited bank erosion
	Older bridges, culverts, and outfalls with bottom elevations at or near grade
	Mouth of tributaries at or near existing main stem stream grade
	No exposed pipeline crossings, bridge footings, or abutments

During field work, it is important to locate and observe both stable and unstable areas within the study reach. By observing the areas with the worst problems, the upper limits of erosion, sedimentation, and flooding can be established. It is equally important to visit reaches of the system where these problems are absent or not as severe. This approach will provide an envelope of values associated with the study area and better describe the variability and physical characteristics of the stream reach.

The information gathered in the reconnaissance and detailed field investigations should be used to divide the channel into geomorphologically similar reaches. When establishing reach limits, consideration should be given to differences in channel slope, tributary locations, presence of geologic controls, planform changes, location of channel control structures (grade control structures, dams, culverts, low-water crossings), changes in bed-material size, major sediment sources (mines, construction activities, sediment laden tributaries), instream gravel mining, maintenance dredging, changes in channel evolution type, and other significant hydrologic or geomorphic changes. Initial reach limits may be made early during the field investigation, but may be refined following more detailed analyses. The choice of an assessment technique should be made with consideration of the study goals. An example of some basic assessments is shown in table 3–3 from Copeland et al. (2001).

Table 3–3 Reach condition assessment

Condition	Bed	Bank
Stable	The channel bed is as close to a stable condition as can be expected in a natural stream. The reach exhibits few signs of or minimal rates of local bed scour or deposition	The channel banks are as close to a stable condition as can be expected in a natural stream and appear to have a low potential to erode. Banks are predominantly covered with extensive vegetation, boulders, or bedrock formations. Local bank erosion is within an allowable rate of change
Moderately stable	The channel bed in the reach is in a moderately stable condition. However, the reach may be in transition. Bed aggradation or degradation occurs at a low rate of change. Moderate to high rates of local bed scour or deposition occur (rapid aggradation immediately above and scour immediately below a minor debris blockage, such as a single tree blocking the channel	The channel banks in the reach are in a moderately stable condition and exhibit medium erodibility. Banks are partially vegetated with moderately erodible soils. Typically, parallel flows do not result in bank erosion. The reach may be in transition. Banks exhibit moderate local bank erosion that does not appear to be spreading (in an otherwise stable reach, a single section of the bank could fall into the stream and result in local, moderate bank erosion)
Unstable	The channel bed in the reach is unstable. The bed is undergoing widespread bed aggradation or degradation at a moderate rate. Moderate scour occurs, and many of the pools are filled with loose sediment	The channel banks in the reach are predominantly unstable. Banks are experiencing widespread erosion at a moderate rate. Channel banks are undergoing local bank erosion at a high rate of change and where the erosion is not likely to be self healing
Very unstable	The channel bed in the reach is in a very unstable condition. Typically the channel shows no signs of approaching equilibrium with the current shape and planform. The bed is undergoing widespread aggradation or degradation at a high rate. Reaches are severely scoured, and all of the pools are filled with loose sediment	The channel banks in the reach exhibit high erodibility and do not have any controls that restrict extensive changes in planform or shape. Riparian root masses are not present to slow rapid bank retreat. Any parallel or impinging flows will cause extensive bank erosion. Reaches have near vertical to overhanging banks

At the conclusion of a field investigation, channel stability in each reach is summarized. General classification techniques are descriptions based primarily on observation and can be useful both in compiling observations and in communicating with stakeholders. Channel typing is an elementary level of stream classification that uses generic terms. Many techniques are available, and they range in complexity and required effort. The channel description may include parameters such as channel and flood plain geometry, bed and bank material, planform, vegetation, bedforms, evidence of aggradation or degradation, and grade control.

Geomorphic channel classification involves the selection of a classification system and categorizing a channel based on factors and measurements such as dominant mode of sediment transport, entrenchment ratio, and sinuosity. Some of the most widely used classification systems are described in chapter 2 of EM 1110–2–1418 (USACE 1994d) and in the FISRWG (1998). Streams can also be classified by their biota, habitat conditions, baseflow levels, and direct measures of water quality.

In summary, data obtained during the field investigation and historical data collection can be used to determine the target stream type in terms of boundary sediments, riparian vegetation, and meander patterns. In many cases, the type and density of bank vegetation will be different from that present in the reference reaches due to ecological, aesthetic, and recreational objectives. It is important that target vegetation is identified prior to channel design because it influences flow resistance. Otherwise, the stability of the restored channel could be affected.

Examples of useful tools for organizing and analyzing stream geomorphology data are the STREAM toolbox developed by the Ohio Department of Natural Resources (NEH654.10) and a streambank inventory and evaluation spreadsheet developed by Illinois NRCS that is described in NEH654 TS3C. This information also describes stream stability and equilibrium, along with a channel evolution model as background material. A detailed procedure for data collection and analysis is presented to better understand the dynamics of a target stream. Another useful tool is Stream Channel Reference Sites: An Illustrated Guide to Field Techniques (Harrelson, Rawlins, and Potyondy 1994). This publication helps in organizing and guiding field assessments and stream measurements.

(a) Geologic assessment

Geologic factors can often be complex, yet they are the foundation of the stream system. Studying both the surface and subsurface geologic conditions is fundamental to a complete understanding of the stream's morphology.

This process should begin with a study of the available geologic maps that are available at a variety of scales from both state agencies and the USGS. Geologic maps generally show whether the materials in a valley are consolidated or unconsolidated, and they indicate the parent material both underlying the stream channel and in the watershed above it. This information can be used to estimate engineering properties and erodibility of the parent material and streambanks, type and amount of sediment available for transport, and potential materials for armoring. It is critical to verify this information in the field.

The engineering properties of the parent material and its resistance to erosion can have significant effects on the morphology and stability of a stream. Where bedrock hard points are part of the streambed, downward migration is limited, and the cross-sectional flow area must be accommodated by lateral erosion. Alternately, if bedrock hard points occur in a streambank, lateral migration is limited, and the cross-sectional flow area will be accommodated by downcutting.

Determining the type and amount of sediment available for transport within the watershed is an important part of the design process. In areas of high erosion rates, significant amounts of sediment can be delivered to the channel, and the quantity and particle-size distribution must be considered. For example, sparsely vegetated desert conditions can contribute enough sediment during rare, but high flows to overwhelm the stream completely. Badland conditions, such as those in the Dakotas, can form in soft, unvegetated shales, and also contribute significant amounts of sediment.

The geology can vary significantly even across small reaches, and its effects can be different depending on the location within the stream system. Different aspects of the geology may be important depending on

whether the stream is in the erosional reaches near the headwaters, in the transporting portion of the mid-stream reaches, or in the depositional reaches near the lower end of the channel system.

Changes that have occurred at the site through geologic time must also be considered. The tectonic history, climatic changes such as ice ages, and other surficial processes are reflected in the current morphology of the stream channel. For example, faults can create soft zones in otherwise hard bedrock that will be more susceptible to erosion and channel development. In addition, the materials in the streambanks are a reflection of the stream's former positions within the landscape (upland, hillslope, fan, terrace, valley bottom, delta) and its previous erosional and depositional history.

In most of North America, the climate has changed drastically since the end of the Pleistocene Epoch, about 100,000 years ago. The climate was significantly wetter, runoff was generally higher, and glacial meltwater carved huge channels still in evidence today. Paleochannels that formed during that time have not experienced significant changes in areas with no active tectonic forces. For example, the Missouri River in northern Montana was pushed south from its original channel by continental glaciers during the last ice age. The Milk River, a much smaller system, now flows through this old channel, appearing to be underfit to the higher flow conditions that formed it.

The channels that were formed during higher flows are composed of coarser grained materials. They are overlain by finer grained materials deposited by today's lesser flows. This situation can be highly susceptible to erosion, but might not be considered without knowledge of the paleoenvironment. In particular, fines and sand can be washed out of gravel deposits during bankfull flows, especially on outside curves. This can undermine the streambank, creating an overhanging condition that fails under its own weight. Finer materials above may be cohesive, exhibiting increased shear strength, but once undermined, will fail and add sediment to the stream.

Coarser grained deposits provide higher resistance to flow than fine-grained deposits. Gravelly stream channels are considered to have formed from lateral accretion, or the extension of gravel bars, and finer textured deposits are considered to have formed from vertical accretion.

Some geologic conditions promote higher bank stability. For example, preconsolidated glacial till and wind-deposited loess both create stable bank configurations, even with high, vertical banks. Peat that is formed in marshy conditions also may form a stable, vertical streambank, if it is not interlayered with other materials.

In general, geologic conditions play an important role in the development of the stream morphology. These conditions should always be thoroughly considered in an interdisciplinary stream study.

(b) Biological assessment

Watersheds are complex systems that integrate many factors. For this reason, a select group of indicators is often examined to infer watershed condition. For instance, instream habitat features, such as riffles, can be used to assess fish productivity potential. The U.S. Environmental Protection Agency (EPA) reviewed stream assessment protocols that range from subjective, visual-based protocols to objective, quantitative assessments that are time consuming measurement-based methods (EPA 2004). Some protocols provide unique approaches or particularly useful methods to address aspects of stream assessment and mitigation. For instance, the Eastern Kentucky Stream Assessment Protocol from the USACE Louisville District, incorporates a wealth of biological data into the calibration of the stream assessment method and integrates biotic and abiotic factors of fluvial systems in eastern Kentucky. The Integrated Streambank Protection Guidelines from the Washington Department of Fish and Wildlife (WDFW) (2003) uses a series of sequential or hierarchic matrices to aid practitioners in selection of practices to treat eroding streambanks (EPA 2004).

Like the canary in the mine, the health of indicator species can be used to reflect the general health and well being of a riparian system and watershed. A somewhat unique example of an indicator species is riparian bats (NEH654 TS3D).

Biotic indicators

Biotic indicators are widely used to assess water quality. Biotic indicators are effective in assessing both past and present human activities on the watershed. While numerous biotic indicators exist, two common practices are briefly described here: the Index of Biotic

Integrity (IBI) and the Ephemeroptera, Plecoptera, and Trichoptera (EPT) Index.

The IBI uses fish surveys to assess human effects on a stream and its watershed. The EPT Index uses benthic macroinvertebrates, such as stoneflies, mayflies, and caddisflies, as indicators to assess land use and water quality within a watershed.

The presence, relative abundance, and diversity of aquatic macroinvertebrates may also be direct or indirect indicators of the surface water regime (Water Quality Field Guide, SCS–TP–160; and Water Quality Indicators Guide, SCS–TP–161). Rocks, sediment, and leaf accumulations can be searched in riffle and pool areas, since they are normally the first and last areas to dry up. The presence of a variety of species from the orders Plecoptera (stoneflies), Ephemeroptera (mayflies), Trichoptera (caddisflies), Coleoptera (aquatic beetles), Diptera (cranefly), and others suggest persistent flow. Intermittent streams generally lack macroinvertebrates, though occasionally, a few early successional species can invade and dominate that niche during wet periods. With no persistent flowing water, pools, or saturation, ephemeral channels normally do not have aquatic insects. The presence of other organisms, such as freshwater mussels, crayfish, or snails, may be helpful when compiling evidence to determine stream type.

Biotic factors, particularly characteristics of stream biota, have been used with great success to evaluate watershed conditions and are one of the oldest approaches to assess water quality. However, biotic indicators have disadvantages in comparison to other indicators. Biotic indicators are not as visible as habitat indicators. For example, a stream habitat feature, such as a sloughing bank and the resulting increase in sediment, is more easily documented than the subtler effect of sediment on biotic communities in the stream.

IBI—The IBI was developed to help resource managers sample, evaluate, and describe the condition of small warm-water streams in central Illinois and Indiana (Karr 1981). The IBI became popular for assessing warm-water streams throughout the United States. Karr and his colleagues explored the sampling protocol and effectiveness in several different regions and on different types of streams. As the IBI became widely used, different versions were developed for different regions and ecosystems. The original version had 12 metrics that reflected fish species richness and composition, number and abundance of species, trophic organization and function, reproductive behavior, fish abundance, and condition of individual fish. The metrics were scored and summed to arrive at an index ranging from 60 (best) to 12 (worst). Newer versions generally retain most of the original metrics, but some have been modified to improve sensitivity to environmental degradation in a particular region or type of stream. The IBI has also been tailored to reflect differences in fish species within a region, and in other types of ecosystems such as estuaries, impoundments, and natural lakes.

Fish are useful in measuring degradation for many reasons:

- Fish are sensitive to a wide array of stresses.
- Fish integrate adverse effects of activities in the watershed.
- Fish are long lived. Their populations show effects of reproductive failure and mortality in many age groups, thereby providing a long-term record of environmental stressors.

To develop an IBI, a 30-foot-wide stream typically requires a four-person team (fig. 3–2). The team samples in an upstream direction using a seine or electroshocker to sample the stream.

Figure 3–2 To develop the IBI, fish samples are collected by means of seines or electroshocking devices.

A state permit is often required for fish collection. Federal permits from the National Oceanic and Atmospheric Association National Marine Fisheries Service (NOAA Fisheries Service) and/or U.S. Fish and Wildlife Service (USFWS) may be required for fish collection, as well. While techniques for fish sampling vary, some studies use a 300-foot stream length. Others may use species-area curves to determine the stream sample length. For detailed information on sampling techniques and development and analysis of an IBI, see the NRCS National Biology Handbook (NHB), part 190.

Both left and right banks of the stream are sampled, taking care to include all stream habitats such as riffles, pools, runs, snags, undercuts, and deadfalls. Stunned or seined fish are netted and placed in buckets until the end of sampling. At the end of the section, the team pauses and allows the water to clear. The team then returns downstream to the starting point, repeating the sampling procedure along the way. Once back at the starting point, all fish are identified by species, counted, and measured. Sores and fish anomalies are also noted. In general, fish species identification requires a trained biologist or person familiar with fish assemblages in the area. Data are recorded, and fish that can not be identified are preserved and sent to a laboratory for analysis. Fish are then returned to the stream after completion of sampling and data recording. IBI scores are determined in the office, using 10 to 12 metrics tailored for the area. An example of the metrics and a brief description are presented in table 3–4 (North Carolina Department of Environment and Natural Resources (NCDENR) 1997).

The example metrics shown in table 3–4 are from piedmont streams. Metrics are tailored to a particular region and are generally available through state departments of water quality.

Table 3–4 Example of metrics used to construct an IBI in Piedmont streams

Metric	Description
Number of fish species and individuals	The total number of species and individuals supported by the stream will decrease with environmental degradation
Number of darters	Darters are sensitive to environmental degradation. Darter habitats may be degraded as the result of sedimentation, and channelization
Number of species of sunfish	These species are particularly sensitive to sediment filling pools and loss of instream cover
Number of species of suckers	Suckers are intolerant of chemical and habitat degradation, and because they are long lived and provide a multiyear perspective
Number of intolerant species	Intolerant species are most affected by stream degradation, and therefore would disappear by the time a stream is rated as fair
Percentage of tolerant species	Tolerant species are present in moderate number, but become dominant as stream degrades
Percentage of omnivores (plant eaters), insectivores (insect eaters), and piscivores (fish eaters)	These are the trophic groups. The trophic groups describe what the fish species eats and where it is in the food web. Deviations from what is expected are noted. For example, the cause of a great number of omnivores than insectivores is nutrient enrichment
Percentage of diseased fish	Skeletal anomalies, fin damage, disease, and tumors increase with stream degradation
Percentage of species with multiple age groups	Determines reproductive success of the fish populations

EPT—Benthic macroinvertebrates are small stream-inhabiting creatures that are large enough to be seen with the naked eye. They spend all or part of their life cycle in or on the stream bottom. The name benthic macroinvertebrate means bottom-dwelling (benthic) small organisms without backbones (invertebrate). Since benthic macroinvertebrates do not move about like fish, they provide an indicator of what has affected the immediate area where they are found. Benthic macroinvertebrates have adapted to life in a stream, using all habitat niches. For example, some are adapted to higher velocity portions of the stream, some live below the bottom of the stream, some crawl for food, while others let the food come to them. Healthy streams can have several hundred kinds of benthic macroinvertebrates.

The EPT Index is named for three orders of aquatic insects that are common in the benthic macroinvertebrate community: Ephemeroptera (mayflies), Plecoptera (stoneflies), and Trichoptera (caddisflies). The EPT Index is based on the premise that high-quality streams usually have the greatest species richness. Many aquatic insect species are intolerant of pollutants and are not found in polluted waters. The greater the pollution, the lower the species richness expected, as only a few species are pollutant tolerant. Some basic identification features of stoneflies, mayflies, and caddisflies are shown in figure 3–3. The common mayfly is up to 1 inch in length (without tail), and has three distinct fuzzy or threadlike tails, and green, brown, gray, but usually black color. Mayflies have variable tolerance to pollution, but are usually considered to inhabit cleaner waters. The common stonefly measures less than 1 inch in length (without tail), and has two wings, two sets of branched gills between the underside of the body, and yellow to brown color. The stonefly is not tolerant to low levels of dissolved oxygen and therefore prefers cold, swift-moving streams. Stoneflies are an important source of food for trout. The streamlined, flat body of stonefly nymphs enables them to move about the streambed in rapid currents. The caddisfly (which resembles a caterpillar) has a soft, wormlike body, a hard covering on the head, and yellow or brown, but usually green color. Larvae build hollow cases that either carry or attach to small rocks. Cases are built from sand, twigs, small stones, crushed shells, or rolled leaves, and are used for protection and pupation. Caddisflies have a large range of tolerance to pollution. Note that identification of many species is straightforward, while others require microscopic identification, requiring expert assistance.

Figure 3–3 Aquatic insects

Common mayfly, Ephemeroptera group

Common stonefly, Plecopetera group

Caddisfly, Trichoptera group

Features of an EPT Index—The EPT Index method uses a rapid sampling technique for determining between-site differences in water quality or for watershed studies with a large number of sites, and emergency sampling where it is desirable to rapidly assess the effects of spills and unusual discharges. The EPT Index should not be used in areas that naturally are known to have low EPT species richness (either inherent or human induced) or in areas where more pollution-tolerant groups are of interest.

The EPT Index is a versatile index because of certain characteristics of benthic macroinvertebrates. Benthic macroinvertebrates are sensitive to stress, both natural and human induced. When their environment is affected either by human or natural causes, the population will change, leading to an impaired or imbalanced community. Much like the canary in the coal mine, the response of aquatic insects gives an early warning of possible harm to a water body. Because many aquatic insects spend their entire lives within aquatic systems, they show the effects of physical habitat alteration, point and nonpoint contaminants, and cumulative pollutants over their life cycle. Other important features of aquatic insects are that they:

- are found in all aquatic environments
- exhibit diversity and are sensitive to pollution
- display a wide range of responses to pollution
- are less mobile than many other groups of organisms (fish)
- are often of easily collectible size

Like all biotic indices, the EPT Index can be used when chemical and physical measurements of a complex mixture of pollutants are not feasible. Moreover, these aquatic insects show responses to a wide array of potential pollutants and are sensitive to both short- and long-term conditions affecting water quality.

Collecting samples to construct an EPT Index—Benthic macroinvertebrates are collected using a variety of methods. The suite of sample collection techniques described consists of the kick-net sample, sweep-net sample, leaf pack sample, and visual collections (EPA 1999b). These techniques are aimed at sampling the favorite habitats and food sources of the aquatic insects. Stream food resources are larger organic matter particles in leaf litter and large woody material; smaller organic matter particles in suspended materials and sediments; and diatoms, algae, and other materials growing on rocks, wood, and plants; and prey (Hauer and Lambert 1996).

Each macroinvertebrate occupies a certain niche according to its feeding group: shredders, collector-gatherers, scrapers, filterers, or predators. Shredders prefer to feed on larger particles of organic matter such as leaves and twigs, in turn churning these into smaller organic matter that can be fed upon by collector-gatherers. Collector-gatherers feed on small particles of organic matter in or on the bottom of the stream. Scrapers feed on diatoms and algae that are attached to underwater surfaces. Filterers feed by straining small organic matter particles out of the water. Filters can be fanlike appendages on the insect's body or built externally by the insect to resemble little underwater nets.

Predators feed on other macroinvertebrates. In healthy streams, all feeding groups are present. Stream impairment may be indicated when one or more feeding groups are missing from a stream. In general, stoneflies are predators, mayflies are scrapers or collectors, and caddisflies are scrapers, collectors, or shredders. The ratio and number of these macroinvertebrates change with the stream food resources and human impacts and, therefore, can be used as a tool for assessing the ecological status of the biotic community and the water quality.

The kick sample is conducted using a rectangular section of window screening attached between two poles. The net is positioned on the stream floor, downstream of the sampler. One person holds onto the net. The other person disturbs the stream bottom upstream of the net and kicks the invertebrates present into the net. Invertebrates collected on the net are washed into a bucket. A long-handled triangular net is also used to disrupt and sweep areas under banks, root masses, and mud banks. Netted invertebrates are washed into a bucket. This procedure collects mayflies and caddisflies which prefer low-current environments. Leaf packs in the stream, snags, sticks, and small logs are examined and macroinvertebrates separated into a bucket. In general, shredders such as the caddisflies prefer these environments. A final visual search of upturned rocks, cobbles, and logs is conducted to collect adhering macroinvertebrates. For example, rocks in low current areas harbor stoneflies. Macroin-

vertebrates are separated or picked from the bucket samples with forceps and placed in vials containing ethanol for later classification and counting.

Macroinvertebrates usually require identification in the laboratory by a trained biologist. However, community watch group volunteers, teachers, and students can be trained to make basic identifications of the three groups used in the EPT Index. The NRCS Stream Visual Assessment Protocol (SVAP) also uses aquatic insects to assess stream condition (USDA NRCS 1999b).

EPT Index score development—The EPT Index is the total number of distinct taxa within the groups Ephemeroptera, Plecoptera, and Trichoptera. For example, if five species of Ephemeroptera (mayflies), five Plecoptera (stoneflies), and two Trichoptera (caddisflies) are found at a site, the total number of EPT taxa and Index would equal 12. The EPT Index is then compared to values on an EPT rating chart that has been developed for that particular region. Many state water quality departments are a good source of information on how to develop a rating chart for a particular ecoregion. The EPT Index increases with improving water quality; that is, there should be a greater number of EPT insect taxa in cleaner water. Ratings are tailored to account for differences in species pollution tolerance between regions. Table 3–5 (modified from NCDENR 1997) shows an example of EPT criteria developed for the Southern Piedmont of North Carolina. In this example, a site with an EPT Index of 12 would have a rating of fair.

The EPT Index can be used to directly assess the cumulative effects of all activities in the watershed. These results allow establishment of baseline or reference conditions for watersheds to characterize their overall condition, identify potential nonpoint and point source pollutants, target resource efforts in impaired watersheds, and evaluate the effectiveness of pollution control measures.

Beavers and beaver management

Beavers were among the most widely distributed mammals in North America, and they were eliminated from much of their range by the late 1800s because of unregulated trapping. Beavers eat the leaves, inner bark, and twigs of aspen, alder, birch, cottonwoods, willows, and other deciduous trees. Conifers such as fir and pine are eaten occasionally. They also eat shrubs, ferns, aquatic plants, grasses, and crops such as corn and soybeans. Beaver dams are created by mud, rocks, and whatever other materials are available to the beaver.

Beaver dams create backwaters that flood areas upstream. This provides protection from predators, access to a food supply and their dens, and wet areas that promote the growth of their favorite foods. Because this backwater may also flood roads, fields and other land, much interest has been placed on beaver management. Beaver management involves trapping and relocation (Tippie 2003); installing flow devices to encourage dam building at more desirable locations (Lisle 2004); and using pond levelers to control water depth and reduce flooding (Snohomish County Public Works 2004; Cooperative Extension Service, Clemson University 1994).

Table 3–5 Example of EPT index ranges and their corresponding water quality ratings for southern Piedmont, NC

Rating	Excellent	Good	Good-fair	Fair	Poor
EPT	>27	21–27	14–20	7–13	0–6

654.0305 Stream classification systems

This description of stream classification systems is designed to help users understand the variety of different systems and their relationship to channel stability, basin geomorphology, riparian and aquatic ecosystems, and watershed condition. Its goal is to help stream professionals recognize how the effectiveness and longevity of riparian restoration activities are related to basic stream classification techniques. Readers can learn the basic terminology of each classification system and acquire sufficient background to communicate with peers and producers about the differing systems. While many other techniques exist, four stream classification methods are presented in this chapter. These are listed in table 3–6. The descriptions provided herein attempt to promote an understanding of the strengths, weaknesses, and limitations of the presented systems.

(a) Overview of stream classification systems

Stream classification systems have been in use in their simplistic forms for at least a hundred years (Davis 1909). Much of the basis for modern stream classification systems, however, began in the 1950s and 1960s with work by Leopold and Wolman (1957), Lane (1957), and Schumm (1963).

River and stream systems are dynamic and continually respond to changes in sediment load, hydrology, and form. Under the current watershed conditions, stream classification systems help users understand the present and expected future status of a stream system. The strengths and weaknesses of these classification systems are described, but the description does not compare one system with another.

Four different types of classification systems are presented in this chapter. The Framework and Integrated Guide includes a listing of classification and mapping criteria. The channel evolution model (CEM) is an example of a system based on nonstable processes. The Montgomery and Buffington system is based on defining channel processes, and the Rosgen system is a classification of the current status of the channel. Each of these classification systems was designed to address a specific set of practical requirements by its developers and as a result, each has specific application areas in which it is strongest and weakest. No one system works for all situations, and professionals working in the field of stream restoration are well advised to match the appropriate classification system to the problem at hand.

Table 3–6 Stream classification systems

Stream classification	Full name	Basis
USDA Forest Service aquatic framework	Framework of Aquatic Ecological Units, and Integrated Resource Inventory Training Guide	Consistency of classification criteria
Schumm, Harvey, Watson, and Simon	Channel evolution model (CEM)	Channel response
Montgomery and Buffington	Classification of Channel Reach Morphology for Mountain Streams in the Pacific Northwest	Channel processes
Rosgen classification	Classification of Natural Rivers	Current channel condition

(b) USDA Forest Service: Framework of aquatic ecological units and the Integrated Resource Inventory Training Guide

The USDA Forest Service developed an aquatic framework (Maxwell et al. 1995) that contains standard terms and classification criteria for aquatic systems and their linkages to terrestrial systems at all spatial scales. Its purpose is to ensure consistency in classifying and mapping aquatic systems, and therefore, enhance the analysis of aquatic systems to reflect their varied forms and functions. The Forest Service has also developed the Integrated Resource Inventory Training Guide, Chapter 3, Common Water Unit (USDA Forest Service 1997a) that has tables of the classification criteria based on the aquatic framework (tables 3–7 and 3–8 (Frissell et al. 1986; Montgomery and Buffington 1993a; Paustian et al. 1992; and Rosgen 1994, USDA Forest Service 1997a). Major stream types are defined by channel entrenchment, shape, and sinuosity.

Potential NRCS use of the framework will primarily be as a guide for data collection and field mapping of stream reaches. The framework does not include estimates of what the next evolutionary phase of a stream might be. The stream reach classifications of Frissell et al. (1986), Montgomery and Buffington (1993a), Paustian et al. (1992), and Rosgen (1994) are based on a group of common geomorphic factors that are included in the framework. For use in classification, the most helpful sections of the framework are valley segments and their subdivision, stream reaches. Stream reaches are defined as uniform in flow and channel morphology and have discrete patterns of aquatic habitats and fluvial processes. A small set of stream reaches is nested within any valley segment.

Strengths

The aquatic framework contains a listing of mapping and classification criteria that are used in several stream classification systems. With the collection of the stream attributes, the user could assign a channel type to a reach in several systems. If the reach is classified in several systems, this method has the advan-

Table 3–7 Defining criteria for classifying stream reach types

Variable	Description
Channel pattern	The plan view of the stream reach. Geomorphic controls and sediment transport regimes create straight, sinuous, meandering, tortuous, braided, and anastamosing channels. Sinuosity is used to describe the overall channel pattern. Sinuosity is the length of the active channel divided by the length of the valley. This attribute is map and photo interpreted
Channel entrenchment	The degree to which the stream is incised into the landscape. This criterion indicates how well floods are contained by a stream channel. It is the width of the flood-prone area divided by the width of the active, or bankfull, stream channel. The flood-prone area is the width of the valley floor at a level corresponding to twice the maximum bankfull depth of the channel. This attribute is field observed
Bank stability	Can be reduced by natural events (floods, fire, landslides) or human disturbances (grazing, logging, roads) that change runoff amounts, sediment loads, and bank vegetation Vegetated–stable. Bank is vegetated with no evidence of active erosion or sloughing and no tension fractures Vegetated–unstable. Bank is vegetated, but tension fractures exist at the top of the bank Unvegetated–stable. Bank is not vegetated, but is composed of bedrock or stable boulders or cobbles Unvegetated–unstable. Bank is not vegetated and is composed of bare gravel, sand, silt, or clay, or a matrix of cobbles and these finer particles
Woody material	Large woody material usually improves habitat complexity and quality in a stream reach, often forming pools. All pieces of large woody material that span the channel or lie totally or partially within it are counted
Temperature	Reflects both the seasonal change in net radiation and the daily changes in air temperatures. It is affected by flow velocity and depth and ground water inflow

tage of compounding the individual system strengths. This method includes some stream health attributes that could be used to diagnose the condition of the stream reach against a reference healthy stream reach. The system is being used by the USDA Forest Service for mapping of aquatic systems, and data already collected would be available for National forests.

Weaknesses

The aquatic framework classification does not have specific recommendations to determine evolutionary trends for each type of stream reach.

Table 3–8 Stream type classes, modifiers, and bed structures

Class	Channel entrenchment	Width-to-depth ratio	Sinuosity	Description
A	<1.4	>12	<1.2	Straight, steep, entrenched, narrow stream
B	1.4–2.2	>12	>1.2	Moderately sinuous, moderately sloped, moderately entrenched stream
C	>2.2	>40	>1.4	Meandering, low-gradient alluvial stream with broad flood plain
D	n/a	40	n/a	Braided, wide, multiple streams with many bars and eroding bank
DA	>4.0	<40	Variable	Anastomosing, flat, narrow multiple streams with stable banks
E	>2.2	<12	>1.5	Tortuous, narrow stream with broad flood plain and stable banks
F	<1.4	>12	>1.4	Meandering, low-gradient, wide, entrenched stream with eroding banks

Modifiers

Materials		Slope	
1	Bedrock	h	Hydraulic (over 10%)
2	Boulder (over 256 mm)	a	Aggressive (4.0–9.9%)
3	Cobble (64–256 mm)	b	Balanced (1.5–3.9%)
4	Gravel (2–64 mm)	c	Cumulative (0.5–1.4%)
5	Sand (0.062–3 mm)	f	Flat (under 0.5%)
6	Silt/clay (under 0.062 mm)		

Bed structure

PR	Pool-riffle (alternating pools and riffles)
PB	Plane-bed (lacking distinct bedforms)
SP	Step-pool (alternating pools and vertical steps)
C	Cascade (tumbling flow over disorganized large rocks)

(c) Channel evolution model

During the 1960s, several stream channels in northern Mississippi were channelized to control out-of-bank flooding. Major incision of the channel (down cutting) occurred from the late 1960s through the 1980s. Subsequently, a geomorphic study was conducted on several of the streams, and the investigations identified a sequence of steps through which all the channels had evolved. This channel evolution model (CEM) describes a predictable sequence of change in a disturbed channel system that was characterized as moving from reach types I through V (fig. 3–4a and table 3–9 (Schumm, Harvey, and Watson 1981)).

The model was developed by Schumm, Harvey, and Watson (1981) from investigating three unstable, channelized watersheds in northern Mississippi: Pigeon Roost Creek, Oaklimiter Creek, and Tippah River. The streams in these watersheds have mainly cohesive bank soils. The increased slope of the constructed channels started a process of significant down cutting after the channelization was completed. Starting at the oversteepened reach, the five types of channel reaches generally can be seen going downstream. In Schumm, Harvey, and Watson (1981), the series of five channel reach types as identified for Oaklimiter Creek can be characterized as shown in table 3–9.

Additional information was obtained from a study on Hotophia Creek Watershed in 1987. This study refined the CEM by introducing a ratio for critical bank height to bank height for each channel type. If the bank height (h) exceeds the critical bank height (h_c), gravity failure is imminent. For Type I, $h < h_c$; for Type II, $h > h_c$; for Type III, $h > h_c$; for Type IV, $h \sim h_c$; and for Type V, $h < h_c$.

A modification to this model was proposed by Simon (1989). This is the CEM that is most typically preferred. The modification by Simon included an extra step to account for channel modification and is perhaps more widely recognized. It is shown in figure 3–4b. The Simon model identifies six stages through which a stream progresses when subjected to destabilizing influences such as the urbanization described earlier in this chapter. Each of these stages is referred to as a class.

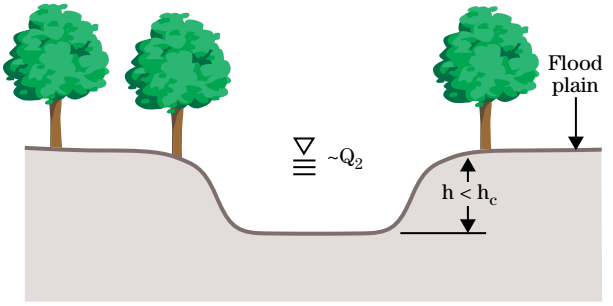
In the Simon (1989) model, class I is the natural channel before modification; class II represents the stream channel morphology directly after human activity such as channel straightening. This class is the new stage added by Simon.

Table 3–9 Characteristics of channel reaches using the CEM (see fig. 3–4a)

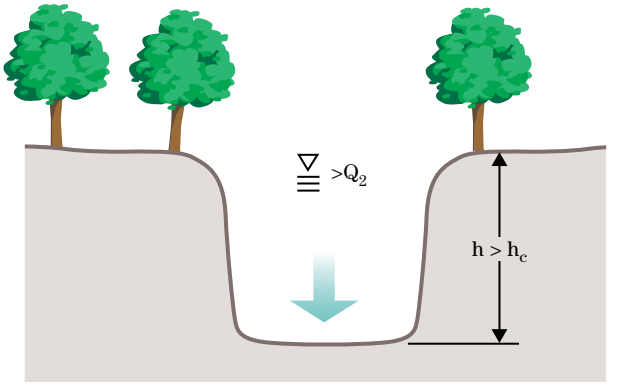
Types in a downstream direction	Sediment storage	Shape	Location and stability	Width-to-depth ratio (F)
Type I	Very little or none	AU ≈ shaped	Upstream of active nickpoints, have oversteepened slopes	Highly variable 4.0–7.0
Type II	Variable	Steep vertical channel banks and increased depth	Immediately downstream of active nickpoints, degrading	30–4.0
Type III	1.5–2.0 ft	Banks failing	Active channel widening and degrading	≈5.0
Type IV	2.5–3.5 ft	Low water sinuous thalweg	Reduced rate of active channel widening, aggrading, beginning of quasi-equilibrium	≈6.0
Type V	Up to 6 ft	Alternate bars	Aggrading, quasi-equilibrium	>8

Figure 3–4a Schumm, Harvey, and Watson (1981) schematic cross sections and longitudinal profile of an incised stream showing features of the five classes of the CEM

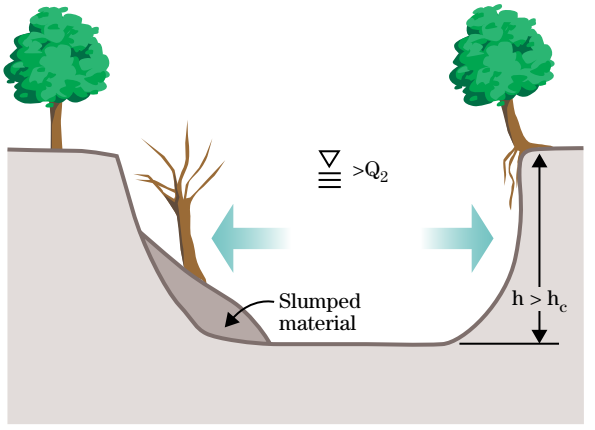
Type I–Stable



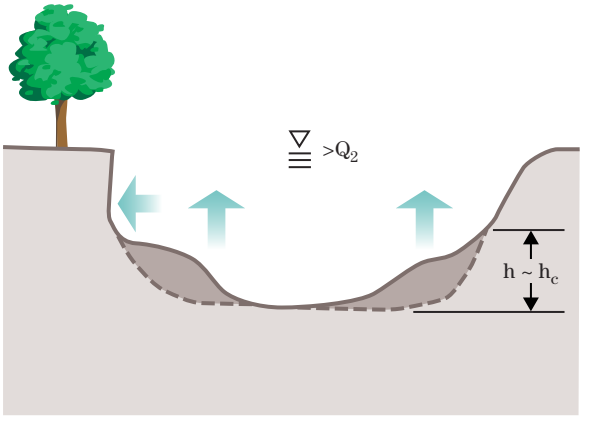
Type II–Incision



Type III–Widening



Type IV–Depostion/stabilizing



Type V–Quasi-equilibrium stable

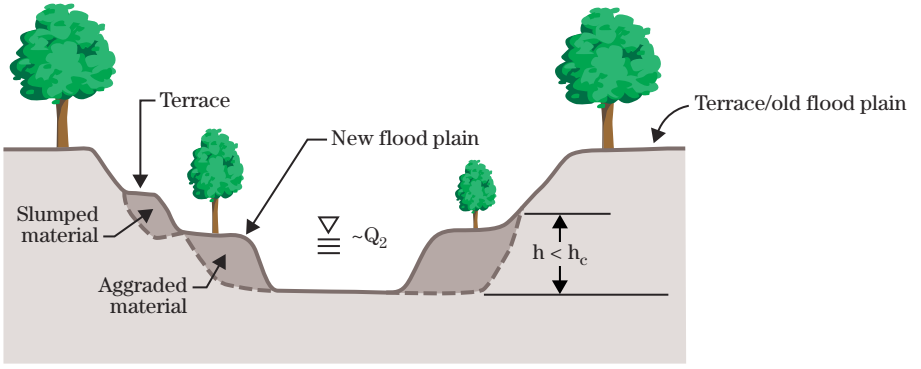
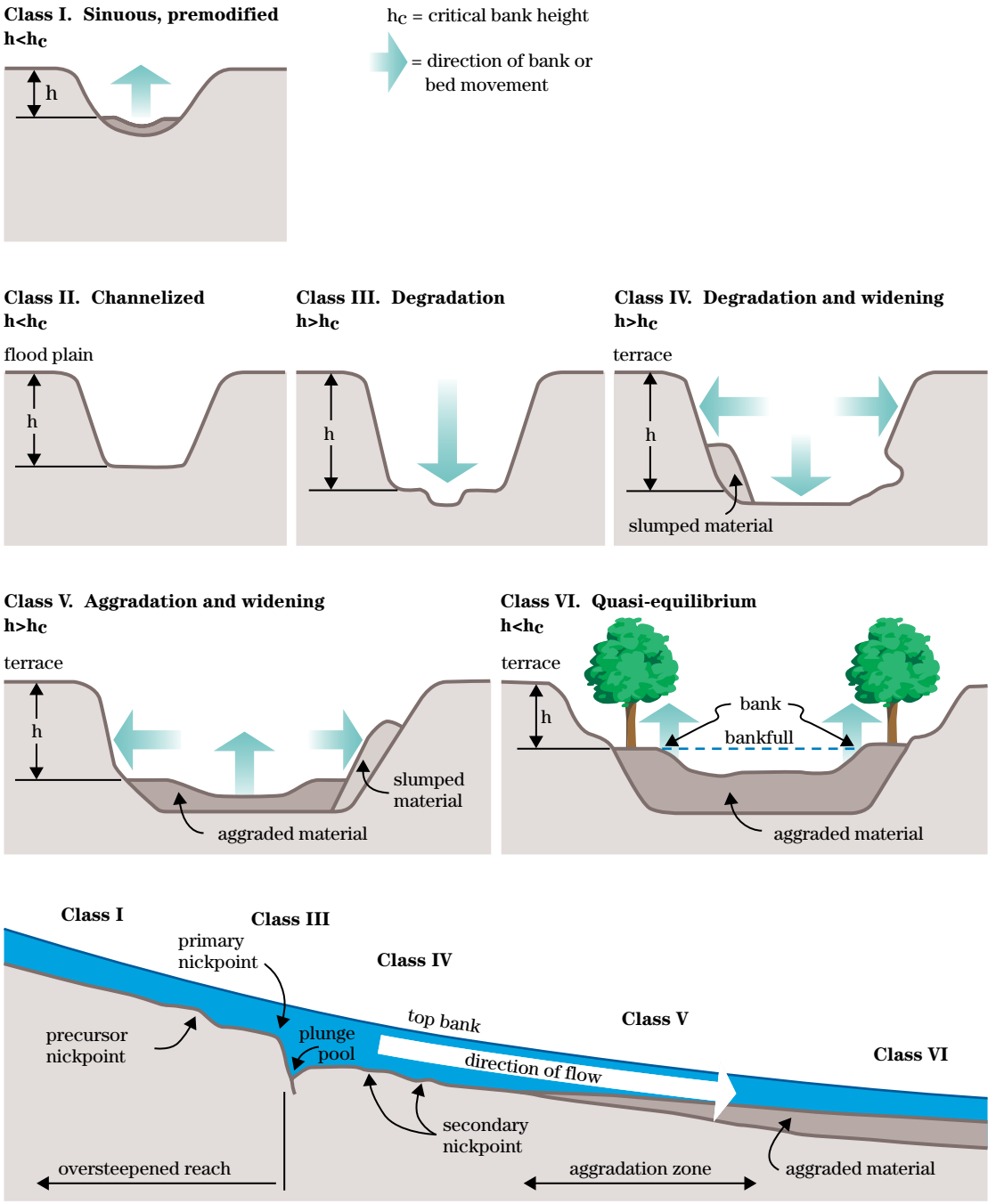


Figure 3–4b Simon (1989) schematic cross sections and longitudinal profile of an incised stream showing features of the five classes of the CEM



Class III is then the first sign of an instability problem, with evidence of downcutting or degradation in the channel bottom. Class III of the Simon 1989 model corresponds to Type II of the Schumm, Harvey, and Watson (1981) model.

As the bottom of the channel changes elevation, support for the banks is removed and the streambanks slump, creating a widening channel shape (class IV of the Simon 1989 model). It corresponds to Type III of the Schumm, Harvey, and Watson (1981) model.

At some point, a new equilibrium is being approached. The sediments from the slumped banks begin to form new, vegetated flood plains at a lower elevation (class V) and a smaller, natural channel within the new banks. It corresponds to Type IV of the Schumm et al. 1981 model. The new stream equilibrium (class VI) has abandoned the former flood plain and created a new one at the lower elevation (FISRWG 1998). This new stream equilibrium corresponds to the Type V of the Schumm et al. 1981 model.

Typical streams will exhibit several of the classes defined in the Simon CEM, depending on the location in the stream relative to the disturbance. The last part of figure 3–4b illustrates a nickpoint: the head of an active erosion event in the stream channel, working its way upstream. Class I describes the state of the stream well above the nickpoint where the effects of the disturbance are not yet in evidence. Progressing downstream, this figure illustrates the primary nickpoint (class III), and varying stages of bank instability in the wake of the nickpoint (classes IV and V). If enough time has passed since the disturbance, conditions farther downstream will approach class VI.

Strengths

The CEM was developed to help predict the changes a channel makes going through the process of headcutting. The CEM is based on geomorphic measurements of a reach of the channel system both upstream and downstream of a headcut. As a result, it is most accurate in its descriptions of what the next stage will be for the disturbed channel. The CEM is most valuable when verified for the watershed of interest. The CEM provides the kind of condition and trend information that is useful for shareholders and engineers to choose and design practices that are most cost-effective and have a greater probability of success. This model provides a means of segregating stream reaches into those requiring lesser or greater intervention to achieve a stable condition. For example, at Simon (1989) model class III (degradation), achieving a successful restoration is likely to be expensive, if at all possible. On the other hand, at class V (aggrading and widening), little effort may be required other than revegetation to speed the recovery process.

Weaknesses

Both the Simon (1989) and the Schumm, Harvey, and Watson (1981) models require a geomorphic study to determine reach stability values. It only applies in watersheds with degraded channels, and it works best in watersheds with fairly uniform soils and geology. Therefore, it is not as useful in systems with highly variable soils, grade, or planform control. The model has three assumptions that may limit its broad application:

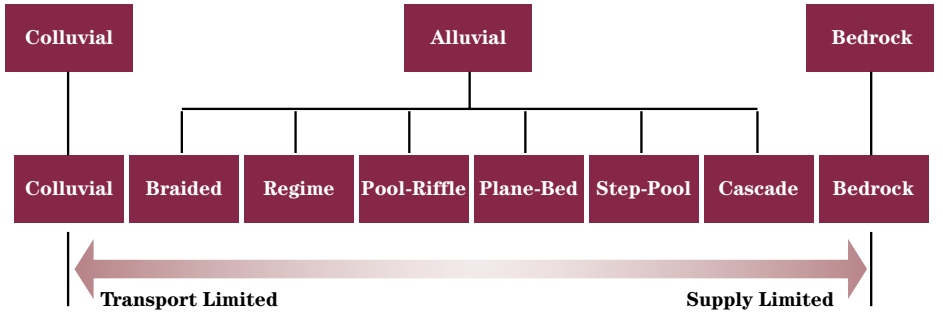
- channel base level will not change
- channel is formed in alluvial material that permits all types of channel adjustment
- land use of the watershed will not change greatly

(d) Montgomery and Buffington classification system

The Montgomery and Buffington (1993a) system classifies channel reach morphology for forested mountain streams. The authors emphasize that there are very distinct differences between mountain channels and their lowland counterparts. Most of the field observations used to develop their system were made in Washington, Oregon, and Alaska. The persistence of significant quantities of large woody material in these mountain channel systems makes the current application of this classification system somewhat regional and unique. Further testing has definite potential to validate its application to other mountainous regions of the country. The morphological processes described by the authors may serve as a template for developing other regional classification systems.

Mountain streams can be categorized into erosional (sediment supply source), transporting, and depositional reaches (fig. 3–5). Montgomery and Buffington have expanded this process-based concept to include a number of channel types in each of the three geo-

Figure 3–5 Montgomery and Buffington stream classification system



	Braided	Regime	Pool-Riffle	Plane-Bed	Step-Pool	Cascade	Bedrock	Colluvial
Typical Bed Material	Variable	Sand	Gravel	Gravel, cobble	Cobble, boulder	Boulder	N/A	Variable
Bedform Pattern	Laterally oscillary	Multi-layered	Laterally oscillary	None	Vertically oscillary	None	•	Variable
Reach Type	Response	Response	Response	Response	Transport	Transport	Transport	Source
Dominant Roughness Elements	Bedforms (bars, pools)	Sinuosity, bedforms (dunes, ripples, bars) banks	Bedforms (bars, pools), grains, LWD, sinuosity, banks	Grains, banks	Bedforms (steps, pools), grains, LWD, banks	Grains, banks	Boundaries (bed & banks)	Grains, LWD
Dominant Sediment Sources	Fluvial, bank failure, debris flow	Fluvial, bank failure, inactive channel	Fluvial, bank failure, inactive channel, debris flows	Fluvial, bank failure, debris flow	Fluvial, hillslope, debris flow	Fluvial, hillslope, debris flow	Fluvial, hillslope, debris flow	Hillslope, debris flow
Sediment Storage Elements	Overbank, bedforms	Overbank, bedforms, inactive channel	Overbank, bedforms, inactive channel	Overbank, inactive channel	Bedforms	Lee & stoss sides of flow obstructions	•	Bed
Typical Slope (m/m)	$S < 0.03$	$S < 0.001$	$0.001 < S$ and $S < 0.02$	$0.01 < S$ and $S < 0.03$	$0.03 < S$ and $S < 0.08$	$0.08 < S$ and $S < 0.30$	Variable	$S > 0.20$
Typical Confinement	Unconfined	Unconfined	Unconfined	Variable	Confined	Confined	Confined	Confined
Pool Spacing (Channel Widths)	Variable	5 to 7	5 to 7	none	1 to 4	< 1	Variable	Variable

morphic zones (Montgomery and Buffington 1993a; Montgomery and Buffington 1997).

A net reach response is dependent on the size and amount of sediment available to transport compared to the reach's hydraulic transport capacity. Reaches with more sediment supply than sediment transport capacity are erosional or source reaches. These reach types usually occur in the headwaters of mountain streams. Some length of midreach stream tends to achieve a balance between sediment load and transport capacity. These reaches are identified as transport reaches. These middle stream transport reaches may be relatively short in some stream systems and quite long in others, depending on the relative balance of sediment supply and size compared to transport capacity. Middle stream reaches tend to exhibit a net long-term balance between aggradation and degradation which is inherent in most definitions of stream stability. A net, long-term sediment balance within a stream reach may not necessarily translate to stream stability because of extreme fluctuations in sediment load and continuous change instream geometry. Finally, the lower end of mountain stream systems are typically depositional reaches due to a reduction in transport capacity as stream gradients are reduced. These depositional reaches are also identified as response reaches.

Many variables other than sediment supply and transport capacity influence channel characteristics. Important geometric properties of stream channels include width, depth, and alignment. Hydraulic properties include slope, roughness characteristics, hydraulic radius, discharge, velocity, velocity distribution, turbulence, fluid properties, and uniformity of discharge. Other geomorphic factors include grain size of suspended sediment and bedload material, frequency of island occurrence, bar types and numbers, and especially the influence of debris flows and the occurrence of large woody material in forested mountain streams.

The Montgomery and Buffington classification system identifies eight distinct channel types (fig. 3–5). The bedrock channel type can occur in a number of positions on the stream profile, although it is more likely to occur on steeper slopes. The colluvial and bedrock stream reach types are normally associated with the headwater portion of a stream system, but they are quite different in morphologic characteristics. Source channels can be further divided into hillslope (flatter hill or mountain tops), hollow (transitional slopes) and colluvial (steeper sloped) channels. The further division of source channels primarily reflects the position of the channel in the headwater profile and has some implication on the relative amount of sediment load that can be anticipated from each type. The colluvial channels are normally the highest yielding source channel type in the watershed system because they contain significantly more sediment than the stream has capacity to transport. Sediment size (boulders, cobble) may be an important transport factor that may limit sediment loads from source channels in headwater streams.

Under certain circumstances, bedrock channels may also temporarily serve as source channels. Bedrock channels are often associated with headwater stream reaches, but they may also occur in the lower gradient portion of the watershed as well. With respect to sediment, bedrock channels are normally opposite of colluvial channels in that transport capacity significantly exceeds sediment supply. Simply stated, most of the available sediment has been removed down to bedrock. However, the sudden introduction of a sediment source such as a debris flow may temporarily cause a bedrock channel to take on the morphologic characteristics of a colluvial channel. The bedrock channel will ultimately return to its bedrock morphology once the temporary sediment source is removed. The time required to revert back to a bedrock morphology will depend primarily on the volume of the sediment obstruction and the particle size of the material to be transported.

Because the hydraulic capacity of bedrock channels normally exceeds available sediment supply, bedrock stream reaches are categorized as transport reaches. The remaining five channel classes are alluvial reach types. They include the cascade, step-pool, plane-bed, pool-riffle, and dune-ripple classes. Sediment in cascade channels is predominantly supply limited resulting in excess transport capacity. These channels occur on steep slopes that result in high rates of energy dissipation, and flows tend to be continuously in the supercritical range. Channel bed material will typically consist of boulders and cobbles since any finer material will have been mobilized and transported downstream. Much of the turbulent energy in cascade channels is dissipated in converging and diverging flows over and around large boulders and other trapped debris or obstructions.

Step-pool channels are also found in transport reaches, and they occur on steeper slopes, exhibit coarse bed material, and have low to moderate width to depth ratios. Step-pool channels, like cascade channels, are characterized as sediment supply limited with excess transport capacity. The primary distinction is that flow regime in step-pool channels is alternately supercritical in the steeper areas with subcritical flow and energy dissipation occurring in the pool areas. The bedrock, cascade, and step-pool stream reach types are all found in transport reaches.

The three remaining Montgomery and Buffington channel types (plane-bed, pool-riffle, and dune-ripple) are also alluvial channel types, but they fall into the response group. Plane-bed channels include channel reaches described as glides, riffles, and rapids. They typically occur on slopes intermediate between step-pool and pool-riffle channels. Plane-bed channels are usually described as armored bed surfaces. Streambed armoring indicates a lack of bedload transport capacity for larger particle or material sizes ,while finer suspended sediments have been readily transported through plane-bed reaches. Depending on sediment size distribution and discharge, plane-bed channels may exhibit either supply or transport limited morphologies.

Pool-riffle stream reaches are also response reaches. The bed of pool-riffle channels tends to be stable over time even though the bed material is mobilized by intermediate and larger flow events. Bars may develop in pool-riffle systems with high width to depth ratios and where the channel gradients are less than about 0.02 foot per foot. Like the plane-bed channels, sediment transport can be either supply limited or transport limited at various discharges. When sediment bars occur in pool-riffle systems, it is an indication that the composite flow regime is transport limited.

The dune-ripple channel is the third response channel type. A mobilized bed even at low flows characterizes the dune-ripple channel reach. They are typically low gradient, sand bed channels. Channel bed material can easily be put into suspension, but the combined sediment load is almost always greater than the available transport capacity. The bed material is constantly being shifted and moved short distances at all flows, but the overall lack of transport capacity compared to total sediment load results in the dune-riffle channel being transport limited.

Another key concept of the Montgomery and Buffington classification system is the recognition and categorization of a number of forced channel morphologies. A forced channel morphology can result from debris flows, geological barriers, bedrock outcrops, and especially from large woody material (LWM) in the Pacific Northwest. In small channels, trees tend to remain where they fall. Where the dominant trees tend to be longer than the channel is wide, woody material can create a sudden and long lasting constraint to the local stream morphology resulting in a forced stream type. On small mountain streams, LWM may dominate channel morphology by stream blockages that may exist for decades or even centuries.

In larger streams where the stream channel tends to be wider than the dominant tree heights, the LWM is typically mobilized and transported downstream. On these larger rivers, hydraulic processes dominate the impact of LWM on channel morphology. During large floods, LWM may be deposited on bar tops during the hydrograph recession, which may leave the impression that the LWM in the stream system has had little impact on channel morphology. Nevertheless, logjams influence channel pattern and flood plain processes in large forest channels through bank cutting or protection, channel unit and side development, and forcing channel avulsions (Bryant 1980; Nakamura and Swanson 1993; Abbe and Montgomery 1996).

LWM can be characterized as a random variable that creates many forced stream morphologies in Northwest streams. In addition to the impact of LWM on Pacific Northwest streams, there are a variety of other changes that can be anticipated. Montgomery and Buffington describe the array of potential channel changes as:

> *In response to changes in sediment supply or discharge, a channel may widen or deepen; change its slope through aggradation, degradation, or modified sinuosity; alter bedforms or particle size, thereby changing the frictional resistance of the bed; or alter the thickness of the active transport layer defined by the depth of channel scour. Drawing on both theory and empirical evidence, previous researchers developed conceptual models of channel response to changes in sediment load and discharge (Montgomery and Buffington 1998).*

Montgomery and Buffington have created a number of conceptual models of channel response supported by hydraulic geometry. Most of the documented experiences were associated with changes in sediment supply and/or transport capacity. Quantitative measurements of total sediment load including both bedload and suspended sediment are difficult and expensive to obtain. Hydraulic transport capacity is easier to obtain, but the accuracy and data format may not fit sediment modeling needs. For a more detailed description of the hydraulic geometry relationships and experiences with predicted changes and validated responses, see Montgomery and Buffington (1998).

Montgomery and Buffington acknowledge some merit to coarse scale classification systems for general planning purposes, but offer a cautionary note regarding the use of classification systems as a substitute for careful field evaluations of complex morphologic issues. Their cautionary note in its entirety is:

> *Channel classification cannot substitute for focused observation and clear thinking about channel processes. Channels are complex systems that need to be interpreted within their local and historical context. Classification simply provides one of a variety of tools that can be applied to particular problems—it is not a panacea. Classifications that highlight specific aspects of the linkages between channel networks and watershed processes are likely to be most useful, but careless application of any channel classification may prove misleading; no classification can substitute for an alert, intelligent, well-trained observer. Nonetheless, it is difficult to fully understand a channel reach without reference of the context defined by its bed morphology, confinement, position in the network, and disturbance history.*

Strengths

The Montgomery and Buffington stream classification system is a geomorphic process-based system that is strongly influenced by extensive experience on mountain streams, especially in the Pacific Northwest and Alaska. This classification system does an excellent job of identifying the morphologic differences in the mountain streams where it was developed. The process-based components of the system can be expected to work well in other mountainous regions, as well. The classification system aids the user in identifying source, transport and response (erosional, transport, and depositional) reaches. Regional variations with the classification system are more likely to occur with forced stream morphologies, especially those resulting from the presence of an abundance of LWM. There is clear reason to test the applicability of this classification system to other mountainous regions across the country, recognizing that the concept of forced stream morphologies may vary significantly.

Weaknesses

The nonfluvial geomorphologist initially may have difficulty applying the classification system with consistent results. The documentation in the past was developed within and written for the scientific community. As with many other systems, the procedure is not readily applied without study or training. However, with field experience, a practitioner should be able to define the nine stream classes identified by Montgomery and Buffington.

(e) Rosgen classification system

The Rosgen Classification of Natural Rivers was developed over 30 years of extensive fieldwork and observations of river systems across North America.

The Rosgen classification system tends to rely on field-measured parameters and is more experience-based than some of the other classification procedures described in this document. Rosgen's classification measures are based on channel dimensions measured at bankfull discharge, also known as channel forming flow. The complete Rosgen system is intended to provide both stream reach classification and guidance for potential restoration. The system includes the addition of a number of practical physical parameters that can be measured in the stream or from photographs and USGS topographic maps depending on the level of classification desired. Use of this method requires fundamental training and experience using this geomorphic method. Not only is a strong background in geomorphology, hydrology, and engineering required, but also an ability to implement the design in the field. The application of the classification system as part of a detailed design process is described in detail in NEH654.11.

The first version of Rosgen's current classification system was published in 1985. The system has contin-

ued to evolve with Rosgen (1994) and in Applied River Morphology (Rosgen 1996). The Rosgen system categorizes or classifies an individual stream reach, rather than an entire stream system. The key to the classification system is shown in fig. 3–6. Rosgen (1994) best describes the description of appropriate reach length as follows:

> *The morphological variables can and do change even in short distances along a river channel, due to such influences of change as geology and tributaries. Therefore, the morphological description level incorporates field measurements from selected reaches, so that the stream channel types used here apply only to individual reaches of channel. Data from individual reaches are not averaged over entire basins to describe stream systems. A category may apply to a reach (of) only a few tens of meters or may be applicable to a reach of several kilometers.*

Rosgen (1994, 1996) identifies four levels of detail in stream classification and assessment. This document primarily concentrates on levels I and II stream classifications. Each successive level provides a more detailed or finer definition of the dimension, pattern, and profile of the stream reach being classified.

Level I stream classification

Level 1 is a general characterization of the stream reach being classified. Level I stream classification is based on geomorphic features that can be interpreted from aerial photography, topographic maps, geologic maps, and a strong individual familiarity with the stream systems and land forms within the watershed of interest (Rosgen 1996).

Level I stream classifications are intended to be preliminary in nature. Level I classification makes use of readily available published information and relies on experience and judgment to the extent possible. The first four delineative criteria for levels I and II classifications are the same, but vary greatly in the intensity of required data. The four required channel characteristics for a level I determination are the number of channels, entrenchment ratio, width-to-depth ratio, and sinuosity. For a level I determination, the four channel characteristics often can be determined using a coarse scale with suitable landform maps.

As a minimum, level I classification requires a judgmental estimate of entrenchment (slight, moderate, or entrenched) based on prior knowledge of the stream system or experienced visual field observations. The specified ranges for width-to-depth ratio are fairly broad with break points at less than 12, 12 to 40, and greater than 40. In level I classification, the width-to-depth ratio is often viewed in terms of the stream reach being described as narrow and deep or flat and wide. With a minimum of experience, judgments of width-to-depth ratio with visual observations are relatively easy in all but borderline cases. The purpose of a level I classification is to designate the eight basic Rosgen stream types of A, B, C, D, DA, E, F, and G. These eight stream types are described in detail in Rosgen (1996).

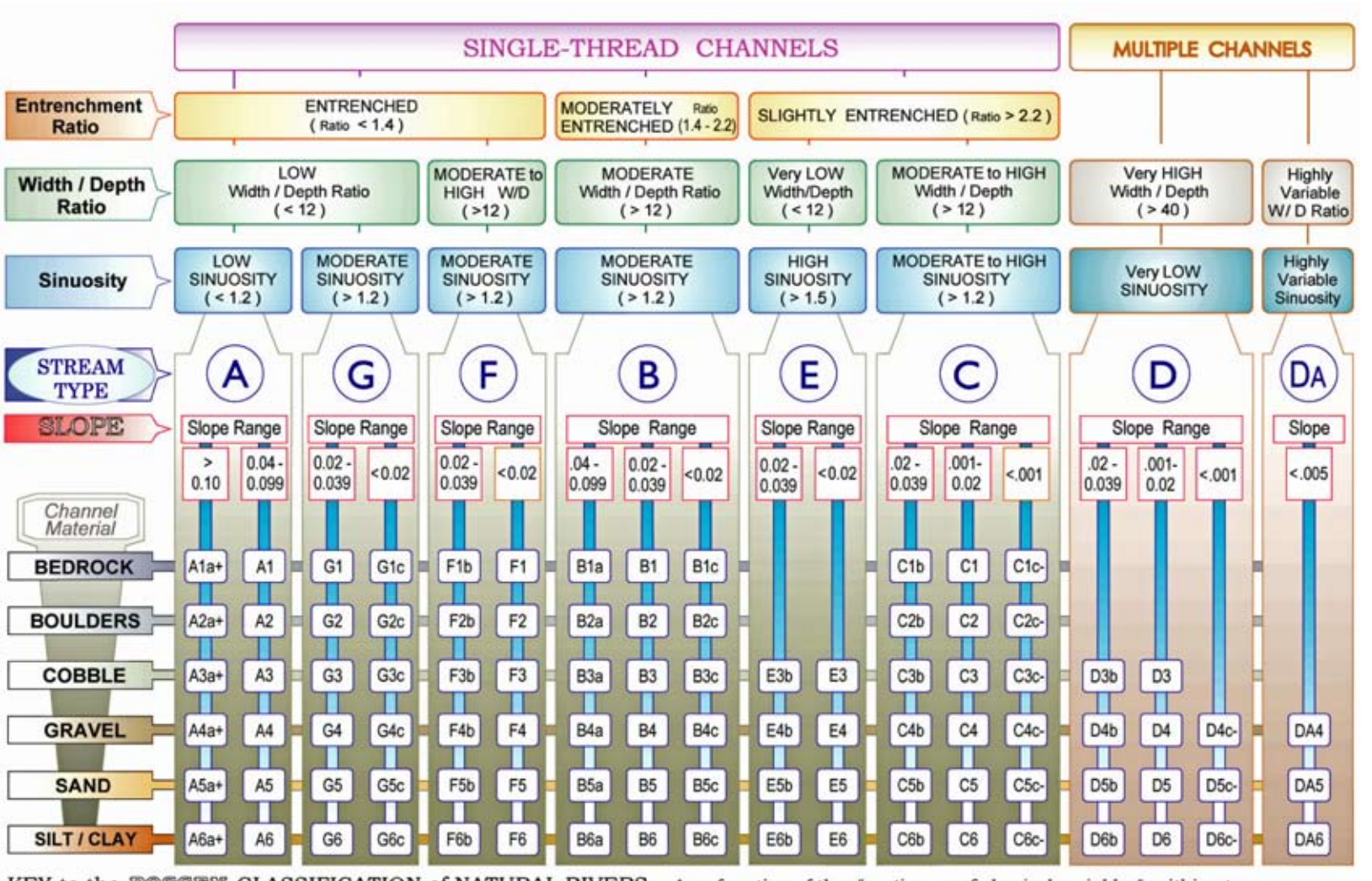
In practice, a level I classification can also include preliminary visual field estimates of bed material (a level II characteristic). An experienced practitioner can differentiate visually between a C channel with a sand bed (C5) and a C channel with a gravel bed (C4) in all but borderline cases. Channel water surface profile slope at bankfull stage (a level II characteristic) is not required to make a level I classification. However, a channel slope measurement from a USGS quadrangle map may be useful in preliminary planning to differentiate between channel types likely to occur on steep slopes versus channel types more likely to occur on flatter slopes. Estimates of channel slope are also useful in characterizing the general stream and valley system morphology.

Level I classification and any additional observations should be clearly identified as preliminary estimates that will have to be supported by actual field measurements in level II classification. Level I classifications can be useful for general discussion purposes, broad inventories, and coarse planning applications. Level I classifications are never suitable for use in the final design of stream restoration activities.

Level II stream classification

Level II stream classification requires actual field measurements and higher resolution landform mapping to delineate the more detailed and defensible stream classifications. The first four delineative criteria in level II classification are the same as were used for level I classification. The difference is that the number of channels, entrenchment ratio, width-to-depth ratio, and sinuosity must be accurately measured in the field for a level II classification.

Figure 3–6 Key to the Rosgen stream classification system



KEY to the *ROSGEN* CLASSIFICATION of NATURAL RIVERS. As a function of the *"continuum of physical variables"* within stream reaches, values of ***Entrenchment*** and ***Sinuosity*** ratios can vary by +/- 0.2 units; while values for ***Width / Depth*** ratios can vary by +/- 2.0 units.

Level II classification requires physical measurement of a number of associated parameters not required in level I including hydraulic characteristics. The hydraulic geometry portion of Rosgen's Classification of Natural Streams is strongly influenced by the early work of Leopold and Maddock (1953) and the work of Leopold, Wolman, and Miller (1964). This work identified eight interdependent hydraulic variables that could be used to characterize stream morphology. The variables are discharge, velocity, channel width, channel depth, channel slope, sediment size, sediment load, and roughness of channel materials.

Leopold, Wolman, and Miller (1964) recognized that a change in any one of these interdependent variables would produce resultant and often compensating changes in the other seven variables. The compensating effect is not uniform for all variables. For example, an increase in channel width will produce proportional, but inverse reduction in mean channel depth since, in many cases, bankfull channel area tends to remain relatively constant. For the same example, corresponding variables such as velocity and discharge may only exhibit minor reductions in magnitude. Rosgen has both directly and indirectly incorporated a number of the hydraulic geometry relationships into his criteria.

Two key field determinations are critical for obtaining accurate information for use in level II classifications. The first is that the elevation of the bankfull flow must be accurately determined, since it is directly linked to many other parameters. The term bankfull as used by Rosgen can be very confusing to the new practitioner who may visualize the common definition of bankfull as the elevation where water first begins to spill out of the channel banks and onto the flood plain. Rosgen uses the Dunne and Leopold (1978) definition of bankfull:

> *The bankfull stage corresponds to the discharge at which channel maintenance is the most effective; that is, the discharge at which moving sediment, forming or removing bars, forming or changing bends and meanders, and generally doing work that results in the average morphologic characteristics of the channels.*

The bankfull discharge and resultant elevation has a typical recurrence interval range of 1.0 to 3.0 years on an annualized frequency curve with a predominance of values occurring in the 1.2- to 1.8-year range. For channel types C, D, DA, and E which are only slightly entrenched, the lay definition of bankfull and the Dunne and Leopold definition are very similar. For the B channel type which is moderately entrenched or the A, F, and G channel types that are entrenched, the Rosgen bankfull is at an elevation well below the top of the banks. A number of good field indicators can be used as reliable indicators such as the top of point bars, a break in bank slope, and the presence of certain riparian vegetative species, which vary by region. An accumulation of indicators aids the practitioner in physically identifying the Dunne and Leopold bankfull elevation in the field. With proper training and concerted practice, individual determinations of bankfull tend to be consistent. Bankfull determinations are not necessary for the general level I classifications, but are a key element for the detailed level II determinations. For a more complete description of bankfull discharge, refer to NEH654.05.

The second important concept in determining level II classifications is entrenchment ratio. Entrenchment or channel incision is basic to geomorphic and geologic literature. Rosgen has established a useful working definition that helps define the relative degree of entrenchment. Rosgen defines entrenchment ratio as the width of the flood-prone area at an elevation twice the maximum bankfull depth, divided by the bankfull width (flood-prone width/bankfull width). Based on Rosgen's database, a total depth equal to twice the maximum bankfull depth constituted a major flood with an approximate recurrence interval of 50 years. Some stream professionals question the validity of this hypothesis. Rosgen defines the total width at two times the maximum bankfull depth as the flood-prone width. Regions outside of the area covered by the database may vary significantly from the flood-prone width/bankfull width relationship established by Rosgen. The procedures for making the necessary field measurements are listed in Rosgen (1996). This reference emphasizes the importance of an accurate determination of the bankfull elevation, since entrenchment ratio and several other parameters are directly related to the bankfull elevation. Important issues and concerns regarding the identification of bankfull indices is addressed in NEH654.05.

The concept of entrenchment ratio is an empirical relationship. Although Rosgen's database includes information from locations across the United States

and Canada, some concern remains that this relationship needs to be evaluated on a national basis and that some regional modifications may be appropriate. The current version of Rosgen's Classification of Natural Rivers is presented in hierarchal form in the book, Applied River Morphology. The determinations of six hierarchal parameters are required to make a complete level II classification of a stream reach.

1. Number of channels—On the surface, this appears to be a simple determination that could be made from field observation or the use of current photographs and maps. However, by definition, there must be three active channels at the bankfull elevation to be considered a multiple-thread channel. Therefore, a bankfull determination is required to verify that there are actually three or more active channels at the bankfull stage. Multiple active channels (three or more), where they are verified to exist, identify the stream reach as either a D or DA classification. All other channels are considered to be single-thread channels.

2. Entrenchment ratio is defined as the width at an elevation twice the maximum bankfull depth divided by the bankfull width. The importance of an accurate determination of the bankfull elevation as it applies to this and other parameters has been described previously. Concerns have been expressed that regional variations in this parameter may be required. Geology, slope, vegetation, and other factors may also influence this parameter.

3. Width-to-depth ratio is defined as the width measured at the bankfull elevation divided by the mean depth of the bankfull channel. The magnitude of the parameter depends on an accurate determination of the bankfull elevation.

4. Sinuosity is defined as the ratio of stream length at the bankfull stage to valley length. Sinuosity can best be measured in the field, but it is a time-consuming measurement. Streams with smaller channels and with extensive canopy cover will likely require field measurements to obtain the needed accuracy. For larger streams and streams with limited canopy cover, sinuosity can also be successfully measured using alternative sources such as recent aerial photography with sufficient resolution. A scale of 1/660 (8 in/mi) is preferred; however, a scale of 1/1,320 (4 in/mi) usually gives acceptable precision for larger open-canopy stream channels. Sinuosity can be measured off USGS 7 1/2-minute quadrangle sheets, but this is not suitable for a level II classification. The scale of the 7 1/2-minute quadrangle sheets (2.64 in/mi or 1 in = 2,000 ft), the age of the quadrangle photo base, and the limited detail used in defining the stream channel on the quadrangle are all concerns that limit the utility of using USGS quadrangle sheets for determining level II sinuosity.

   At this point, a level II basic classification, A through E, of the stream reach can be obtained. The difference between the levels I and II classification is that the criteria have been validated with actual field measurements. In actual practice, level II classification is rarely terminated at this point. The remaining delineation criteria for a complete level II classification are:

5. Channel material is a determination of the surface particles that make up both the bed and bank material within the bankfull channel. The Rosgen classification procedure uses a modified version of the Wolman (1954) pebble count procedure for the determination of surface particle sizes. A number of cross sections selected to represent the distribution of pools and riffles within the reach to be classified are sampled using the pebble count procedure. Although the parameter being defined is channel bed material, each cross section is surveyed using equally spaced stations up to the bankfull elevation. Since each data point is counted equally in the process, the procedure is normally heavily weighted toward channel bed material, especially on wide shallow channels. For specific details on making a modified Wolman pebble count, refer to Rosgen (1996). Although exceptions are noted for bimodal particle size distributions, generally the D_{50} particle size determined from the modified Wolman pebble count procedure is used to classify the channel bed and bank materials up to the bankfull elevation. Rosgen's first channel material class based on a field determination is bedrock. The five remaining material classes are based on the D_{50} particle size of the streambed and bank material up to the bankfull stage as determined

from pebble count information. The six Rosgen material classes, including bedrock are:

- bedrock
- boulder—greater than 256 millimeters (10 in)
- cobble—64 to 256 millimeters (2.5 to 10 in)
- gravel—2 to 64 millimeters (0.08 to 2.5 in)
- sand—0.062 to 2.0 millimeters
- silt/clay—less than 0.062 millimeters

The channel material makes up the left-hand side of the Rosgen classification matrix. Pebble counts are more appropriate for boulder, cobble, and gravel bedded streams. Other protocols may need to be developed for sampling fine-grained bed and bank material (sand, silt, and clay).

Classification of sediment into particle-size classes is arbitrary, with class breaks frequently based upon standard sieve sizes. It should be pointed out that the class size breaks and most of the descriptive terms used by Rosgen were derived from the Udden-Wentworth classification system used by geologists (Wentworth 1922). This system employs different size breaks and some differing terminology from the particle size classification systems used by NRCS engineers (Unified Soil Classification System, American Society for Testing and Materials International (ASTM) D2487) or NRCS soil scientists (USDA soil texture classification system).

6. Slope is the local slope of the bankfull water surface within the reach that is being classified. Water surface slope is typically measured over a length equivalent to 20 bankfull channel widths or a minimum of two meander wavelengths. For applications in level II or higher, measurements of the actual water surface on both pool and riffle sections is also a requirement. Ephemeral streams may require the use of computed water surface profiles with sufficient cross section data to define the pool-riffle sequence.

The field determination of the bed material as defined provides the criteria to make a complete level II determination such as A3, which is an A channel with cobble bed material. If the slope of the local water surface profile is outside of the normal slope range for an A3 channel (0.040–0.099), the channel can be further described based on a slope subscript. An example would be an A3a+ which describes an A channel with cobble bed material on a slope greater than 10 percent. Some channel types such as B and C channels may have slope variations that are greater than normal (+) or less than normal (–).

Level III and IV assessments

Levels I and II are the levels of classification that characterize and describe stream types. Although detailed descriptions of levels III assessment and IV validation are not included here, it is useful to understand their scope. Levels I and II are a classification of the current status of the stream reach based on two distinct levels of data acquisition. Level III assessment is used to evaluate stream condition and its departure from the optimum or potential condition. Level III data are necessary to quantify numerous parameters (sediment load, bedload, bank erosion) that more clearly define trends and expected long-term changes in the current stream status. Level III data are critical as a basis for restoration designs and installation. Level IV is the validation level where the parameters of stream function are monitored over time to either validate a stream's status or the success of a restoration activity.

Management interpretations

Rosgen (1996) provides examples of how stream classification can be related to numerous NRCS activities. Stream type can be related to expected impacts due to disturbance, recovery potential, sediment supply, streambank erosion potential, and the potential for vegetation to control the dominant channel influence. Rosgen's database may not be completely representative of all regions of the country, and all final decisions should be supported by a field assessment. This method requires field data that represent local stream morphology.

Strengths

The Rosgen classification system is currently the most widely used of the four systems addressed in this document. While initially applied regionally, the method has been used nationally and internationally. Levels I and II stream classifications have found acceptance among a variety of disciplines. The greatest value of

Rosgen's classification system is in the establishment of a common language for communication among the associated stream disciplines. For example, a geologist in Alaska can talk to a biologist in Florida regarding a (C5) stream type and both will have a common frame of reference.

Rosgen's procedures go beyond levels I and II stream classifications. They are linked to design procedures and criteria for a wide range of stream restoration activities. Rosgen's levels III and IV procedures appeal to disciplines that do not have professional backgrounds in stream mechanics and geomorphology, but have a strong desire to do a better job in stream restoration activities. This may be considered as a strength or weakness depending on the competence of the individual(s) using the procedures.

Weaknesses

Combining silt/clay as a similar channel material is inconsistent with the general erosion, stability, and structural integrity characteristics of the two materials. Additional data on silt and clay channels from across the country may resolve this issue. The type B stream classification has been criticized as a catch all category. It represents the only stream type between slightly entrenched and entrenched. It also covers the largest slope range of any of the stream types with a slope range from <0.020 to 0.099. Additional data may warrant additional breakdown of the B stream type.

Other weaknesses are the lack of an upper limit on the width-to-depth ratio for C stream types, and the requirement of three active channels at bankfull for the classification of D stream types.

Levels I and II classifications describe the current condition of the stream reach, but does not address time variability issues such as the rate of entrenchment or rate of lateral migration. A number of time variability issues such as rates of aggradation or degradation and lateral migration are addressed in level III assessment and level IV validation.

Beyond levels I and II stream classifications, there is an often-expressed concern among geomorphologists and stream mechanics engineers that the overly enthusiastic novice in stream restoration may attempt projects that are beyond their technical capabilities. There may be a tendency by well-meaning users to overlook the need for interdisciplinary input. Another concern is that a stepwise procedure may mask the ability of an inexperienced user to fully understand the interrelationship between the watershed and channel processes.

Although Rosgen (1994, 1996) empirically derived boundary values for geomorphic predictors in his classification system, users should be aware that local calibration is very important to determine a tighter range of values more applicable to a given geographic area. Local calibration is even more important if a significant number of projects using the system are planned, or many streams in a project fall on the cusp of classification boundaries. Further, local calibration may highlight elements of the Rosgen classification system as better, more geomorphically significant descriptors of local stream systems than others.

Detailed information concerning the application of the Rosgen stream classification technique is provided in NEH654 TS3E. Design guidance is included in NEH654.11.

654.0306 Conclusion

This chapter briefly summarized procedures for watershed assessments and site investigations. Stream system inventory and assessment techniques were identified and compared. Information was provided on stream stability, as well as geological and biological assessments. The uses, advantages, and disadvantages of various geomorphic stream classification systems were also described. This chapter addressed fluvial processes and broader geologic issues related to ecological function, as well as stream design and behavior.

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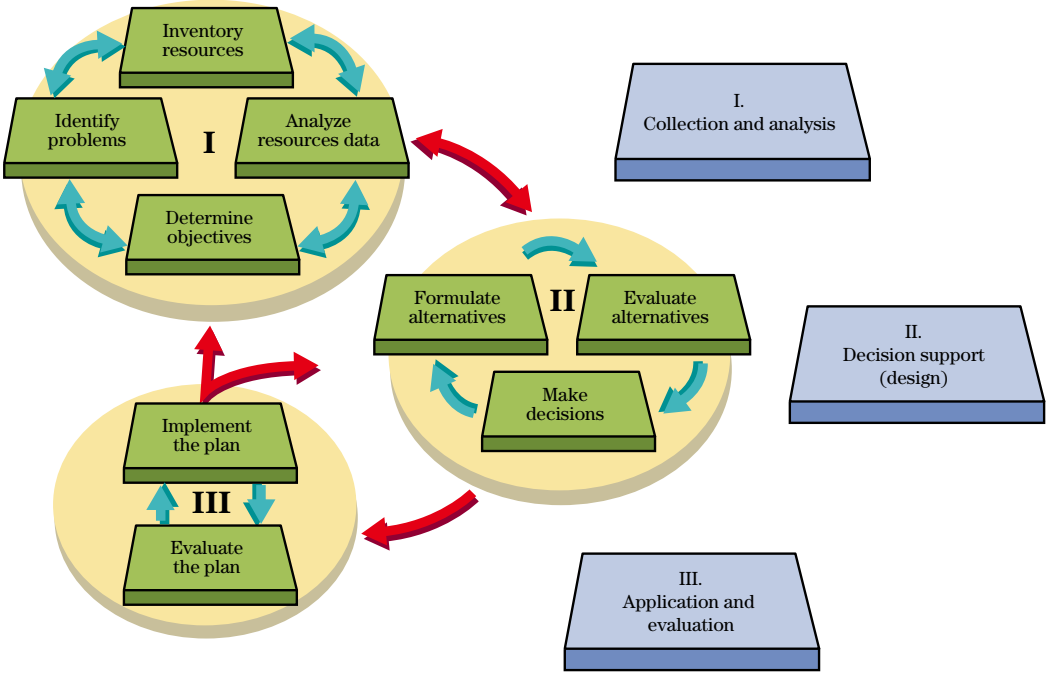
**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 4 Stream Restoration Design Process



Issued August 2007

Cover illustration: Design of solutions for stream problems is a part of the restoration planning process. Designs translate the desired changes into the stream and riparian zone. Changes to the design may cause goals and objectives to be reevaluated, as the planning process may be iterative.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 4 Stream Restoration Design Process

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Chapter 4 Stream Restoration Design Process

654.0401 Purpose

The purpose of this chapter is to provide an overview of the process for designing stream restoration solutions. The design process is integrated with the overall planning process. To design a solution means to fit it into the landscape, into the stream system, so that the result meets the goals and objectives of the plan. Solutions may range from no action to management and simple removal of perturbations, to site-specific practices, to riparian corridor and watershed-scale restoration systems.

Often, solutions to stream problems (fig. 4–1) are suggested at the time that they are identified, such as: "My streambank is eroding. We need to put rock riprap on it." It could be that the problem merits that response. It could also be that the nature of the bank erosion problem is more complex and may be related to a general instability of the stream system. An interdisciplinary, onsite analysis is critical to the development and success of any designed solution or system.

The design of a solution to a stream problem must address the goals and objectives developed from the planning process. Once a solution is agreed upon, the design process determines the feasibility of the solution and whether goals or objectives must be revised or whether a different designed solution should be pursued. A cookbook design procedure is not recommended since each project and each design will have differing goals and objectives, physical or biological constraints, and jurisdictional requirements and constraints.

This chapter provides an integration of the conservation planning process (CPP) with stream restoration design concepts and provides the foundation for using the tools and procedures in the following chapters. Note that design of stream restoration solutions may include a wide range of design elements, from management practices to structural practices, the selection of which depends on the nature of each individual project.

654.0402 Introduction

Planning actions to fix stream problems can be a complex process. This is due to the interactions between possibly many stakeholders: people who affect or are affected by the stream problems and any potential solutions. How streams are supposed to look and function are ideals that vary from one person to another. Philosophies and approaches to stream restoration abound: restore to what conditions or functions?

This chapter overviews the process for developing designs to solve stream problems. There are many steps in the overall process (NEH654.02). Some steps may be accomplished quickly, while others may require lengthy analysis, data gathering (NEH654.03), or discussion with stakeholders, depending on the magnitude and complexity of the problems, as well as constraints posed by boundary conditions, funding, attitudes, or local requirements (fig. 4–2). Problems that are localized and involve only a single land user may be planned and designed rapidly. Designs must also address environmental and ecological factors, as well as satisfy the immediate stream restoration need. Streams in urban areas present unique challenges for restoration (fig. 4–3).

Figure 4–1 Severe bank erosion along the Connecticut River eroded cornfield and resulted in excessive sediment in the river

Appropriate and effective stream solutions can only be designed when the goals and objectives of the planned solutions are clear, realistic, and adequately formulated. NEH654.02 focuses on the importance of identifying the goals and objectives of any proposed stream action that will drive the design approaches. It also expands on the concept of risk associated with stream solutions—the risk of failure of the implemented design elements, the risk of creating ecological imbalances, as well as the risk of not achieving the intended results.

The importance of collecting the right information to assess the nature of the temporal, physical, and biological nature of the problem are addressed in NEH654.03. The information collected will also facilitate the design process and form the basis for making assessments of the overall success of the project after implementation.

This chapter introduces an overall design procedure, which is an integral part of the planning process for stream design. The purposes of stream designs can range from simply conveying water to restoring self-sustaining ecological functions and values to the stream corridor. The design process may be iterative, in that the initial design may require cycling back through some of the planning steps, making decisions, possibly modifying goals and objectives, and redesigning alternatives. Stream designs may include a variety of solutions ranging from upland watershed and riparian area management practices that may be needed, large-scale reconstructions of entire stream reaches, localized applications that can involve earth materials, live and inert plant materials, and manufactured materials.

Figure 4–2 How streams are supposed to look and function are ideals that vary from one person to another.

Figure 4–3 Lined channels may be necessary, based on boundary constraints, drainage needs (discharge capacity), and maintenance costs.

654.0403 The CPP for stream design

The design and management of streams must address the myriad of resource concerns, as well as client objectives, to support near- and instream stabilization activities in a sustained manner. In general, prescribing the treatment for a stream is based on the site or reach conditions including historic and reference stream corridor information and objectives of the decision-making client, as well as stakeholders who influence the client. This important issue was first addressed in NEH654.02.

Stream restoration is defined here as one or more conservation practices used to overcome resource impairments and accomplished the identified purposes based on client objectives for a conservation management unit (CMU) containing, in whole or part, the stream corridor needing treatment.

A stream corridor includes the stream and extends in cross section from the channel's bankfull level towards the upland (perpendicular to the direction of streamflow) to a point on the landscape where channel-related surface and/or soil moisture no longer influence the plant community. Figure 4–4 illustrates an idealized cross section of a stream corridor (modified from Stanford and Ward 1992).

This description encompasses moisture influenced land on both sides of a channel. The length of a stream corridor is typically a sinuous band that follows both sides of the channel from the headwaters to the mouth of the stream system. Depending on channel conditions (stream order, channel evolution model (CEM) stage, bankfull width, degree of incision, and flood plain characteristics), the width of this longitudinal band fluctuates with corresponding influences on the kind and composition of riparian vegetation within the band. In mountainous areas, changing elevations along the stream corridor determine riparian community composition because of the varying cold-hardiness capacity of individual plants. Also in effect are cross-sectional variations in microclimate and soils, which influence the kind and mix of riparian species. Stream corridor soils are typically not a single soil series, but a complex of named soil series and taxadjuncts. Taxadjuncts are soils closely associated with a named series that differ somewhat in one or more soil characteristics, which may further complicate the planning process.

Table 4–1 lists and figure 4–5 illustrates the steps in the Natural Resources Conservation Service (NRCS) CPP. This was described in NEH654.02. Detailed information about each of these steps is provided in the NRCS National Planning Procedures Handbook (NPPH) (USDA NRCS 2003b). These steps are applied for each CMU on a client's planning unit. An important aspect of the planning process is how the proposed stream restoration practice(s) will interact and work compatibly with other practices in the resource management system (RMS) to address all pertinent resource concerns in achieving resource quality criteria (refer to Section III of the local NRCS Field Office Technical Guide (FOTG), available for each county at *http://www.nrcs.usda.gov/Technical/efotg/*).

Figure 4–4 Cross-sectional view of a generalized stream corridor segment. Biota may reside in all dimensions (riparian, inchannel, hyporheic and/or ground water zone). Inundation and desiccation of the blue shaded area occurs as the amplitude of the discharge increases and decreases under a natural flow regime. Sd designates sediment deposition sites, and Se is a site of bank erosion. The solid line is the thalweg, and the broken lines indicate the different directions of flow and materials among inchannel, hyporheic, and ground water zones.

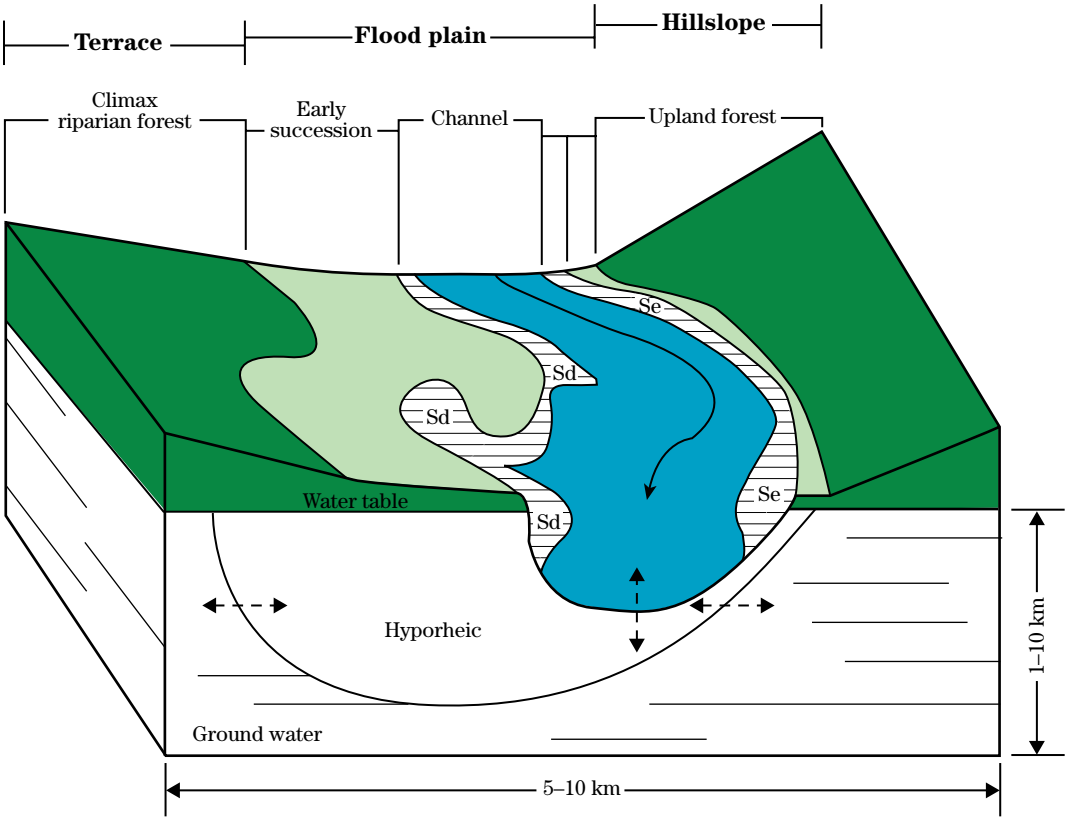
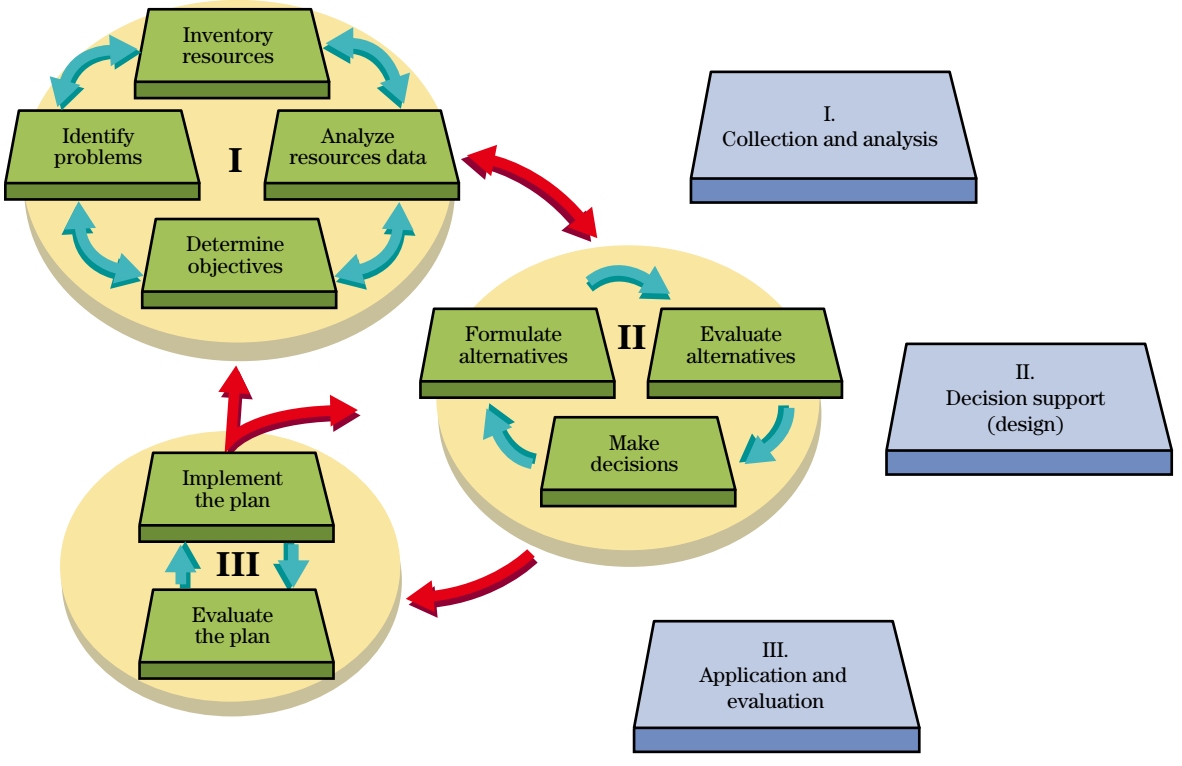


Table 4–1 Steps in the NRCS CPP

Steps	Planning activity	Level	Potential iterations
1	Identify problems and opportunities	I	
2	Determine objectives		
3	Inventory resources		
4	Analyze resource data		
5	Formulate alternatives	II	
6	Evaluate alternatives		
7	Make decisions		
8	Implement the plan	III	
9	Evaluate the plan		

Figure 4–5 NRCS CPP showing the dynamic interaction between the steps



654.0404 Designing solutions for the stream corridor

While an array of suitable practices is available for most stream corridor conditions, client objectives usually focus treatments to a more limited range. Planning site problems, however, typically exceed the client's focus and, more than likely, are symptomatic of larger area and watershed concerns (habitat fragmentation or an imbalance in sediment transport).

Table 4–2 describes the interplay of stream restoration with three common client problem/objective scenarios. While all objectives are justifiable management options, multiple resource concerns and ecological functions are usually only addressed with example objective C.

(a) Landscape context for restoration

Once site problems and client objectives have been evaluated (NPPH planning steps 1 through 4), potential treatment and restoration activities can be considered. From a context viewpoint, an important first step is to recognize the site-level landscape settings or zones on the CMU/stream corridor that influence the selection of potential practices. Figure 4–6 (adapted from Hoag et al. 2001) illustrates an idealized, conceptual cross section of a stream and one side of the stream's riparian area. Not all of these zones will exist in all streams and rivers. Table 4–3 provides more detail on applicable landscape zones, descriptions of each zone, and an overview of correlated practices.

Because of the strong physical and ecological interaction of streams with their flood plain and adjacent corridors, a CMU should be delineated to encompass the stream corridor, which includes the streambed, banks, and riparian areas. These landscape components strongly interact and are best planned as a whole to optimize desired effects and meet client and ecological objectives.

Tables 4–4, 4–5, and 4–6 provide information on selection of NRCS Conservation Practice Standards and their considerations and effects related to landscape zones in and along the stream. Table 4–4 focuses on accelerated erosion, sediment, and site instability, whereas tables 4–5 and 4–6 focus on habitat and biodiversity and production and land use, respectively. Each zone depicted in these tables has particular characteristics and correlated practices. Practices can be reviewed and studied by referring to section IV of the FOTG. From a biotic perspective, the plant community potential is an important ecological reference in thinking about restoration and triggering site-level planning questions:

- Is the reference plant community (or a successional stage of it) present?
- Are the site's landscape zones in a physical condition to mutually sustain the reference community or stages?
- Are watershed-level landscapes in a physical and biotic condition to sustain site-level stream corridor recovery or restoration?

(b) Selecting conservation practices for stream restoration

With an understanding of the planning process and stream corridor landscape settings, the planner is ready to match site impairments, landscape zones, and client objectives with conservation practices (idealized in fig. 4–7). Tables 4–4, 4–5, and 4–6 provide information for use by the planner in choosing appropriate treatments and vegetation types. An important strategy when using the table is to match existing problems in each impairment category with coinciding conservation practices suited to the particular landscape zone or zones making up the CMU. An assumption is that once impairments are recognized, the client's objective is to remedy problems by using a system of conservation practices (fig. 4–8).

Selection of some practices at the beginning of tables 4–4, 4–5, and 4–6 will influence or curtail the selection and extent of others listed later. For example, a client wants to improve forage resources in the overbank and transitional zone, but has eroding banks and overbank zones lacking protective ground cover. Treatment of the bank and overbank zones (using rock, mulching, and/or specialized vegetation) to control bank or surface erosion will necessarily restrict the use and extent of forage establishment practices. In this situation, the bank and overbank zones may require livestock exclusion temporarily or permanently, with a corresponding revision of the site's prescribed grazing management.

Table 4–2 Relationship of stream restoration components with client problems and objectives

Example problems/client objectives	Channel/riparian/watershed characteristics	Desired outcome/effects
A. Erosion and sediment control (streambank erosion, channel aggradation, channel degradation, concentrated flow and scour erosion, sheet and rill erosion)	• Excessive bank recession rates • Instream bar formation • Incised channels that are deepening, then widening • Lack of vegetative cover on banks, flood-prone zones and riparian areas, allowing concentrated flow, sheet, rill, and scour erosion • Concentrated-flow gullies from adjacent areas and land uses • Overall watershed cover has less native perennial cover, more impervious areas or more direct flow paths, which are unbuffered	• Return to normal reference bank recession rates and point bar dynamics • Incised channels are stabilized and flood-prone areas are reestablished. This occurs at a lower elevation than preincisement conditions • Aggressive herbaceous plants substantially reduce surface erosion and hinder the invasion of weeds, but they can impede successional progression to the desirable plant community • Woody plants bind streambank soils and in adjacent flood-prone areas, increase surface roughness, which can reduce scour erosion • Buffers and associated practices in adjacent uplands can slow runoff, reducing stress to streambanks and channel degradation processes
B. Production and use of stream and streamside vegetation (game fish, livestock forage, forest products)	• Channel banks and bed are modified and maintained to favor specific game fish • Streamside herbaceous plants, woody plants or a combination consistent with the client's operation and marketing capability are grown to satisfy economic requirements	• Production and utilization goals are achieved when fish and vegetation products reach desired biomass, size, or quality • Aquatic and plant community succession is retarded/managed (or completely supplanted by a production community) to maintain the desired operational condition
C. Restoration of ecological functions (creation of a successional stage which can be maintained or allowed to succeed to a desired plant community)	• Herbaceous plants, woody plants or a combination consistent with desired successional stage or progression to the reference reach plant community	• Functions such as site-soil stability, vertical and spatial habitat, and nutrient cycling are achieved when vegetation reaches the desired successional condition • Domestic use for recreation, grazing, timber harvesting, or other exploitation is excluded or sufficiently restricted so that the desired successional stage is reached and maintained

Figure 4–6 Conceptual cross section of a riparian area with landscape zones for planning restorations

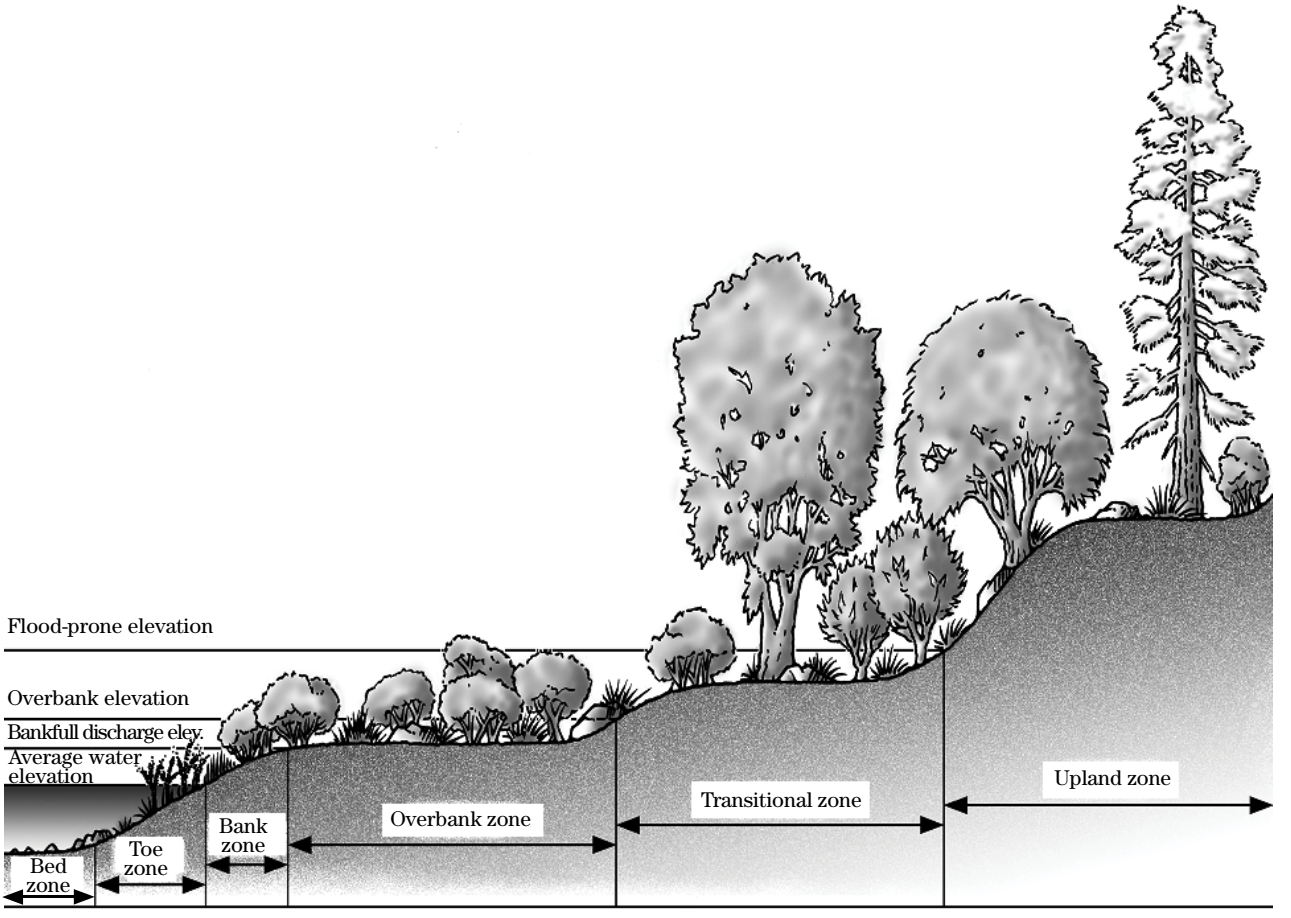


Table 4–3 Description of riparian landscape zones for stream corridor design and management

Definitions and descriptions of landscape zones	Potential RMS conservation practices*
Bed zone—The bottom of the channel that can consist of a variety of bed materials. Vegetation may consist of periphyton (diatoms, algae, phytoflagellates attached to substrate material), phytoplankton (suspended in the water column), and macrophytes (vascular and nonvascular plants), depending on bed material, pool, riffle, run proportions, and flow rate	Channel Stabilization (584)*, Clearing and Snagging (326), Fish Passage (396), Open Channel (582), Stream Crossing (578), Stream Habitat Improvement and Management (395)
Toe zone—The portion of the bank that is between the average water level and the upper edge of the bottom of the channel. This zone has the highest stress because of frequent exposure to wave wash, channel-forming currents, ice and debris movement, and wet-dry and freeze-thaw cycles. Vegetation is generally herbaceous emergent aquatic species, tolerant of long periods of inundation	Channel Stabilization (584), Clearing and Snagging (326), Fish Passage (396), Streambank and Shoreline Protection (580), Stream Crossing (578), Stream Habitat Improvement and Management (395)
Bank zone—The area above the toe zone located between the average water level and the bankfull discharge elevation. (The bankfull discharge elevation, in natural streams, is the elevation at which water fills the channel without overflowing onto the flood plain.) This zone is exposed periodically to wave wash, erosive river currents, ice and debris movement, and traffic by animals or humans. Vegetation may be herbaceous or woody and is typically characterized by flexible stems and rhizomatous root systems. Plants are periodically submerged	Channel Stabilization (584), Channel Bank Vegetation (322), Streambank and Shoreline Protection (580), Stream Crossing (578), Stream Habitat Improvement and Management (395), Use Exclusion (472)
Overbank zone—The area located above the bankfull discharge elevation continuing upslope to an elevation equal to two-thirds of the flood-prone depth. Vegetation may consist of some proportion of herbaceous plants, shrubs and trees, depending on the plant community potential of the site	Critical Area Planting (342), Early Successional Habitat Development/ Management (647), Fence (382), Filter Strip (393), Forest Stand Improvement (666), Irrigation System (441/442/443), Mulching (484), Nutrient Management (590), Pasture and Hay Planting (512), Pest Management (595), Prescribed Grazing (528), Range Planting (550), Recreation Area Improvement (562), Restoration and Management of Declining Habitats (643), Riparian Forest Buffer (391), Riparian Herbaceous Cover (390), Stream Crossing (578), Use Exclusion (472), Watering Facility (614), Wetland Wildlife Habitat Management (644)
Transitional zone—One or more levels of terraces located between the overbank zone and the flood-prone width elevation. On forest potential sites, vegetation is usually larger shrub and tree species with a shrub/ herbaceous understory. On herbaceous potential sites, a combination of overbank and upland herbaceous vegetation is usually present in some proportion, as well as other herbaceous species intolerant of upland (dryer) or overbank conditions (wetter)	
Upland zone—The area above the transitional zone. This area is seldom influenced by stream/riparian soil moisture	Various practices, section IV of the local FOTG consisting of 150+ practices

*NRCS National Conservation Practice Standard codes, Specific information for these practice standards is available at the following Web site: *http://www.nrcs.usda.gov/technical/Standards/nhcp.html*

Table 4–4 NRCS Conservation Practice Standards selection for accelerated erosion, sediment, and site instability. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system.

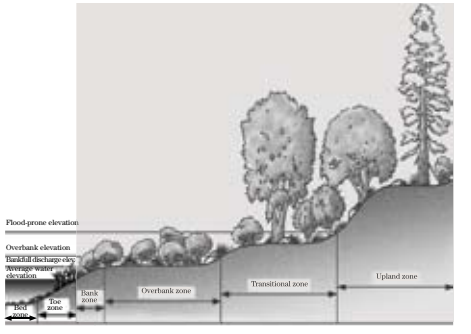
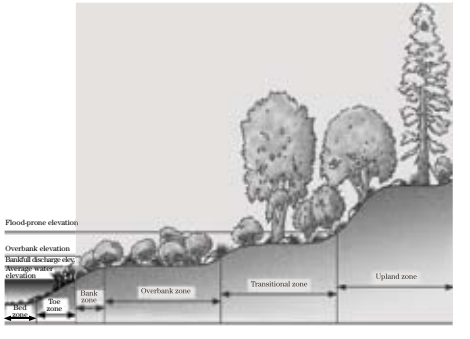
Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: accelerated erosion, sediment, and site instability	Landscape zones
Unbalanced channel sediment transport and deposition; unstable channel bed and/or gradient[2/]	Bed, toe	Open Channel (582)	Various techniques including channel meander reconstruction at a site will reconfigure the bed and bank topography and influence the extent of overbank and transitional zones and related soil moisture and the selection of vegetation species	
		Channel Stabilization (584)	Measures to support balance and stability will reduce risk of bank recession and damage to overbank zone vegetation	
		Clearing and Snagging (326)	Where practical, restore native vegetation to all bank, overbank and transitional areas disturbed by use, ingress, or egress of obstruction removal equipment	
Accelerated bank erosion and instability[2/]	Bank, toe	Channel Bank Vegetation (322)	In the overbank zone nearest the stream, use the same or similar riparian area plant species as channel bank vegetation to provide additional support to controlling bank erosion	
		Streambank and Shoreline Protection (580)	See notes for Channel Bank Vegetation (322), which is the vegetation component of this practice	
		Clearing and Snagging (326)	Restore vegetation (native species where practical) to all bank, overbank and transitional areas disturbed by use, ingress and egress of obstruction removal equipment	

Table 4–4 NRCS Conservation Practice Standards selection for accelerated erosion, sediment, and site instability. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system—Continued

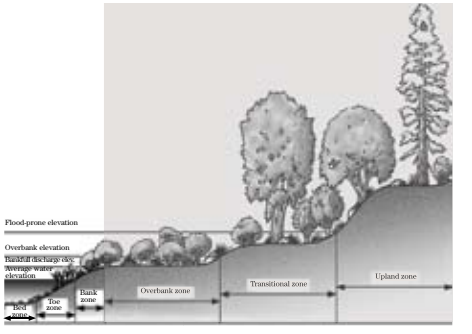
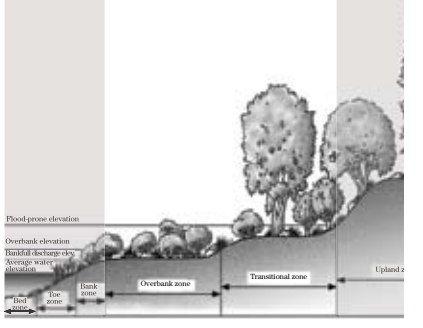
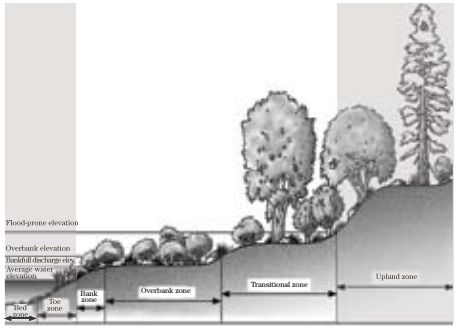
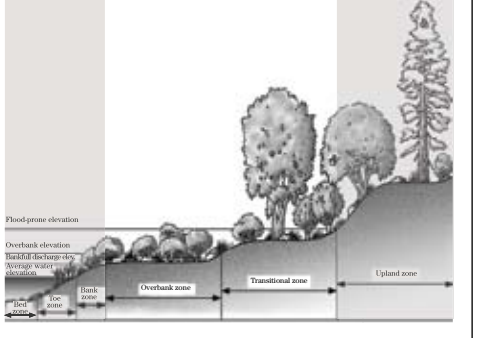
Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: accelerated erosion, sediment, and site instability	Landscape zones
Excessive damage by animals, people or vehicles (soil compaction and rutting, loss of protective ground cover) and associated liability and health concerns	Bed, toe, bank,	Use Exclusion (472) Fence (382)	Use Exclusion by use of a fence or other means may be sufficient in restoring the desired vigor and density of the site's vegetation to mitigate damage. Use Exclusion is also used to protect new plantings and accelerate their establishment period	
Accelerated or potential high-rate surface erosion from sheet, rill, ephemeral, or flood scour erosion processes	Overbank and transitional	Critical Area Planting (342)	Introduced plant species and cultivars are usually chosen over native plant species because of improved vigor or establishment density. Flood scour may require additional, shrubby plantings of sufficient height and width (perpendicular to flow) placed strategically to slow out-of-bank flows	
		Mulching (484)	Mulch materials can accelerate establishment of riparian area erosion control plantings by suppressing weed growth, moderating soil temperature, and conserving soil moisture. Seeding can be incorporated into and applied concurrently with some mulching techniques (hydroseeding)	

Table 4–4 NRCS Conservation Practice Standards selection for accelerated erosion, sediment, and site instability. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system—Continued

Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: accelerated erosion, sediment, and site instability	Landscape zones
Overgrazing	Overbank and transitional	Prescribed Grazing (528)	Prescribed grazing controls the timing, duration and intensity of domestic animals, while maintaining some use of existing forage. Based on the degree of damage to riparian and bank vegetation, use exclusion and livestock deferment may be needed for several years before grazing can resume	
Excessive sediment and/or other pollutants in runoff reaching the channel	Overbank and transitional	Filter Strip (393)	Introduced herbaceous species filter sediment in runoff reaching and passing through the strip. In areas with forest potential, filter strips are used as zone 3 of a riparian forest buffer to filter and slow upland runoff	
		Riparian Forest Buffer (391)	Tree and shrub species further slow upland runoff and aid in the infiltration of pollutant-laden water. Uptake and microbial processes break down nitrates and pesticides. Riparian forest buffers are not intended to withstand unabated upland runoff. Native woody species may not be of sufficient vigor or establish quickly enough for some pollutant loadings	

1/ NRCS National Conservation Practice Standard codes, *http://www.nrcs.usda.gov/technical/Standards/nhcp.html*
2/ The feasibility of site-level versus watershed-level treatment needs to be assessed during the planning process to determine if the erosion problems are due to local conditions or are the result of stream instability in multiple reaches or over a wide area.

Table 4–5 NRCS Conservation Practice Standards selection for unsuited or insufficient habitat and biodiversity. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system.

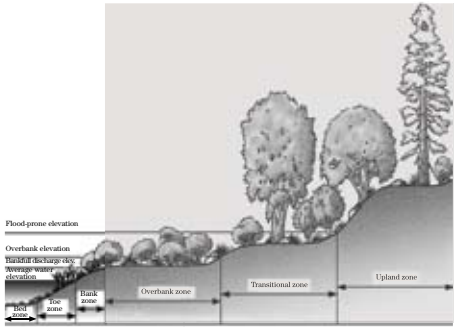
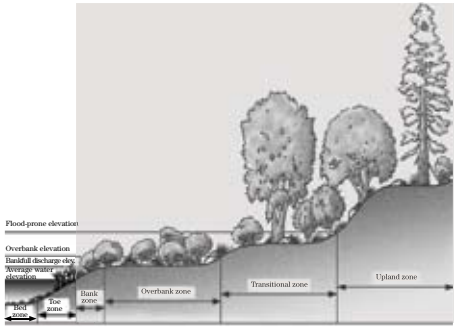
Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: unsuited or insufficient habitat and biodiversity	Landscape zones
Unsuited instream physical habitat (lack of pools, large and fine woody debris, channel depth)	Bed, toe, and bank	Stream Habitat Improvement and Management (395)	Measures applied instream for aquatic species habitat can be enhanced with supporting shade, detritus, and debris from adjacent bank and overbank vegetation. The needs of aquatic species using this practice must be coordinated closely with Channel Bank Vegetation (322), Riparian Forest Buffer (391) (if forest potential) and Riparian Herbaceous Cover (390) (if herbaceous only potential)	
Unsuited near-stream habitat (lack of spatial and vertical structure)	Bank and toe	Channel Bank Vegetation (322)	All practices dealing with vegetation must be coordinated to provide needed habitat for the wildlife species of concern in the bank, overbank, and transitional zones	

Table 4–5 NRCS Conservation Practice Standards selection for unsuited or insufficient habitat and biodiversity. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system—Continued

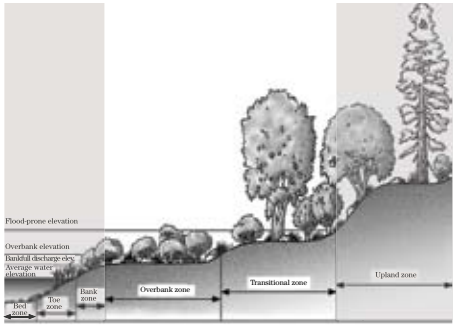
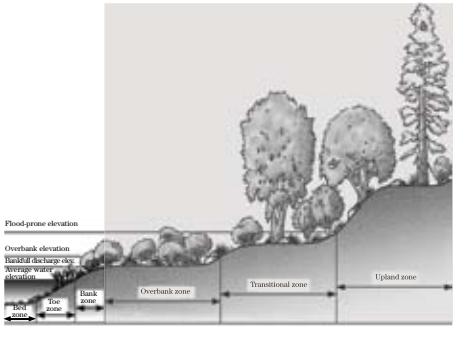
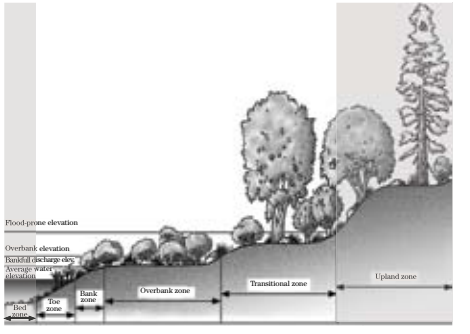
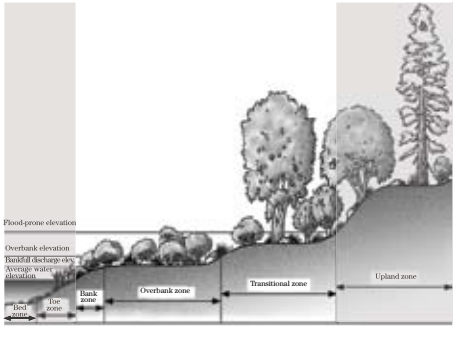
Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: unsuited or insufficient habitat and biodiversity	Landscape zones
Unsuited near stream habitat (lack of spatial and vertical structure)	Overbank and transitional	Riparian Forest Buffer (391) Riparian Herbaceous Cover (390)	Depending on the site's plant community potential for forest or herbaceous, Riparian Forest Buffer (391) and Riparian Herbaceous Cover (390) are used singly, but not together	
		Wetland Wildlife Habitat Management (644)	Wetland Wildlife Habitat Management (644) is used on those areas within the overbank and transitional zones that are wetland in nature	
		Prescribed Grazing (528)	For sites grazed by livestock, use Prescribing Grazing (528) to enhance and maintain desired habitat structure. As a general rule for all practices, native plant species are chosen or favored over introduced species	
Obstructions or channel configurations affecting flow capacity or fish passage	Bed, toe, and bank	Clearing and Snagging (326)	Restore vegetation (native species where practical) to all bank, overbank and transitional areas disturbed by use, ingress or egress of obstruction removal equipment	
		Fish Passage (396)	Consider the quality of stream corridor habitat upstream of obstructions before applying Fish Passage (396)	
		Open Channel (582)	Various techniques including channel meander reconstruction at a site will reconfigure the bed and bank topography and influence the overbank extent, soil moisture and vegetation species	

Table 4–5 NRCS Conservation Practice Standards selection for unsuited or insufficient habitat and biodiversity. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system—Continued

Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: unsuited or insufficient habitat and biodiversity	Landscape zones
Lack of early successional habitat for target wildlife	Bank, toe, overbank, and transitional	Early Successional Habitat Development/ Management (647)	Coordinate plant selection and management of Channel Bank Vegetation (322), Riparian Forest Buffer (391), Riparian Herbaceous Cover (390), and/or Wetland Wildlife Habitat Management (644) to coincide with specifications developed for Early Successional Habitat Development/ Management (647). For sites grazed by livestock, use Prescribed Grazing (528) to enhance and maintain early successional habitat. Field Border (386) can be used at the edge of adjacent upland cropland nearest to the transitional zone to ease movement into and along agricultural land	
Presence of rare or declining native plant communities and impacted wildlife	Bank, overbank, and transitional	Restoration and Management of Declining Habitats (643)	Coordinate specifications and supporting management of all instream and near-stream practices to coincide with specifications developed for Restoration and Management of Declining Habitats (643). Rare and declining sites may need temporary or permanent Use Exclusion (472) to buffer from intensive land use and management. Field Border (386) can be used at the edge of adjacent upland cropland nearest to the transitional zone to ease movement into and along agricultural land	

1/ NRCS National Conservation Practice Standard codes. Specific information for these codes is available at the following Web site: *http://www.nrcs.usda.gov/technical/Standards/nhcp.html*.

Table 4–6 NRCS Conservation Practice Standards selection for unsuited or insufficient production/land use. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system.

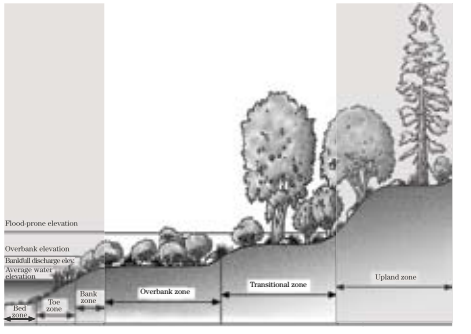
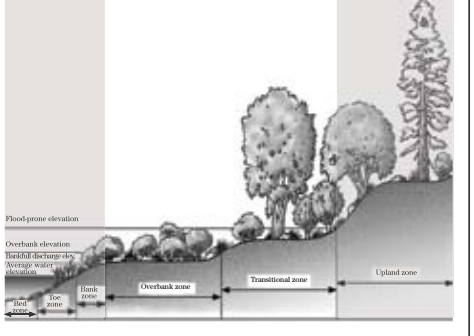
Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: unsuited or insufficient production/land use	Landscape zones
Insufficient forage quantity and quality for livestock	Overbank and transitional	Pasture and Hay Planting (512) Range Planting (550), (528) Prescribed Grazing (328) Silvopasture Establishment (381) Forest Stand Improvement (666)	Plant species are chosen and managed for their forage quality and quantity attributes insofar as compatible with site erosion and sediment control, instability improvement, and habitat improvement. For sites with a combined forage and wood production use, Silvopasture (381) and Forest Stand Improvement (666) are used to manipulate the tree or tall shrub overstory to maintain production of forage cultivars in the understory. For native understory species, only Forest Stand Improvement (666) is used to manipulate the tree or tall shrub overstory. To reduce the risk of erosion, sediment, instability and lack of habitat, the area devoted to forage production may need to be reduced particularly in the overbank zone by Use Exclusion (472), Fence (382) or a modification to Prescribed Grazing (328)	
Under or overstocked forest stands for wood products	Overbank and transitional	Riparian Forest Buffer (391) Forest Stand Improvement (666)	Tree and shrub species are chosen and managed for their wood quality and quantity attributes insofar as compatible with site erosion and sediment control, instability improvement, and habitat improvement. To reduce the risk of erosion, sediment, instability and lack of habitat, the area devoted to wood harvesting may need to be reduced particularly in the overbank zone. Certain techniques (directional felling and skidding) could be used for harvesting in the overbank zone on a periodic basis to maintain vigor of overstory and understory species	

Table 4–6 NRCS Conservation Practice Standards selection for unsuited or insufficient production/land use. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system—Continued

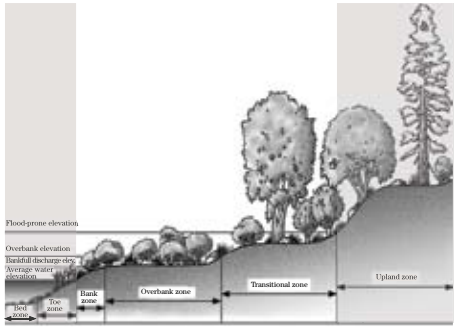
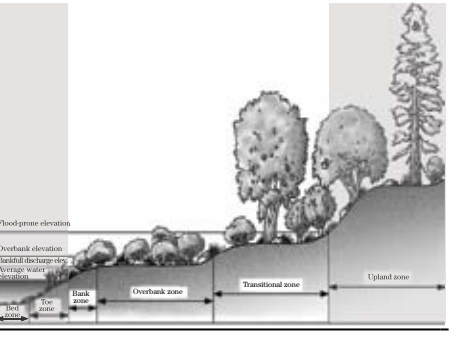
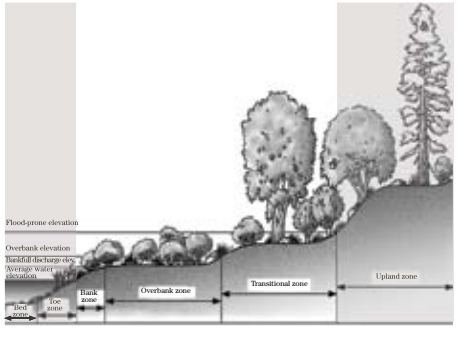
Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: unsuited or insufficient production/land use	Landscape zones
Unimproved recreational opportunities	Bank, overbank, and transitional	Recreation Area Improvement (562) Stream Habitat Improvement and Management (395)	Vegetation is established and/or manipulated to enhance specific recreational uses suited to the site and compatible with site erosion and sediment control, instability improvement, and habitat improvement Recreation structures, land grading, and trails may be concurrently applied with vegetation management. Manipulation of bank and overbank conditions for recreation purposes can be detrimental if not tied to and compatible with a geomorphic/hydraulic analysis at bankfull and flood stages	
Insufficient moisture for desired plant communities	Bank, overbank, and transitional	Irrigation System Microirrigation (441) Sprinkler (442) Surface and Subsurface (443)	A suitable irrigation system with associated practices (Pipeline (430), Irrigation Water Conveyance (428)) can be installed to overcome moisture-deficit conditions detrimental to plant growth and establishment. Irrigation is particularly effective on overbank and transitional zones on incised channel reaches, but it can be costly. To minimize costs, select plant materials that, when well established, can reach their site potential size using available amounts and timing of natural precipitation	
Insufficient nutrients for desired plant communities	Bank, overbank, and transitional	Nutrient Management (590)	Meeting nutritional requirements for existing or new plantings can accelerate their growth, establishment, and function. This is particularly the case with nonnative herbaceous species. Addition of nutrients must be carefully balanced with the nutrient loading at the site and any incoming nutrients in surface and subsurface flows. Nutrition becomes less important as plants and onsite nutrient cycling become well established	

Table 4–6 NRCS Conservation Practice Standards selection for unsuited or insufficient production/land use. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system—Continued

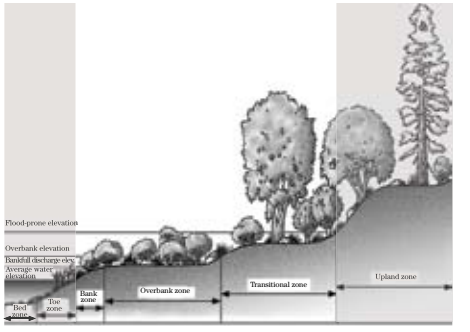
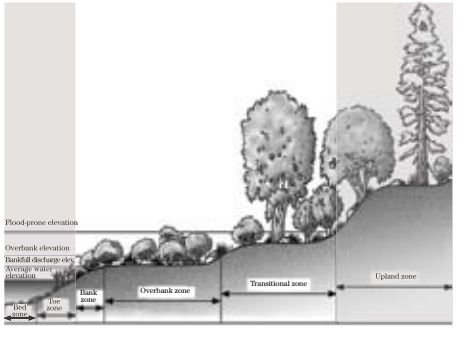
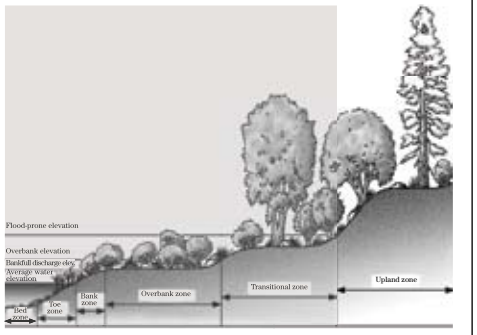
Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: unsuited or insufficient production/land use	Landscape zones
Presence of pests	Bank, overbank, and transitional	Pest Management (595)	A first and foremost step in pest management is selection of plant materials that help achieve desired site conditions and resist local pests. If pests become problematic (weeds, insects, diseases, animals, and other organisms including invasive and noninvasive species), sufficient control helps assure continued function of existing plantings and establishment of new plantings. A variety of control methods are available including cultural, biological, and chemical which must be matched to the problem, the site, and the vegetation. In all cases, pest management design includes an environmental risk analysis to assure that additional problems are not caused (excess pesticides in surface or ground waters)	
Lack of or need for a conveyance structure or travel way across a channel to facilitate land management	Bank, overbank, and transitional	Stream Crossing (578)	Crossings are located where the streambed is stable or where grade control can be provided to create a stable condition. Crossings are typically not placed in shaded conditions if the stream corridor is grazed and there is a potential for livestock loafing in the stream. Stream crossings allow for the passage of water, fish and other aquatic animals within the channel during all seasons of the year. Restore vegetation (native species where practical) as soon after construction as possible to all bank, overbank and transitional areas disturbed by use, ingress or egress of construction equipment)	

Table 4–6 NRCS Conservation Practice Standards selection for unsuited or insufficient production/land use. Guidance is shown using impairment category and landscape zone, with notes on considerations and effects. Impairments and practices[1/] are listed in general order of consideration for use in formulating a resource management system—Continued

Impairment	Landscape zones	Primary NRCS Conservation Practice Standards	Considerations and effects: unsuited or insufficient production/land use	Landscape zones
Point source and nonpoint source pollution, water diversions, flow modifications caused by structures (dams), hydrologic modifications caused by urbanization and other changed land uses	Upland	Nutrient Management (590) Residue and Tillage Management (329, 344, 345, 346), Conservation Crop Rotation (328) Conservation Cover (327) Filter Strip (393) Terrace (600) Water and Sediment Control Basin (638) Waste Treatment and Storage (313, 359. 367, 629, 633, 635) Sediment Basin (350) Subsurface Drain (606) Surface Drainage (606, 607, 608) Constructed Wetland (656) – and others	Protection of watershed areas that contribute water, sediment, and chemicals to the stream may be required to reach the restoration goals of the project. Watershed land use and cover, conservation treatments, and the amount of land converted to urban or suburban uses can have significant effects on runoff to the stream, both in terms of lag times and peak flows	

1/ NRCS National Conservation Practice Standard codes. Specific information for these codes is available at the following Web site: *http://www.nrcs.usda.gov/technical/Standards/nhcp.html*

Figure 4–7 The planner is cautioned to clearly understand the degree and extent of impairments (both onsite and offsite), applicable landscape zones within the CMU, and specific client objectives before considering the selection of conservation practices and vegetation types.

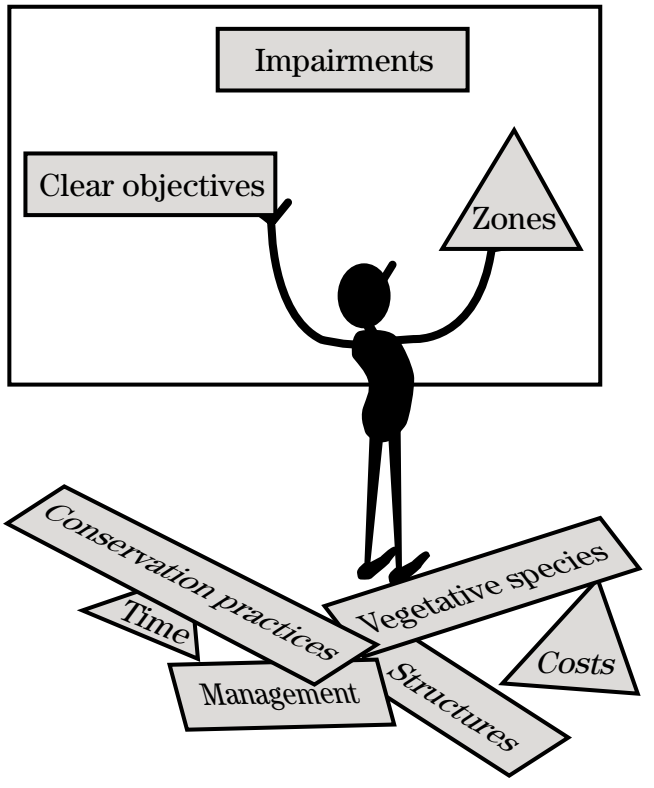


Figure 4–8 Terraces, conservation tillage, and conservation buffers form a system to treat the watershed and protect the stream (Woodbury County in northwestern IA)

(c) Formulating a resource management system for stream restoration

After studying table 4–3 and the applicable conservation practice standards in the FOTG, the planner will have a preliminary idea of the conservation practices that can address site problems and client objectives. Some practices will be either selected or eliminated, based on degree of effect, cost, duplication of outcome (mulching in place of critical area planting for erosion control), or perhaps a change in the client's objectives during the formulation process. This stage of planning (steps 5, 6, and 7) is an iterative phase of the process that must be accomplished with the client. The product of this stage is a plan for the CMU, listing the practices and their locations, types of structures and vegetation, and management requirements. Although more thorough information about certain practices (design options, costs, materials) is sometimes needed by the client to make informed decisions, preparing detailed or preliminary specifications or designs for any practice is not the intent of this phase of planning.

A critical strategy in formulating an RMS for a stream corridor CMU is the interplay between candidate practices in achieving desired conditions. The scenario below illustrates two RMSs developed for a stream corridor-grazing situation. Note how the characteristics of plant materials affect RMS formulation.

Example 1: Streambank erosion control using an RMS that emphasizes management elements for the designed solution

Given benchmark conditions: A CMU at one edge of a farm has a third order stream with fairly wide overbank and transitional zones. Channel banks have accelerated erosion, and the overbank zone has periodic scour erosion; both can be controlled by vegetative means. The client wishes to graze livestock on the entire stream corridor/riparian CMU (both sides of the channel) and use the stream as a water source. Livestock cross the channel at many locations, causing soil compaction and additional bank erosion.

- *RMS Option A*—The Channel Bank Vegetation (322) standard specifies that suitable erosion control plants with low palatability will be used in the bank zone. Pasture and Hay Planting (512) specifies the establishment of forage species with fibrous root systems in the overbank zone. Specifications for Prescribed Grazing (528) identify certain plants in the overbank zone as key forage species that are closely monitored to maintain protection against flood scour erosion. Also, incidental use and trampling damage of bank zone vegetation is monitored, and the livestock are removed immediately when any degradation or loss of vigor is detected. A Stream Crossing (578) in the form of a rock ford is installed to concentrate livestock movement across the channel (livestock choose the ford because of ease of crossing).
- *RMS Option B*—Both Channel Bank Vegetation (322) and Critical Area Planting (342) specify that suitable erosion control plants with low palatability will be used in the bank and overbank zones, respectively. Livestock Use Exclusion (472) is installed in the form of a Fence (382) between the overbank and transitional zones on both sides of the channel. A gated and fenced Stream Crossing (578) is installed to allow ready access to the far side of the CMU and periodic grazing of the bank and overbank zones. Livestock periodically graze the bank and overbank zones for short periods with close monitoring to maintain erosion control (a requirement in the Prescribed Grazing (528) specifications). Because access to water is variable, a Watering Facility (614) is developed within the transitional zone on the far side of the channel. This also improves use of forage on both sides of the stream.

Summary: Both RMSs meet quality criteria and client objectives for livestock grazing. RMS option B affords the greatest assurance that resource concerns and practice purposes will be met, but likely at a higher investment in installation costs and management time. Additional fencing in this option can impact certain types of wildlife and pose a periodic maintenance chore if fences are damaged by floods. RMS option A may require additional monitoring to maintain desired conditions.

Other situations and examples exist for stream/riparian CMUs with cropland, wood production, recreation, or other intensive land uses. Obviously, those sites with little or no demands for crops, wood, forage, or recreation will have the fewest planning constraints and interplay between practices. However, streams and associated riparian areas are typically landscapes with favorable moisture and potential for exploitation. Intensive use of such landscapes will remain the rule, rather than the exception. Planners will need to think through each scenario using the process and techniques presented in this section to formulate sustainable RMSs. Consultation with specialists for complex situations is advised.

Example 2: Streambank erosion control using an RMS that emphasizes a combination of vegetation and structural design elements

Given benchmark conditions: Severe streambank erosion is attacking stream banks in a suburban area, with damage to utilities, loss of land, and degraded habitat. The stream is enlarged, excessive sediment yield, and loss of property and utility services are concerns. In some locations, sewer pipes and gas lines are in imminent danger of collapse. The site constraints are such that relocation of these utilities is not possible. The streambed appears to be stable with no active incision.

- *RMS options*—The objectives are to solve the bank erosion problem, protect the utilities (water and sewer lines) and property, and retain flow-carrying capacities. Measures that are considered include Streambank and Shoreline protection (580) and Channel Bank Vegetation (322). It would be necessary to confirm that the bed is indeed stable. If it is not, some grade control may be necessary. The emphasis will be on protecting the streambank from future undercutting and collapse, so that the toe will be stabilized with rock riprap or gabions. The design will focus on the slope stability that can be achieved with the least impact on land (backyards and easement areas) and result in a stable bank condition. Soil bioengineering will be used to establish woody vegetation that will protect the bank from the erosion of flowing water and also knit together the bank with roots. Where riparian infrastructure is in imminent danger, harder structures such as gabions, sheetpile, and ACBs will be considered. Final design of the solution will depend on hydraulic analyses for the site.

Example 3: Streambank erosion control using an RMS that emphasizes a reliance on vegetative design elements

Given benchmark conditions: Streambank erosion is present along a long stretch of an outside meander bend. This erosion is impacting a farm in a rural area. At one point, a dirt farm road is in jeopardy. No utilities or riparian infrastructure is threatened. The streambed appears to be stable with no active incision. Minimal funds are available for any work.

- *RMS options*—The objectives are to solve the bank erosion problem. The dirt road itself may be relocated. Measures that are considered include Streambank and Shoreline Protection (580) and Channel Bank Vegetation (322). The design may focus on soil bioengineering practices including vertical bundles, live stakes and vegetated stream barbs. All of these techniques, once designed, could be constructed without construction equipment. A site assessment would need to be made to see if Livestock Use Exclusion (472) in the form of a Fence (382) between the overbank and transitional zones on the bank would also reduce some of the stress.

Example 4: Streambank erosion control using an RMS that emphasizes a complete reconstruction of a natural stream channel

Given benchmark conditions: A natural channel was straightened and widened to provide flood control benefits to rural farmland. Historically, the stream had supported healthy populations of fish which are now considered to be threatened. This old channelization of the stream has resulted in a loss of habitat and impacts to the threatened fish species. Riparian land use has now changed such that the original flood control purposes of the project are no longer an issue.

- *RMS options*—The objectives are to restore the stream so that it will support a population of the target fish species. A healthy stream corridor is necessary to achieve this goal and to solve the bank erosion problem. Measures that are considered include Streambank and Shoreline Protection (580), Channel Bank Vegetation (322), Channel Stabilization (584), Open Channel (582), Restoration and Management of Declining Habitats (643), Riparian Forest Buffer (391), and Stream Habitat Improvement and Management (395). The design may focus on a complete recreation of the stream channel. This will involve determining a stable planform, section, and profile of the stream. Techniques which serve to stabilize the grade and protect the banks in targeted areas may be necessary. Instream and edge habitat features, such as soil bioengineering practices including vertical bundles, live stakes, and vegetated stream barbs, may also be used. The analysis, design, and construction effort for such a project may be significant.

(d) Specifications and designs at the conservation practice level

After examining the RMS alternatives and associated practices with the planner, the client makes decisions about the CMU. With a definitive plan of what will occur, the preliminary specifications and designs for stream corridor restoration can be formulated. This is the start of NPPH planning step 8, Implement the Plan. Individual conservation practice standard criteria contained in the local FOTG will guide the planner in:

- location, extent, design specifications, and operation of physical structures
- plant species selection, layout, and spacing
- site preparation and planting techniques
- site management to progress to and maintain desired site conditions

Generic specifications and designs will not be presented here because of the wide array of ecological regions, site conditions, implementation techniques, and plant materials. However, the considerations and effects column in tables 4–4, 4–5, and 4–6 provides important considerations for developing specifications and designs. The planner is advised to carefully study the tables and local FOTG practice standard criteria. During the development of detailed specifications and designs, the planner and client may decide to modify the original RMS. This is a normal iteration of the planning process.

654.0405 Evaluating success of stream restoration designs

The main purpose for evaluating the restoration treatment (planning step 9 in table 4–1) is to determine if desired future conditions are being achieved at the expected level and rate. In the NRCS CPP, desired conditions in relation to existing benchmark conditions are first established and documented in planning step 4 and later used in step 6 to evaluate alternative RMSs. During treatment evaluation, two basic questions are answered:

- Have practices been installed as planned?

  This is answered by examining the plan, implementation schedule, designs in planning steps 7 and 8 (table 4–1), and confirming that practices have been installed. Plans are subject to change and modification, so it is important to verify that practices have actually been applied when and where specified.

- Are desired future conditions being achieved at the specified level and timing?

  The desired conditions were originally specified during planning step 4 and used again in step 6 to evaluate the expected performance of RMS options and, ultimately, help the client choose the best one. To respond to this question, conditions may be measured on an absolute basis (tons of sediment passing a reference point, presence or absence of instream bars, soil loss calculated in scour areas), on an interpretive basis (some kind of index score based on habitat components), or using modeling where before and after values for model variables compared.

The specific measurements and techniques used in planning steps 3 and 4 (table 4–1) are again remeasured during the step 9 evaluation. The results of the evaluation or progress towards success can be expressed in terms of yes or no or as a percentage of improvement. The following examples illustrate two cases of treatment evaluation.

Case 1

Given: Bank erosion has a recession rate of about 0.5 foot per year. Site and upstream conditions have remained constant during the past 10 years. The RMS was developed based on determination that bank erosion is controllable at the site, using primarily Channel Bank Vegetation (322) and Riparian Forest Buffer (391). The recession rate is determined to be of an accelerated nature (above and beyond natural, geologic erosion for the stream type). The desired condition is no evidence of bank recession within 10 years of the last applied RMS practice.

Evaluation: RMS practices have all been applied, with the last practice installed 10 years ago. There is no visual evidence of bank recession. The evaluation indicates yes, the RMS is successful.

Case 2

Given: The client owns a significant part of an upstream watershed. The Agricultural Nonpoint Source Pollution Model (AGNPS) has been used to estimate sediment production at the lowest downstream point of the client's stream system. The stream corridor CMU is continuous along both sides of the stream system and excludes agricultural and other intensive land uses to 150 feet on each side of the stream (perpendicular to streamflow) as measured from the bankfull level. The desired condition is a 70 percent reduction of annual sediment within 10 years of the last applied RMS practice.

Evaluation: RMS practices which include riparian vegetation establishment have all been applied, with the last practice installed 10 years ago. Conditions have been reassessed, and the model rerun. Results indicate there has been only a 30 percent decrease in annual sediment. The evaluation indicates the RMS is not yet successful. If the planner and client are dissatisfied with the evaluation outcome, they can decide to reassess the efficacy of the RMS, reset the target or threshold for success, increase the allowable response time, or use a more exacting model (if available) to improve the sediment yield estimate.

The evaluation procedure presented above is based on the assumption that desired future conditions have been well established during the planning process. If this is not the case, land and water treatment evaluation may require the development of a special strategy in consultation with interdisciplinary specialists to determine the effects of applied RMSs. A challenge with a retroactive approach is re-creating pretreatment conditions and estimates of measurements.

654.0406 Conclusion

The process for designing stream restoration solutions is an integral part of the CPP. Solutions may result in simple designs that may only require changes in management and removal of disturbance factors. However, depending on the complexity of the problem, a solution that integrates both management and structural approaches may be needed. Structural approaches may include design elements that integrate soil materials, plants, large woody material, concrete, rock, steel, or other materials. In either case, the focus should be on solving the stated problems, as well as conserving and restoring natural resources to the extent possible. Local conditions determine what kinds of data are needed for preliminary and detailed designs, and the design process will vary according to the types and number of design elements, complexity of the project, and degree of risk involved, as described in NEH654.02.

The best design will be one that results in minimal maintenance and is also self-sustaining. This may not always be possible, depending on the specific goals and objectives and overall constraints, especially in areas with impaired watersheds, rigid constraints of land ownership, or jurisdictional requirements.

The following chapters provide more detailed information on the use and applicability of various tools for analyzing and designing stream restorations. Some are well entrenched in scientific research and experience, while others reflect the state of the art.

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**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 5 Stream Hydrology

Issued August 2007

Cover photo: Quantifying the flow of the stream involves analysis of rainfall/runoff, storm recurrence intervals, and watershed and flood plain conditions.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 5 Stream Hydrology

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Chapter 5 Stream Hydrology

654.0500 Purpose

Stream restoration design should consider a variety of flow conditions. These flows should be considered from both an ecological, as well as a physical perspective. Many sources and techniques for obtaining hydrologic data are available to the designer. This chapter provides a description of the flows and their analyses that should be considered for assessment and design. The computation of frequency distributions, with an emphasis on the log-Pearson distribution as provided in the U.S. Water Resources Council (WRC) Bulletin 17B, is addressed in detail. Examples have been provided to illustrate the methods. Transfer equations, risk, and low-flow methods are also addressed. Finally, this chapter describes advantages and limitations of four general approaches widely used for estimating the channel-forming discharge or dominant discharge.

654.0501 Introduction

Hydrologic analysis has historically been the starting point for channel design. Current and future flows were estimated, then the designer proceeded to further analysis. However, the complexities of stream restoration projects often require that hydrologic analysis be conducted in close coordination with a study of stream geomorphology and stream ecology.

Hydrologic computations are an integral part of any stream design and restoration project. However, design objectives for a stream restoration project cannot adequately be met by assessing channel behavior for only a single discharge. A stream restoration project usually has several design flows selected to meet various objectives. For example:

- Estimates of future flow conditions are often required to properly assess project performance over the long term.
- Estimates of low flows such as 7-day low flow often define critical habitat conditions.
- Estimates of channel-forming discharges are used to estimate stable channel dimensions.
- Flood flow estimates are used to assess stability of structures and flood plain requirements, as well as for scour depth prediction.

Many techniques are available to the designer for determining the various discharges used in assessment and design. The level of accuracy required for the different hydrologic analyses, as well as the need to estimate the different flows, is dependent on the site-specific characteristics of each project. Therefore, it is important to understand not only what each design flow represents, but also the underlying assumptions and the limitations of the techniques used to estimate the flow.

654.0502 Overview of design discharges

A description of some of the various types of design discharges is provided in this section. Although a project may not require the use of all of these flows for design, the hydraulic engineer/designer should still consider how the project will perform during a range of flow conditions.

(a) Low flows

Design of a low-flow channel may be required as part of a channel restoration. Normally, the design of the project for low flows is performed to meet biological goals. For instance, summer low flows are often a critical period for fish, and project goals may include narrowing the low-flow channel to provide increased depths at that time. Design flows may also be necessary to evaluate depths and velocities for fish spawning areas or fish passage during critical times of the year. Coordination with the biologist on the study team and familiarity with regulatory requirements are essential to make sure an appropriate flow (or range of flows) is selected.

(b) Channel-forming discharge

A determination of channel-forming discharge is used for many stability assessment tools and channel design techniques. The channel-forming discharge concept is based on the idea that for any given alluvial stream there exists a single discharge that, given enough time, would produce the width, depth, and slope equivalent to those produced by the natural flow in the stream. This discharge, therefore, dominates channel form and process. The channel-forming discharge concept evolved from the dominant discharge concept used to design irrigation canals in the latter part of the nineteenth and early part of the twentieth centuries. It is recognized, however, that the channel-forming discharge is a theoretical concept and may not be applicable to all stream types, especially flashy and ephemeral streams.

Depending on the application, channel-forming discharge can be estimated by several methods, based on:

- bankfull indices
- effective discharge
- specific recurrence interval
- drainage area

The distinction between channel-forming discharge and the other deterministic discharges is frequently confused, as the terms are used interchangeably. This chapter describes advantages and limitations of the four widely used general approaches.

(c) High discharge

The reaction of a channel to a high discharge can be the impetus for a stream restoration project. An identified high-flow event is often used in the design and specification of a design feature. The choice of a maximum design flow for stability analysis should be based on project objectives and consequences of failure. For example, the 100-year discharge might be used to design bank protection in a densely populated area, while a 10-year discharge might be appropriate in a rural stream. Other examples include:

- It may be a requirement to demonstrate that a proposed project will not raise the water surface profiles produced by a 5-year event (often referred to as nuisance-level flooding) sufficiently to adversely affect riparian infrastructure such as county roads, parks, and playgrounds.
- A significant flood event (typically no smaller than the 10-year frequency discharge) is used to estimate forces and compute scour depths at proposed habitat features constructed with logs. The goal is that these hard project features will withstand a flood of this magnitude without major damage, movement, or flanking.
- A significant flood event may have caused severe bank erosion, initiating a request to fix the erosion problems. It may be a requirement that any proposed fix provide stabilization that will be able to withstand a repetition of the forces produced by this event.
- It may be a requirement of the project design that the impacts of a 25-year flood event be limited to minor deposition of sediment and de-

bris; localized scour, erosion, and stone movement; and erosion of vegetation.

- Often the impact on the water surface profile for the 100-year flood event must be submitted as part of the project's permitting requirements. In many cases, it is a requirement to demonstrate that a proposed project will not result in increases to the 100-year flood plain area.
- It may also be necessary to estimate the flood-level reduction of a project on a 50-year flood event or for a larger event (such as the design discharge for a flood control project.).

(d) Flow duration

A flow-duration curve represents the percentage of time that a flow level is equaled or exceeded in a stream. This analysis is done for sediment transport assessments and ecological assessments, as well as for assessments of the duration of stress on soil bioengineering bank stabilization techniques.

Comparing flow-duration curves of different systems in a single basin or across a larger physiographic region can lend useful insight into a variety of watershed concerns. Issues such a flow contributions from ground water, watershed geology and geomorphology, and degree of flow regulation can also be examined, in part, with such a comparative analysis.

(e) Seasonal flows

It is often important to determine how the proposed restoration project will perform with low or normal flows. In addition, seasonal flow variations can have critical habitat importance. For example, a project goal may include a minimum flow depth during a critical spawning period for anadromous fish species and a lower minimum depth for resident fish species. The same techniques used to develop flow-duration curves for sediment analysis can also be used to assess and design for habitat conditions.

In many states, the U.S. Geological Survey (USGS) has developed regional regression curves for the critical flow periods. This might be the 10-year, 7-day low flow.

(f) Future flows

Estimates of future flow conditions are often required to properly assess future project performance. In some areas, the USGS has developed regional peak flow frequency curves that include a variable that can be used to estimate the impact of future changes in land use, such as an increase in the percent of impervious area for urban development. For example, typically 10 to 20 percent of the average rainfall event becomes runoff for an undeveloped watershed, while 60 to 70 percent of the average rainfall event becomes runoff for a developed (urbanized) watershed. However, regional equations typically do not include this variable, and a hydrologic model must be used to determine the change in the peak flow.

(g) Regulatory

Some Federal and state agencies have established minimum streamflow requirements for fish habitat. For example, the Federal Emergency Management Agency (FEMA) has established flood hazard maps for the 100-year and 500-year flood events and has estimated the flow associated with these events. Consultation with the appropriate authorities is needed if there is a possibility that a project will impact this flood level. Also, the U.S. Environmental Protection Agency (EPA) has established minimum flow requirements in many areas. These should be considered when determining the required design flows. While the determination and maintenance of these established flows may be based more on administrative decisions than current hydrologic data and analysis, they can be a critical component of a stream analysis or project design. A further description of regulatory requirements is provided in NEH654.17.

654.0503 Probability

Streamflow events are typically referred to by their return period. A return period of Rp means that in any given year, the event has a probability of occurrence (P):

$$P = \frac{1}{Rp} \qquad \text{(eq. 5–1)}$$

For example, a 100-year storm has an annual probability of occurrence of P=1/Rp=1/100=0.01 or 1 percent. Therefore, it is synonymous to speak of a 1 percent storm as a 100-year storm.

Risk is defined as the probability that one or more events will exceed a given magnitude within a specified period of years. Risk is calculated by means of the binomial distribution given in simplified form as follows:

$$R = 1 - (1 - P)^n \qquad \text{(eq. 5–2)}$$

where:
R = risk in decimal number
P = exceedance probability of event
n = number of years

The risk formula may be applied to many different scenarios, including the following:

- The likelihood of a 100-year flood occurring at least once in the next 100 years is 63 percent

$$R(100) = 1 - \left(1 - \frac{1}{100}\right)^{100} = 0.63$$

- The likelihood of a 100-year flood occurring at least once in the next 50 years is 39 percent

$$R(100) = 1 - \left(1 - \frac{1}{100}\right)^{50} = 0.39$$

- The likelihood of a 100-year event occurring at least once in 1,000 years is 99.996 percent, a very high probability, but never 100 percent.
- There is a 97 percent risk of a bankfull, 2-year recurrence interval discharge (50% annual chance) being exceeded in the next 5 years.
- Likewise, the 10-year discharge has a 41 percent risk of being exceeded in the next 5 years, or conversely, a 59 percent chance of not being exceeded.

Expected probability is a measure of the central tendency of the spread between confidence limits. Expected probability adjustment attempts to incorporate effects of uncertainty in application of frequency curves. The adjustment lessens as the stream record lengthens. Use of expected probability adjustment is often based on a policy decision.

It is important to note that a precipitation event may not have the same return period as a flow event. On small watersheds, a 100-year rainfall event may produce a 100-year flow or flood event. On large watersheds, however, the 100-year flow event may be produced by a series of smaller rainfall events. This distinction should particularly be kept in mind by the practitioner who is working with projects in large watersheds.

Equation 5–2 can also be rearranged to aid in determining a design storm (see example 1).

Example 1: Risk-based selection of design storm

Problem: A bank protection project involves considerable planting and soil bioengineering. However, the proposed planting would not be able to withstand the design storm until firmly established. The designer is asked to include reinforcement matting that will have a 90 percent chance of success over the next 5 years. What is the design storm?

Solution:

Step 1. Calculate the probability of an event occurring that is larger.

90 percent chance of success means that there is a 10 percent chance that an event will be larger.

Step 2. Rearrange equation 5–2 as follows:

$$R = 1 - (1 - P)^n$$

$$P = 1 - \sqrt[n]{(1 - R)}$$

And solve as:

$$P = 1 - \sqrt[5]{1 - 0.1}$$

$$= 0.0208$$

Step 3. Rearrange equation 5–1 as follows:

$$P = \frac{1}{R_p}$$

$$R_p = \frac{1}{P}$$

And solve as:

R_p = 47.9 or about a 50-year storm

654.0504 Gage analysis for flow frequency

Flow frequency analysis relates the magnitude of a given flow event with the frequency or probability of that event's exceedance. If a stream gage is available and the conditions applicable, a gage analysis is generally considered preferable, since it represents the actual rainfall-runoff behavior of the watershed in relation to the stream. A variety of Federal, state, and local agencies operate and maintain stream gages. Currently, the USGS operates about 7,000 active stream gaging stations across the country. Such data are also available for about 13,000 discontinued gaging stations. Historical peak flow data can be found at the following USGS Web site at:

http://nwis.waterdata.usgs.gov/usa/nwis/peak

It is important to determine if the present watershed conditions are represented by the stream gage record or if there has been a significant change in land use. If there has been a significant increase in urbanization, the historical record may not represent current conditions. While many hydrologic techniques are available for the prediction of frequency of flow events, this chapter presents concepts and techniques for analyzing peak flows and, to a lesser extent, low flows, following the recommendations of Guidelines for Determining Flood Flow Frequency, Bulletin 17B (WRC 1981).

Flow event data may be analyzed graphically or analytically. In graphical analysis, data are arrayed in order of magnitude, and each individual flow event is assigned a probability or recurrence interval (plotting point). The magnitudes of the flow events are then plotted against the probabilities, and a line or frequency curve is drawn to fit the plotted points. Peak flow data are usually plotted on logarithmic probability scales, which are spaced for the log-normal probability distribution to plot as a straight line. Data are often plotted to verify that the general trend agrees with frequency curves developed analytically.

(a) Analysis requirements and assumptions

In performing a frequency analysis of peak discharges, certain assumptions need to be verified including data independence, data sufficiency, climatic cycles and trends, watershed changes, mixed populations, and the reliability of flow estimates. The stream gage records must provide random, independent flow event data. These assumptions need to be kept in mind, otherwise, the resultant distribution of discharge frequencies may be significantly biased, leading to inappropriate designs and possible loss of property, habitat, and human life.

Data independence
To perform a valid peak discharge frequency analysis, the data points used in the analysis must be independent, that is, not related to each other. Flow events often occur over several days, weeks, or even months, such as for snowmelt. Only the peak discharge for each flow event should be used in the frequency analysis. Secondary peaks are dependent on each other and are not appropriate for use in a frequency analysis. Using secondary peaks would result in lower peak flows for a given frequency, since it would exaggerate the frequency of the magnitude of the event. It is common practice to minimize this problem by extracting annual peak flows from the streamflow record to use in the frequency analysis.

Data sufficiency
Gage records should contain a sufficient number of years of consecutive peak flow data. To minimize bias, this record should span both wet and dry years. In general, a minimum of 10 years is required (WRC 1981). However, longer gage records are generally recommended to estimate larger return periods and/or if there is a potential bias in the data set. This is addressed later in the climate bias example. If a gage record is shorter than optimum, it may be advisable to consider other methods of hydrologic estimations to support the gage analysis.

It is also important to use data that fully capture the peak for peak flow analysis. If a stream is flashy (typical of small watershed), the peak may occur over hours or even minutes, rather than days. If daily averages are used, then the flows may be artificially low and result in an underestimate of storm event values.

Therefore, for small watersheds, it may be necessary to look at hourly or even 15-minute peak data.

Annual duration

Gage analysis for flows with return intervals in excess of 2 years is typically conducted on annual series of data. This is the collection of the peak or maximum flow values that have occurred for each year in the duration of interest. Each year is defined by water year (Oct. 1 to Sept. 30).

Partial duration

When the desired event has a frequency of occurrence of less than 2 years, a partial duration series is recommended. This is a subset of the complete record where the analysis is conducted on values that are above a preselected base value. The base value is typically chosen so that there are no more than three events in a given year. In this manner, the magnitude of events that are equaled or exceeded three times a year can be estimated. Care must be taken to assure that multiple peaks are not associated with the same event, so that independence is preserved.

The return period for events estimated with the use of a partial duration series is typically 0.5 years less than what is estimated by an annual series (Linsley, Kohler, and Paulhus 1975). While this difference is fairly small for large events (100-yr for a partial vs. 100.5-yr for an annual series), it can be significant at more frequent events (1-yr for a partial vs. 1.5-yr for an annual series). Therefore, while an annual series may be sufficient to estimate the magnitude of a channel-forming discharge, it may not provide a precise estimate for the actual frequency of the discharge. It should also be noted that there is more subjectivity at the ends of both the annual and partial duration series frequency curves.

Climatic cycles and trends

Climatic cycles and trends have been identified in meteorological and hydrological records. Cycles in streamflow have been found in the world's major rivers. For example, Pekarova, Miklanek, and Pekar (2003) identified the following cycles of extreme river discharges throughout the world (years): 3.6, 7, 13, 14, 20 to 22, and 28 to 29. Some cycles have been associated with oceanic cycles, such as the El Niño-Southern Oscillation in the Pacific (Dettinger et al. 2000) and the North Atlantic Oscillation (Pekarova, Miklanek, and Pekar 2003). Trends in streamflow volumes and peaks are less apparent. However, trends in streamflow timing are likely, as have been presented in Cayan et al. (2001) for the western United States.

The identification of both cycles and trends is hampered by the relatively short records of streamflow available, as streamflow data increase, more cycles and trends may be identified. However, sufficient evidence does currently exist to warrant concern for the impact of climate cycles on the frequency analysis of peak flow data, even with 20, 30, or more years of record.

When performing a frequency analysis, it can be important to also analyze data at neighboring gages (that have longer or differing period of records) to assess the reasonableness of the streamflow data and frequency analysis at the site of interest. Keeping in mind the design life of the planned project and relating this to any climate cycles and trends identified during such a period, one can identify, in at least a qualitative manner, the appropriateness of the streamflow data. A case study is provided in example 2 that describes an analysis completed to assess climatic bias.

Paleoflood studies use geology, hydrology, and fluid dynamics to examine evidence often left by floods and may lead to a more comprehensive frequency analysis. Such studies are more relevant for projects with long design lives, such as dams. For more information on paleoflood techniques, see Ancient Floods, Modern Hazards: Principles and Applications of Paleoflood Hydrology (House et al. 2001).

Watershed changes

Land use and water use changes in watersheds can alter the frequency of high flows in streams. These changes, which are primarily caused by humans, include:

- urbanization
- reservoir construction, with the resulting attenuation and evaporation
- stream diversions
- construction of transportation corridors that increase drainage density
- deforestation from logging, infestation, high intensity fire
- reforestation

Example 2: Climatic bias case study

The Willow Creek watershed of the northern San Juan Mountains of Colorado has a wide range of stream-related projects being designed. This includes the remediation of drainage from tailings piles and mines; a braided to sinuous stream restoration; and the rehabilitation of a flume which carries flood flow through the town of Creede, Colorado. Discharge frequency estimates are necessary for all of these projects. The USGS had a gage operable in Creede for 32 years, from 1951 through 1982. Flow peaks measured for this 35.3-square-mile watershed ranged from 66 to 410 cubic feet per second. Thirty-two years of data is usually a reasonable record length for performing a frequency analysis. However, when six historic events were taken into account, the results of the 32-year frequency analysis appeared to be biased on the low side.

Records show that the historic events (with estimated peak flows of 1,200 ft^3/s and greater) occurred in the first half of the century in the Willow Creek watershed. This leads to a series of issues that should be examined:

- Were these historic peak estimates computed properly?
- Were these high flows random occurrences?
- Does this confliction indicate that all of the systematic record was recorded during a period of lesser precipitation and runoff?
- Or is some other mechanism occurring?

To shed more light on this situation an analysis was performed on two nearby, primarily undeveloped, watersheds: Carnero Creek, a 117-square-mile watershed with 78 years of record, and LaGarita Creek, a 61-square-mile watershed with 81 years of record. A sensitivity analysis was performed to assess the impact of a varying period of record. It was assumed that the records at these two locations cover three different periods: the actual period of record, the first half of the record, and the second half of the record. Frequency analyses were performed on each of these records. Results from this analysis are shown in table 5–1.

Table 5–1 Sensitivity analysis on gage record, Willow Creek case study

Gage ID	**08230500**			**Gage ID:**	**08231000**		
Stream	**Carnero Creek**			**Stream:**	**LaGarita Creek**		
Drainage area:	**117 mi^2**			**Drainage area:**	**61 mi^2**		
Years of record:	**1920–23, 26–28, 30, 32–2001**			**Years of record:**	**1920, 22–2001**		
	Log-Pearson results				**Log-Pearson results**		
	Full	**First half**	**Second half**		**Full**	**First half**	**Second half**
Record (yr)	78	39	39	Record (yr)	81	40	41
200-yr (ft^3/s)	1,690	1,780	695	200-yr (ft^3/s)	840	816	552
100-yr (ft^3/s)	1,290	1,470	554	100-yr (ft^3/s)	711	736	468
50-yr (ft^3/s)	958	1,180	435	50-yr (ft^3/s)	591	652	392
25-yr (ft^3/s)	694	921	333	25-yr (ft^3/s)	481	564	322
10-yr (ft^3/s)	424	618	223	10-yr (ft^3/s)	348	441	239
5-yr (ft^3/s)	271	417	155	5-yr (ft^3/s)	257	340	181
2-yr (ft^3/s)	118	187	80	2-yr (ft^3/s)	141	193	108
1.25-yr (ft^3/s)	53	79	43	1.25-yr (ft^3/s)	77	99	66

Example 2: Climatic bias case study—Continued

This analysis indicates sensitivity towards the specific period of record. Possible reasons for this bias include watershed changes such as forestry practices, climate cycles, and climate trends. For this Willow Creek example, additional stream gages within the region could be analyzed and extrapolated, using a regional regression methodology, to develop a more robust discharge frequency. A comparison of the computed discharges on a square-mile basis for selected discharges may show that the full record for the three stations is not that dissimilar.

For all frequencies, varying time periods used in a frequency analysis result in readily apparent differences. If a project's design were based on a frequency analysis for a gage with data gathered only during the second half of the twentieth century (as is the case in Willow Creek), this design may have attributes that are inappropriately sized.

Before a discharge-frequency analysis is used, or to judge how the frequency analysis is to be used, watershed history and records should be evaluated to assure that no significant watershed changes have occurred during the period of record. If such significant change has occurred, the period of record may need to be altered, or the frequency analysis may need to be used with caution, with full understanding of its limitations.

Particular attention should be paid to watershed changes when considering the use of data from discontinued gages. It was common to discontinue the small (<10 mi^2) drainage areas in the early 1980s. Aerial photographs can provide useful information in determining if the land use patterns of today are similar to those during the gage's period of record. Each gage site must be evaluated on an individual basis to determine whether today's watershed is still represented by yesteryear's flow records.

Mixed populations

At many locations, high flows are created by different types of events. For example, in mountain watersheds, high flow may result from snowmelt events, rain on snow events, or rain events. Also, tropical cyclones may produce differences from frontal systems. Gages with records that contain such different types of events require special treatment.

Reliability of flow estimates

Errors exist in streamflow records, as with all measured values. With respect to USGS records, data that are rated as excellent means that 95 percent of the daily discharges are within 5 percent of their true value, a good rating means that the data are within 10 percent of their true value, and a fair rating means that the data are within 15 percent of their true value. Records with greater than 15 percent error are considered poor (USGS 2002b).

These gage inaccuracies are often random, possibly minimizing the resultant error in the frequency analysis. Overestimates may be greatest for larger, infrequent events, especially the historic events. For example, research indicates that mobile bed streams cannot maintain supercritical flow over long distances and time periods. Therefore, a critical flow assumption is more appropriate in these situations. For more information on these methods, see Grant (1997) and Webb and Jarrett (2002). If consistent overestimation has occurred, the error is not random, but is instead, a systematic bias.

Regulated flows

Flows from dams are considered to be regulated flows. The normal statistical techniques in Flood Flow Frequency, Bulletin No. 17B (WRC 1981) cannot be used in these situations. However, in some cases, standard graphic statistical techniques can be used to determine the frequency curve. A review of the reservoir operation plan and project design document will provide information on the downstream releases.

(b) Frequency distributions

A flow frequency analysis is a consistent, statistical method for denoting the probability of occurrence of flows at a specific point in a stream system. Such relationships are required in the planning and design of structures in and near streams. However, peak flow frequency analysis techniques have limitations as described in NEH654.0502. Until hydrologic process modeling becomes more developed, the use of the following statistical methods is necessary.

Statistical parameters

The basic statistical parameters used in frequency analyses (applied to both normal and logarithmic values) are:

Mean:

$$\text{Mean: } \overline{X} = \frac{\sum X}{n} \qquad \text{(eq. 5–3)}$$

Standard deviation:

$$\text{Standard deviation: } S = \left[\frac{\sum\left(X-\overline{X}\right)^2}{n-1}\right]^{0.5} \qquad \text{(eq. 5–4)}$$

Skew coefficient:

$$\text{Skew coefficient: } G = \frac{n\sum\left(X-\overline{X}\right)^3}{(n-1)(n-2)S^3} \qquad \text{(eq. 5–5)}$$

where:

X = annual peak flow or logarithm of annual peak flow

n = length of data set

The mean is the arithmetic average of the data. It is the expected value of the data. The standard deviation is essentially an indication of how much the data is

spread about the mean. The smaller the standard deviation value, the closer are the data points to the mean. For a normally distributed data set, approximately two-thirds of the data will be within plus or minus one standard deviation of the mean, while almost 95 percent will be within two standard deviations of the mean. Skewness is the third central moment about the mean and a measure of symmetry (or rather the lack of symmetry) of a data set (Fripp, Fripp, and Fripp 2003). If values are further from the mean on one side than the other, the distribution will have a larger skew. The skew has a large effect on the shape, and thus, the value of a distribution. Transformations (such as converting to logarithmic forms) are often made on skewed data. Spreadsheets are commonly used to compute these parameters.

Common distributions

Four distributions are most common in frequency analyses of hydrologic data, specifically the normal distribution, log-normal distribution, Gumbel extreme value distribution, and log-Pearson type III distribution. The log-Pearson distribution has been recommended by the WRC and is the primary method for discharge-frequency analyses in the United States. It is also recommended in NEH630.18.

However, the use of the log-Pearson distribution is not universal. For example, Great Britain and China use the generalized extreme value distribution and the log-normal distribution, respectively, while other countries commonly use other distributions (Singh and Strupczewski 2002). This section presents an overview of the four distributions. However, only the log-Pearson distribution will be addressed in detail.

Normal distribution

The normal or Gaussian distribution is one of the most popular distributions in statistics. It is also the basis for the log-normal distribution, which is often used in hydrologic applications. The distribution, as used in frequency analysis computations, is provided:

$$X_{N,T} = \overline{X} + K_{N,T}S \qquad \text{(eq. 5–6)}$$

where:

$X_{N,T}$ = predicted discharge, at return period T

$\overline{X}$ = average annual peak discharge

$K_{N,T}$ = normal deviate (z) for the standard normal curve, where area = $0.50 - \frac{1}{T}$

S = standard deviation, of annual peak discharge

Log-normal distribution

The annual maximum flow series is usually not well approximated by the normal distribution; it is skewed to the right, since flows are only positive in magnitude, while the normal distribution includes negative values. When a data series is left-bounded and positively skewed, a logarithmic transformation of the data may allow the use of normal distribution concepts through the use of the log-normal distribution. This transformation can correct this problem through the conversion of all flow values to logarithms. This is the method used in the log-normal distribution:

$$X_{LN,T} = \overline{X}_l + K_{LN,T}S_l \qquad \text{(eq. 5–7)}$$

where:

$X_{LN,T}$ = logarithm of predicted discharge, at return period

$\overline{X}_l$ = average of annual peak discharge logarithms

$K_{LN,T}$ = normal deviate (z), of logarithms for the standard normal curve, where area = $0.50 - \frac{1}{T}$

S_l = standard deviation, of logarithms of annual peak discharge

Gumbel extreme value distribution

Peak discharges commonly have a positive skew, because one or more high values in the record result in the distribution not being log-normally distributed. Hence, the Gumbel extreme value distribution was developed.

$$X_{G,T} = \overline{X}_l + K_{G,T}S \qquad \text{(eq. 5–8)}$$

where:

$X_{G,T}$ = predicted discharge, at return period T

$\overline{X}_l$ = average annual peak discharge

$K_{G,T}$ = a function of return period and sample size, provided in table 5–2

S = standard deviation of annual peak discharge

Log-Pearson Type III distribution

The log-Pearson type III distribution applies to nearly all series of natural floods and is the most commonly used frequency distribution for peak flows in the United States. It is similar to the normal distribution, except that the log-Pearson distribution accounts for the skew, instead of the two parameters, standard deviation and mean. When the skew is small, the log-Pearson distribution approximates a normal distribution. The basic distribution is:

$$X_{LP,T} = \overline{X}_l + K_{LP,T}S_l \qquad \text{(eq. 5–9)}$$

where:

$X_{LP,T}$ = logarithm of predicted discharge, at return period T

$\overline{X}$ = average of annual peak discharge logarithms

$K_{LP,T}$ = a function of return period and skew coefficient, provided in table 5–3

S_l = standard deviation of logarithms of annual peak discharge

The mean in a log-Pearson type III distribution is approximately equal to the logarithm of the 2-year peak discharge. The standard deviation is the slope of the line, and the skew is shown by the curvature of the line.

The log-Pearson type III distribution has been recommended by the WRC, and the NRCS has adopted its use (NEH630.18). Details on the use of the log-Pearson distribution for the determination of flood frequency are presented later in this chapter. More information is also available in WRC Bulletin 17B.

Plotting position

The graphical evaluation of the adequacy-of-fit of a frequency distribution is recommended when performing an analysis. Plotting positions are used to estimate the return period of actual annual peak flows in these plots. The Weibull equation is provided:

$$\text{Weibull:} \qquad PP = \frac{100m}{n} \qquad \text{(eq. 5–10)}$$

where:

n = sample size

m = data rank

The basic computations for a discharge-frequency analysis are illustrated in example 3.

Application of log-Pearson frequency distribution

Numerous statistical distributions that can provide a fit of annual peak flow data exist. The hydrology committee of the WRC (WRC 1981) recommended the use of the log-Pearson type III distribution because it provided the most consistent fit of peak flow data. NRCS participated on the hydrology committee and has adopted the use of WRC Bulletin 17B for determining flood flow frequency, using measured streamflow data.

Several computer programs and Microsoft® Excel® spreadsheet programs exist that can be used to perform log-Pearson frequency analysis. A spreadsheet example is used in many of the examples in this chapter.

Table 5–2 K-values for the Gumbel extreme value distribution

Sample size	Return period, T (yr)								
	1.11	1.25	2.00	2.33	5	10	25	50	100
15	–1.34	–0.98	–0.15	0.06	0.97	1.70	2.63	3.32	4.01
20	–1.29	–0.95	–0.15	0.05	0.91	1.63	2.52	3.18	3.84
25	–1.26	–0.93	–0.15	0.04	0.89	1.58	2.44	3.09	3.73
30	–1.24	–0.91	–0.16	0.04	0.87	1.54	2.39	3.03	3.65
40	–1.21	–0.90	–0.16	0.03	0.84	1.50	2.33	2.94	3.55
50	–1.20	–0.88	–0.16	0.03	0.82	1.47	2.28	2.89	3.49
60	–1.18	–0.87	–0.16	0.02	0.81	1.45	2.25	2.85	3.45
70	–1.17	–0.87	–0.16	0.02	0.80	1.43	2.23	2.82	3.41
80	–1.16	–0.86	–0.16	0.02	0.79	1.42	2.21	2.80	3.39
100	–1.15	–0.85	–0.16	0.02	0.77	1.40	2.19	2.77	3.35
200	–1.11	–0.82	–0.16	0.01	0.74	1.33	2.08	2.63	3.18
400	–1.07	–0.80	–0.16	0.00	0.70	1.27	1.99	2.52	3.05

Table 5–3 K-values for the log-Pearson type III distribution

Skew coefficient C_S	Recurrence interval/percent chance of occurrence								
	1.0526 **95**	**1.25** **80**	**2** **50**	**5** **20**	**10** **10**	**25** **4**	**50** **2**	**100** **1**	**200** **0.5**
–2.00	–1.996	–0.609	0.307	0.777	0.895	0.959	0.980	0.990	0.995
–1.90	–1.989	–0.627	0.294	0.788	0.920	0.996	1.023	1.307	1.044
–1.80	–1.981	–0.643	0.282	0.799	0.945	1.035	1.069	1.087	1.097
–1.70	–1.972	–0.660	0.268	0.808	0.970	1.075	1.116	1.140	1.155
–1.60	–1.962	–0.675	0.254	0.817	0.994	1.116	1.166	1.197	1.216
–1.50	–1.951	–0.690	0.240	0.825	1.018	1.157	1.217	1.256	1.282
–1.40	–1.938	–0.705	0.225	0.832	1.041	1.198	1.270	1.318	1.351
–1.30	–1.925	–0.719	0.210	0.838	1.064	1.240	1.324	1.383	1.424
–1.20	–1.910	–0.732	0.195	0.844	1.086	1.282	1.379	1.449	1.501
–1.10	–1.894	–0.745	0.180	0.848	1.107	1.324	1.435	1.518	1.581
–1.00	–1.877	–0.758	0.164	0.852	1.128	1.366	1.492	1.588	1.664
–0.90	–1.858	–0.769	0.148	0.854	1.147	1.407	1.549	1.660	1.749
–0.80	–1.839	–0.780	0.132	0.856	1.166	1.448	1.606	1.733	1.837
–0.70	–1.819	–0.790	0.116	0.857	1.183	1.488	1.663	1.806	1.926
–0.60	–1.797	–0.800	0.099	0.857	1.200	1.528	1.720	1.880	2.016
–0.50	–1.774	–0.808	0.083	0.856	1.216	1.567	1.777	1.955	2.108
–0.40	–1.750	–0.816	0.066	0.855	1.231	1.606	1.834	2.029	2.201
–0.30	–1.726	–0.824	0.050	0.853	1.245	1.643	1.890	2.104	2.294
–0.20	–1.700	–0.830	0.033	0.850	1.258	1.680	1.945	2.178	2.388
–0.10	–1.673	–0.836	0.017	0.846	1.270	1.716	2.000	2.252	2.482
0.00	–1.645	–0.842	0.000	0.842	1.282	1.751	2.054	2.326	2.576
0.10	–1.616	–0.846	–0.017	0.836	1.292	1.785	2.107	2.400	2.670
0.20	–1.586	–0.850	–0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.30	–1.555	–0.853	–0.050	0.824	1.309	1.849	2.211	2.544	2.856
0.40	–1.524	–0.855	–0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.50	–1.491	–0.856	–0.083	0.808	1.323	1.910	2.311	2.686	3.041
0.60	–1.458	–0.857	–0.099	0.800	1.328	1.939	2.359	2.755	3.132

Table 5–3 K-values for the log-Pearson type III distribution—Continued

Skew coefficient C_S	Recurrence interval/percent chance of occurrence								
	1.0526 95	1.25 80	2 50	5 20	10 10	25 4	50 2	100 1	200 0.5
0.70	–1.423	–0.857	–0.116	0.790	1.333	1.967	2.407	2.824	3.223
0.80	–1.388	–0.856	–0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.90	–1.353	–0.854	–0.148	0.769	1.339	2.018	2.498	2.957	3.401
1.00	–1.317	–0.852	–0.164	0.758	1.340	2.043	2.542	3.022	3.489
1.10	–1.280	–0.848	–0.180	0.745	1.341	2.066	2.585	3.087	3.575
1.20	–1.243	–0.844	–0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.30	–1.206	–0.838	–0.210	0.719	1.339	2.108	2.666	3.211	3.745
1.40	–1.168	–0.832	–0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.50	–1.131	–0.825	–0.240	0.690	1.333	2.146	2.743	3.330	3.910
1.60	–1.093	–0.817	–0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.70	–1.056	–0.808	–0.268	0.660	1.324	2.179	2.815	3.444	4.069
1.80	–1.020	–0.799	–0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.90	–0.984	–0.788	–0.294	0.627	1.310	2.207	2.881	3.553	4.223
2.00	–0.949	–0.777	–0.307	0.609	1.302	2.219	2.912	3.605	4.398
2.10	–0.914	–0.765	–0.319	0.592	1.294	2.230	2.942	3.656	4.372
2.20	–0.882	–0.752	–0.330	0.574	1.284	2.240	2.970	3.705	4.444
2.30	–0.850	–0.739	–0.341	0.555	1.274	2.248	2.997	3.753	4.515
2.40	–0.819	–0.725	–0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.50	–0.790	–0.711	–0.360	0.518	1.250	2.262	3.048	3.845	4.652
2.60	–0.762	–0.696	–0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.70	–0.736	–0.681	–0.376	0.479	1.224	2.272	3.093	3.932	4.783
2.80	–0.711	–0.666	–0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.90	–0.688	–0.651	–0.390	0.440	1.195	2.277	3.134	4.013	4.909
3.00	–0.665	–0.636	–0.396	0.420	1.180	2.278	3.152	4.051	4.970

Example 3: Example computations of a discharge-frequency analysis

Problem: A streambank stabilization project is being designed for the Los Pinos River in the San Luis Valley of the Upper Rio Grande Basin. The project is less than a mile downstream from a USGS stream gage. The 167-square-mile watershed consists of forests, grass, and sage on a rural, primarily public land setting. This problem illustrates the analysis of the gaged flow data with the four common distributions.

Solution: First, the gage information is downloaded from the USGS (*http://waterdata.usgs.gov/nwis*), and the data are sorted and transformed. Then the basic statistics are calculated (table 5–4).

After computing these basic statistics, the distributions can be generated. Specifically, the magnitudes of the 1.25-, 2-, 5-, 10-, 25-, 50-, and 100-year events are computed and plotted with the source data. Example computation and plotting of distributions are shown in figure 5–1. Both the Gumbel and log-Pearson distributions fit the plotted data reasonably well.

Table 5–4 Discharge peaks, with basic statistics

USGS 08248000 Los Pinos River near Ortiz, CO

Year	Peak discharge (ft^3/s)	ln (peak discharge)	Rank	Weibull plotting position (yr)
1915	1,620	7.3902	27	3.1
1916	1,690	7.4325	22	3.8
1917	1,750	7.4674	18	4.7
1918	1,020	6.9276	57	1.5
1919	1,550	7.3460	30	2.8
1920	2,300	7.7407	5	16.8
1925	1,160	7.0562	50	1.7
1926	1,600	7.3778	29	2.9
1927	1,680	7.4265	23	3.7
1928	1,240	7.1229	47	1.8
1929	1,180	7.0733	48	1.8
1930	1,100	7.0031	51	1.6
1931	684	6.5280	72	1.2
1932	2,000	7.6009	10	8.4
1933	1,490	7.3065	35	2.4
1934	569	6.3439	75	1.1
1935	1,420	7.2584	40	2.1
1936	1,640	7.4025	26	3.2
1937	2,770	7.9266	4	21.0
1938	2,270	7.7275	6	14.0
1939	1,360	7.2152	43	2.0
1940	887	6.7878	63	1.3
1941	3,160	8.0583	1	84.0
1942	2,000	7.6009	10	8.4
1943	1,370	7.2226	42	2.0
1944	3,030	8.0163	2	42.0
1945	2,180	7.6871	7	12.0
1946	1,090	6.9939	54	1.6
1947	1,740	7.4616	19	4.4
1948	1,660	7.4146	24	3.5
1949	1,620	7.3902	27	3.1
1950	876	6.7754	65	1.3
1951	563	6.3333	77	1.1
1952	2,790	7.9338	3	28.0
1953	924	6.8287	62	1.4
1954	882	6.7822	64	1.3
1955	700	6.5511	71	1.2
1956	926	6.8309	61	1.4
1957	1,850	7.5229	14	6.0
1958	1,490	7.3065	35	2.4
1959	646	6.4708	73	1.2
1960	1,100	7.0031	51	1.6
1961	1,420	7.2584	40	2.1
1962	1,480	7.2998	37	2.3
1963	532	6.2766	78	1.1
1964	1,000	6.9078	60	1.4

Table 5–4 Discharge peaks, with basic statistics—Continued

Year	Peak discharge (ft^3/s)	ln [peak discharge]	Rank	Weibull Plotting Position (yr)
1965	2,000	7.6009	10	8.4
1966	1,010	6.9177	59	1.4
1967	755	6.6267	70	1.2
1968	1,340	7.2004	44	1.9
1969	1,180	7.0733	48	1.8
1970	1,500	7.3132	34	2.5
1971	488	6.1903	81	1.0
1972	385	5.9532	82	1.0
1973	1,940	7.5704	13	6.5
1974	841	6.7346	68	1.2
1975	2,020	7.6109	8	10.5
1976	1,060	6.9660	55	1.5
1977	379	5.9375	83	1.0
1978	1,050	6.9565	56	1.5
1979	1,810	7.5011	15	5.6
1980	1,660	7.4146	24	3.5
1981	580	6.3630	74	1.1
1982	1,530	7.3330	32	2.6
1983	1,700	7.4384	21	4.0
1984	1,790	7.4900	16	5.3
1985	2,020	7.6109	8	10.5
1986	1,710	7.4442	20	4.2
1987	1,430	7.2654	39	2.2
1988	501	6.2166	80	1.1
1989	860	6.7569	66	1.3
1990	564	6.3351	76	1.1
1991	1,470	7.2930	38	2.2
1992	845	6.7393	67	1.3
1993	1,780	7.4844	17	4.9
1994	1,540	7.3395	31	2.7
1995	1,510	7.3199	33	2.5
1996	840	6.7334	69	1.2
1997	1,340	7.2004	44	1.9
1998	1,100	7.0031	51	1.6
1999	1,020	6.9276	57	1.5
2000	516	6.2461	79	1.1
2001	1,300	7.1701	46	1.8

For Q: average = 1,366
standard deviation = 598
skew coefficient = 0.660

For lnQ: average = 7.1165
standard deviation = 0.4763
skew coefficient = -0.507

For lnQ: average = 7.1165
standard deviation = 0.4763
skew coefficient = -0.507

Figure 5–1 Plotting distributions for return period peak discharges

For the normal distribution, equation 5–6 is used to compute the peaks. But first, a table of areas under the standard normal curve (found in most statistics books) is used to determine the K_N and K_{LN} values.

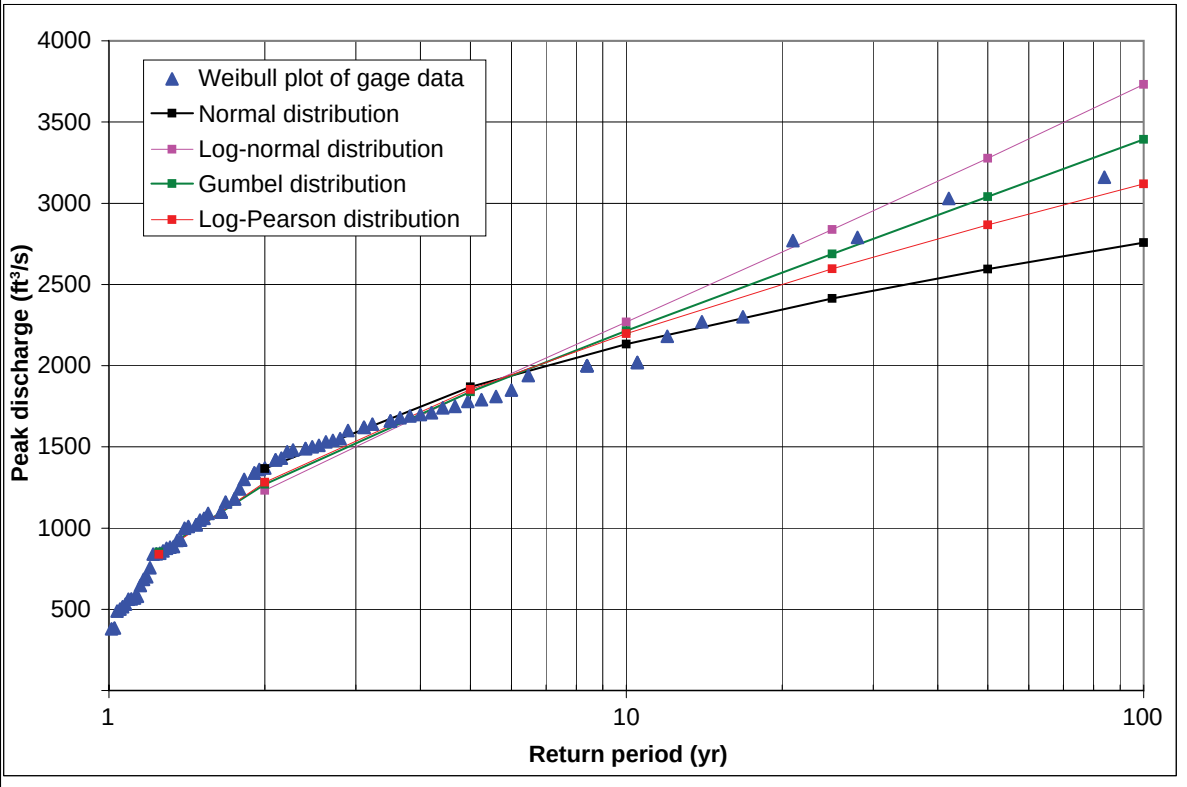
Using the equation $K_{N,T} = K_{LN,T} = 0.50¢–1/T$, the following table is populated:

Return Period	1.25	2.00	5.00	10.00	25.00	50.00	100.00
K_N & K_{LN}	----	0.00	0.84	1.28	1.75	2.05	2.33

With these K-values and addiing the K-values for the Gumbel and Log-Pearson distributions, the following table is generated, using equations 5–6 through 5–7.

Q: average:	1366	**ln Q:** ıverage:	7.12
standard deviation:	598	standard deviation:	0.48
skew coefficient:	0.66	skew coefficient:	-0.51

Method		Return Period						
		1.25	2	5	10	25	50	100
Normal Distribution	K_N	----	0.00	0.84	1.28	1.75	2.05	2.33
	Q_N (ft³/s)	----	1,366	1,869	2,132	2,413	2,594	2,757
Log-normal distribution	K_{LN}	----	0.00	0.84	1.28	1.75	2.05	2.33
	Q_{LN}	----	7.12	7.52	7.73	7.95	8.09	8.22
	Q_{LN} (ft³/s)	----	1,232	1,840	2,269	2,837	3,277	3,732
Gumbel distribution	K_G	-0.86	-0.16	0.79	1.42	2.21	2.80	3.39
	Q_G (ft³/s)	852	1,270	1,838	2,215	2,688	3,040	3,393
Log-Pearson distribution	K_{LP}	-0.81	0.08	0.86	1.21	1.56	1.77	1.95
	Q_{LP}	6.73	7.16	7.52	7.70	7.86	7.96	8.05
	Q_{LP} (ft³/s)	839	1,282	1,852	2,198	2,596	2,867	3,119



General log-Pearson distribution

This distribution was provided previously as equation 5–9. The average, standard deviation, and skew coefficient were defined by equations 5–5 through 5–7. A complete table of K-values, with skews from –9.0 to +9.0, can be obtained from appendix 3 of WRC Bulletin 17B.

Generalized skew and weighting the skew coefficient

The computed station skew is sensitive to large events, especially with short periods of records. This problem can be minimized by weighting the station skew with a generalized skew that takes into account skews from neighboring gaged watersheds.

Three methods to develop this generalized skew are to:

- develop a skew isoline map
- develop a skew regression (or prediction) equation
- compute the mean and variance of the skew coefficients

These methods should incorporate at least 40 stations with at least 25 years of record within the gage of interest's hydro-physiographic province. Plate 1 of WRC Bulletin 17B could also be used; but, due to the vintage of this compilation, a detailed study may be preferred.

To develop a skew isoline map, the station skews are plotted at the centroid of the watershed and trends are observed. A regression or prediction equation can also be developed to relate skews to watershed and climatologic characteristics. If no relationship can be found with the isoline or regression approach, the arithmetic mean ($\overline{X}$) can be simply computed and used as the generalized skew. Care needs to be taken to ensure that all of the gages are in a similar hydro-physiographic province.

Once the best generalized skew is computed, a weighted skew is computed for the log-Pearson analysis using equation. 5–11.

$$G_W = \frac{S^2_{\overline{G}}(G) + S^2_G(\overline{G})}{S^2_{\overline{G}} + S^2_G} \qquad \text{(eq. 5–11)}$$

where:

G_W = weighted skew coefficient
G = station skew
$\overline{G}$ = generalized skew
$S^2_{\overline{G}}$ = variance (mean square error) of generalized skew
S^2_G = variance of station skew

When generalized skews are read from Plate 1 of WRC Bulletin 17B, a variance of 0.302 should be used in equation 5–11.

The variance of the logarithmic station skew is a function of record length and population skew. This variance can be approximated with equations 5–12, 5–13, and 5–14.

$$S^2_G = 10^{\left[A - B\left(\log_{10}\left(\frac{n}{10}\right)\right)\right]} \qquad \text{(eq. 5–12)}$$

$$A = -0.33 + 0.08|G| \quad \text{if } |G| \le 0.90$$
$$A = -0.52 + 0.30|G| \quad \text{if } |G| > 0.90 \qquad \text{(eq. 5–13)}$$
$$B = 0.94 - 0.26|G| \quad \text{if } |G| \le 1.50$$

$$B = 0.55 \quad \text{if } |G| > 1.50 \qquad \text{(eq. 5–14)}$$

where:

n = record length in years
$|G|$ = absolute value of the station skew

If an historic record adjustment has been made. Historically adjusted values should be used.

Broken or incomplete records

Annual peaks for certain years at a gage are often missing. If this happens, the two or more record lengths are analyzed as a continuous record, with a record length equal to the sum of individual records.

Incomplete records refer to a high or low streamflow record that is missing due to a gaging failure. Usually, the gaging agency uses an indirect flow estimate to fill this void. If this has not occurred, effort to fill this gap may be warranted.

Historic flood data

As described in reliability of flow estimates, high flow values and historic events can be overestimated. If historic data are judged not to be biased high, WRC Bulletin 17B provides a special procedure for dealing

with these events, instead of using a broken record approach. This method assumes that data from the systematic record is representative of the period between the historic data, and the systematic record and its statistics are adjusted accordingly.

First, a systematic record weight is computed.

$$W_s = \frac{H - Z}{n + L} \qquad \text{(eq. 5–15)}$$

where:
W_s = systematic record weight
H = historical period
Z = number of historic peaks
n = systematic record length
L = number of low values excluded, including low outliers and zero flow years

The historically adjusted average ($\tilde{X}$) is computed using:

$$\tilde{X} = \frac{W_s \sum X_s + \sum X_h}{H - W_s L} \qquad \text{(eq. 5–16)}$$

where:
X_s = logarithmic systematic record peaks
X_h = logarithmic historic record peaks

The historically adjusted standard deviation of logarithms ($\tilde{S}$) is:

$$\tilde{S} = \sqrt{\tilde{S}^2} = \sqrt{\frac{W_s \sum \left(X - \tilde{X}\right)^2 + \sum \left(X_h - \tilde{X}\right)^2}{\left(H - W_s L - 1\right)}} \qquad \text{(eq. 5–17)}$$

The historically adjusted skew coefficient of logarithms ($\tilde{G}$) is:

$$\tilde{G} = \frac{H - W_s L - 1}{\left(H - W_s L - 1\right)\left(H - W_s L - 2\right)} \left[\frac{W_s \sum \left(X - \tilde{X}\right)^3 + \sum \left(X_h - \tilde{X}\right)^3}{\tilde{S}^3} \right] \qquad \text{(eq. 5–18)}$$

Outliers

Outliers are data points that depart significantly from the trend of the remaining data. Including such outliers may be inappropriate in a frequency analysis. The decision to retain or eliminate an outlier is based on both hydrologic and statistical considerations. The statistical method for identifying possible outliers, as presented in WRC Bulletin 17B, uses equations 5–19 and 5–20:

$$X_H = \bar{X}_1 + K_o S_1 \qquad \text{(eq. 5–19)}$$

$$X_L = \bar{X}_1 - K_o S_1 \qquad \text{(eq. 5–20)}$$

where:
X_H = high outlier threshold, in logarithm units
X_L = low outlier threshold, in logarithm units (no historic adjustment)
$\bar{X}_1$ = average of annual peak discharge logarithms
K_o = based on sample size n, as listed in table 5–5
S_1 = standard deviation of logarithms of annual peak discharge

If the station skew is greater than +0.4, high outliers are considered first and possibly eliminated. If the station skew is less than –0.4, low outliers are considered first, and then possibly eliminated. When the skew is between ± 0.4, a test for both high and low outliers should be first applied before possibly eliminating any outliers from the data set.

If an adjustment for historic flood data has already been made, the low outlier threshold equation is modified in the form:

$$X_{L,H} = \tilde{X} - K_o \tilde{S} \qquad \text{(eq. 5–21)}$$

where:
$X_{L,H}$ = low outlier threshold, in logarithm units (with historic adjustment)
$\tilde{X}$ = historically adjusted mean logarithm
$\tilde{S}$ = historically adjusted standard deviation

Mixed populations

In many watersheds, annual peak flows are caused by different types of events such as snowmelt, tropical cyclones, and summer thunderstorms. Including all types of events in a single frequency analysis may result in large and inappropriate skew coefficients. For such situations, special treatment may be warranted. Specifically, peak flows can be segregated by cause, analyzed separately, and then combined. Importantly, separation by calendar date alone is not appropriate, unless it can be well documented that an event type always varies by time of year.

Zero flow years

Some streams in arid regions may have no flow during the entire water year, thus having one or more zero peak flow values in its record. Such situations require special treatment. See appendix 5 in WRC Bulletin 17B

Table 5–5 Outlier test K_o values, from WRC Bulletin 17B

Sample size	K_o value	Sample size	K_o value	Sample size	K_o value	Sample size	K_o value
10	2.036	45	2.727	80	2.940	115	3.064
11	2.088	46	2.736	81	2.945	116	3.067
12	2.134	47	2.744	82	2.949	117	3.070
13	2.175	48	2.753	83	2.953	118	3.073
14	2.213	49	2.760	84	2.957	119	3.075
15	2.247	50	2.768	85	2.961	120	3.078
16	2.279	51	2.775	86	2.966	121	3.081
17	2.309	52	2.783	87	2.970	122	3.083
18	2.335	53	2.790	88	2.973	123	3.086
19	2.361	54	2.798	89	2.977	124	3.089
20	2.385	55	2.804	90	2.981	125	3.092
21	2.408	56	2.811	91	2.984	126	3.095
22	2.429	57	2.818	92	2.989	127	3.097
23	2.448	58	2.824	93	2.993	128	3.100
24	2.467	59	2.831	94	2.996	129	3.102
25	2.486	60	2.837	95	3.000	130	3.104
26	2.502	61	2.842	96	3.003	131	3.107
27	2.519	62	2.849	97	3.006	132	3.109
28	2.534	63	2.854	98	3.011	133	3.112
29	2.549	64	2.860	99	3.014	134	3.114
30	2.563	65	2.866	100	3.017	135	3.116
31	2.577	66	2.871	101	3.021	136	3.119
32	2.591	67	2.877	102	3.024	137	3.122
33	2.604	68	2.883	103	3.027	138	3.124
34	2.616	69	2.888	104	3.030	139	3.126
35	2.628	70	2.893	105	3.033	140	3.129
36	2.639	71	2.897	106	3.037	141	3.131
37	2.650	72	2.903	107	3.040	142	3.133
38	2.661	73	2.908	108	3.043	143	3.135
39	2.671	74	2.912	109	3.046	144	3.138
40	2.682	75	2.917	110	3.049	145	3.140
41	2.692	76	2.922	111	3.052	146	3.142
42	2.700	77	2.927	112	3.055	147	3.144
43	2.710	78	2.931	113	3.058	148	3.146
44	2.719	79	2.935	114	3.061	149	3.148

for specific details on how to account for zero flow years.

Confidence limits

A frequency curve is not an exact representation of the population curve. How well a stream record predicts flooding depends on record length, accuracy, and applicability of the underlying probability distribution. Statistical analysis allows the advantage of calculating confidence limits, which provide a measure of the uncertainty or spread in an estimate. These limits are a measure of the uncertainty of the discharge at a selected exceedance probability. For example, for the 5 percent and 95 percent confidence limit curves, there are nine chances in ten that the true value lies in the 90 percent confidence interval between the curves. As more data become available at a stream gage, the confidence limits will normally be narrowed. As presented in WRC Bulletin 17B, the following method is provided to develop confidence limits for a log-Pearson type III distribution.

$$X_{CI,U} = \bar{X}_l + S_l\left(K_{CI,U}\right) \qquad \text{(eq. 5–22)}$$

$$X_{CI,L} = \bar{X}_l + S_l\left(K_{CI,L}\right) \qquad \text{(eq. 5–23)}$$

where:

$X_{CI,U}$ = logarithmic upper confidence limit
$X_{CI,L}$ = logarithmic lower confidence limit
$\bar{X}_l$ = logarithmic peak flow mean
S_l = logarithmic peak flow standard deviation

$$K_{CI,U} = \frac{K_{LP,T} + \sqrt{K_{LP,T}^2 - ab}}{a} \qquad \text{(eq. 5–24)}$$

$$K_{CI,L} = \frac{K_{LP,T} - \sqrt{K_{LP,T}^2 - ab}}{a} \qquad \text{(eq. 5–25)}$$

$$a = 1 - \frac{z_c^2}{2(n-1)} \qquad \text{(eq. 5–26)}$$

$$b = K_{LP,T}^2 - \frac{z_c^2}{n} \qquad \text{(eq. 5–27)}$$

where:

n = record length
$K_{LP,T}$ = as listed in table 5–3, as a function of return period and skew coefficient
z_c = standard normal deviate, that is, the zero-skew $K_{LP,T}$ value at a return period of 1 decimal confidence limit. For the 95 percent confidence limit (0.05), z_c = 1.64485.

Comparisons of the frequency curve

Comparisons of distributions between the watershed being investigated and other regional watersheds can be useful for error checking and to identify possible violations of the underlying assumptions for the analysis. This can be especially illuminating for gages in the same watershed, upstream and downstream of the gage of interest, possibly identifying particular and unexpected hydrologic phenomena.

Discharge estimates from precipitation can be a helpful complement to gage data. However, such discharge estimates require a valid rainfall-runoff model. Such models are best when calibrated, which requires gage information. Such a calibrated model can be useful at other points within the watershed.

Example 4 illustrates analysis for outliers and confidence intervals.

Computation resources for flow frequency analysis

Several computer programs are available for assistance in performing the flood frequency analysis including the U.S. Army Corps of Engineers HEC–FFA (USACE 1992b) or the USGS PEAKFQ (USGS 1998). The USGS provides a computer program, PEAKFQ, at the Web site:

http://water.usgs.gov/software/peakfq.html

Spreadsheet programs have also been used to perform flood frequency analysis calculations as detailed by WRC Bulletin 17B. One of these spreadsheets is used in the examples presented in this chapter. This spreadsheet includes algorithms for generalized skew, 90 percent confidence intervals (95 percent confidence limits), historic data inclusion, and outlier identification. Example output sheets of this spreadsheet are provided in the example 5. It should be noted that these computational aids are for unregulated rivers and streams and that special precautions are necessary when evaluating flood frequencies on rivers with dams and significant diversions.

Transfer methods

Peak discharge frequency values are often needed at watershed locations other than the gaged location. Peak discharges may be extrapolated upstream or downstream from stream gages, for which frequency curves have been determined. In addition, peak dis-

Example 4: Confidence interval and outlier example

Problem: This example illustrates the analysis of USGS 06324500 Powder River at Moorhead, Montana, gage data in Eastern Montana (table 5–6). The following questions are addressed:

- frequency distribution for the gage
- 90 percent confidence interval
- outlier check
- impacts of the use of historic methodology and the impacts of inclusion of any outliers are assessed

Solution: The peak streamflow data were downloaded from the USGS NWIS data system (*http://waterdata.usgs.gov/nwis/*). These data, along with basic statistical computations, are provided in table 5–6.

Inspect the comments accompanying the peak flow data. For this data set, two of the data points are daily averages (instead of peak flows), six data points are estimates, and one estimate is an historic peak. The event on March, 17, 1979, is missing the peak flow (though the gage record does include the associated stage). Effort should be made to populate this point, but for this exercise, the data point is ignored.

It can be useful to plot frequency data to help in the identification of outliers and trends. Figure 5–2 includes a plot of these data.

This plot clearly shows a possible outlier and also qualitatively indicates a possible downward trend during the second half of the data set. A step trend would be more evident in the case of greater reservoir regulation within this watershed.

To assess the impact of using the historic methodology and inclusion or exclusion of the possible outlier, several frequency analyses need to be computed. A frequency analysis is performed on all of the data. Equation 5–9 is used to compute the distribution and confidence limits area, using equations 5–22 and 5–23. The computations and results are provided in table 5–6. The data are plotted in figure 5–3.

Outliers are identified using equations 5–5, 5–19, and 5–20, and the Weibull plotting positions are computed using equation 5–10. The high and low outlier thresholds are 10.96 and 6.48, respectively. Since the skew is between +0.4, high and low outliers are checked at the same time. The identification of outliers and computation of plotting position are shown in table 5–7. An outlier identified by the WRC Bulletin 17B methodology has been highlighted in yellow.

Since the 1923 event has been identified as a possible outlier, a frequency analysis is performed on a data set that excludes this high-flow value. The results of this computation are provided in table 5–8.

WRC Bulletin 17B provides a special methodology for historic peaks. The basis of this method is the assumption that data from the systematic record is representative of the period of the historic data. The systematic record and statistics are adjusted accordingly. This method is applied to the entire record of the Powder River at Moorhead gage, with computations that use equations 5–15 to 5–18 and those results are provided in table 5–9. Results have been plotted in figure 5–3.

Example 4: Confidence interval and outlier example—Continued

Inspection of the plotted results reveals a number of characteristics in the frequency distributions, specifically:

- The frequency analysis that includes the 1923 outlier in its computations (but does not incorporate the historic methodology) has the highest frequency distribution estimate. With the exception of the outlier, it also matches the higher data well. The 90 percent confidence interval brackets the higher data, with the exception of the outlier. This distribution does somewhat overestimate lower frequency events.
- The frequency analysis based on the historic methodology also well represents the higher data and somewhat overestimates lower data. This historic distribution is slightly lower than the nonhistoric distribution.
- The frequency analysis that excludes the outlier from its computations provides a distribution that is much lower than the distributions that include the outlier point. This distribution does not represent the higher flow data well—its 90 percent confidence interval excludes two additional data points, as plotted using the Weibull methodology. It does represent the lower peak data better.

It can be concluded from these observations that inclusion of the high outlier likely best represents the less frequent (higher) events. Exclusion of the data point provides a distribution that better represents more frequent (lower) events. For the Powder River at Moorhead, Montana, gaging station, it may be best to use the distribution that best represents the frequency of a desired event. If one distribution is required for all frequencies, the inclusion of the outlier using the historic methodology is likely best, due to its slightly better representation of all data than the nonhistoric, included outlier computation.

In addition, with the 1923 outlier perhaps being biased high, it may be best to revisit the computation of this historic peak. Additionally, it may be prudent to incorporate the generalized skew procedure to counteract any bias in the skew of the gage data.

Example 5: Log-Pearson spreadsheet frequency analysis example

The frequency analysis for the USGS gage 08251500 Rio Grande near Labatos, CO, is required for a stream stabilization project. The distribution was computed using a log-Pearson spreadsheet. The output sheets are provided in figures 5–4 to 5–6.

Visual observation of the graph of the plotted data indicates the computed record should be accepted for the analysis.

Table 5–6 Peak streamflow data at gage 06324500 Powder River at Moorhead, MT

Date	Peak flow (ft^3/s)	Notes	Date	Peak flow (ft^3/s)	Notes	Date	Peak flow (ft^3/s)	Notes
09/30/1923	100,000	2,3	06/15/1953	8,590		05/27/1980	2,210	
06/03/1929	8,610		08/06/1954	9,740		05/31/1981	2,160	
07/14/1930	4,040		06/18/1955	5,610		07/26/1982	6,350	
05/06/1931	6,040		06/16/1956	7,200		06/13/1983	2,870	
06/08/1932	3,550		06/07/1957	5,600		05/19/1984	4,620	
08/30/1933	14,800		06/12/1958	4,900		07/31/1985	1,410	
06/16/1934	1,920		03/19/1959	5,740		06/09/1986	4,540	
06/01/1935	8,140		03/20/1960	6,200		07/18/1987	11,400	
03/02/1936	9,240		05/30/1961	1,320		05/19/1988	1,990	
07/14/1937	14,500		06/17/1962	23,000		03/12/1989	800	1,2
05/30/1938	5,720		06/15/1963	7,010		08/21/1990	8,150	
06/02/1939	7,200		06/24/1964	15,000		06/04/1991	5,460	
06/04/1940	6,820		04/02/1965	18,300		11/12/1991	6,410	
08/13/1941	8,360		03/13/1966	4,000		06/09/1993	6,740	
06/26/1942	5,070		06/17/1967	17,300		07/09/1994	3,920	
03/26/1943	8,800		06/08/1968	8,580	2	05/11/1995	8,250	
05/20/1944	10,700		07/16/1969	5,280		03/13/1996	3,500	1,2
06/06/1945	6,190		05/24/1970	8,900	2	06/10/1997	4,290	
06/11/1946	5,720		06/01/1971	8,340		07/04/1998	2,760	
03/19/1947	9,300	2	02/29/1972	7,800		05/04/1999	3,960	
06/17/1948	9,320		06/19/1975	12,100		05/20/2000	3,930	
03/06/1949	9,360		06/23/1976	5,370		07/13/2001	1,490	
05/19/1950	2,620		05/17/1977	4,750				
09/09/1951	2,020		05/20/1978	33,000				
03/25/1952	15,300		03/17/1979					

Notes:
1/ Discharge is a maximum daily average
2/ Discharge is an estimate
3/ Discharge is an historic peak

Figure 5–2 Data plot at gage 06324500 Powder River at Moorhead, MT

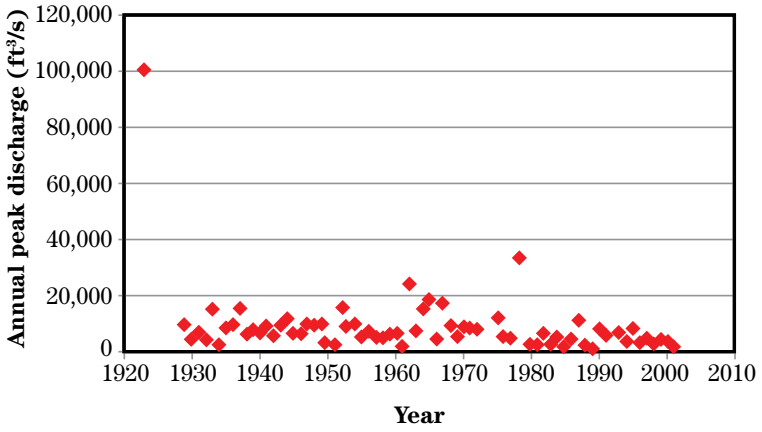


Figure 5–3 Data and frequency plots

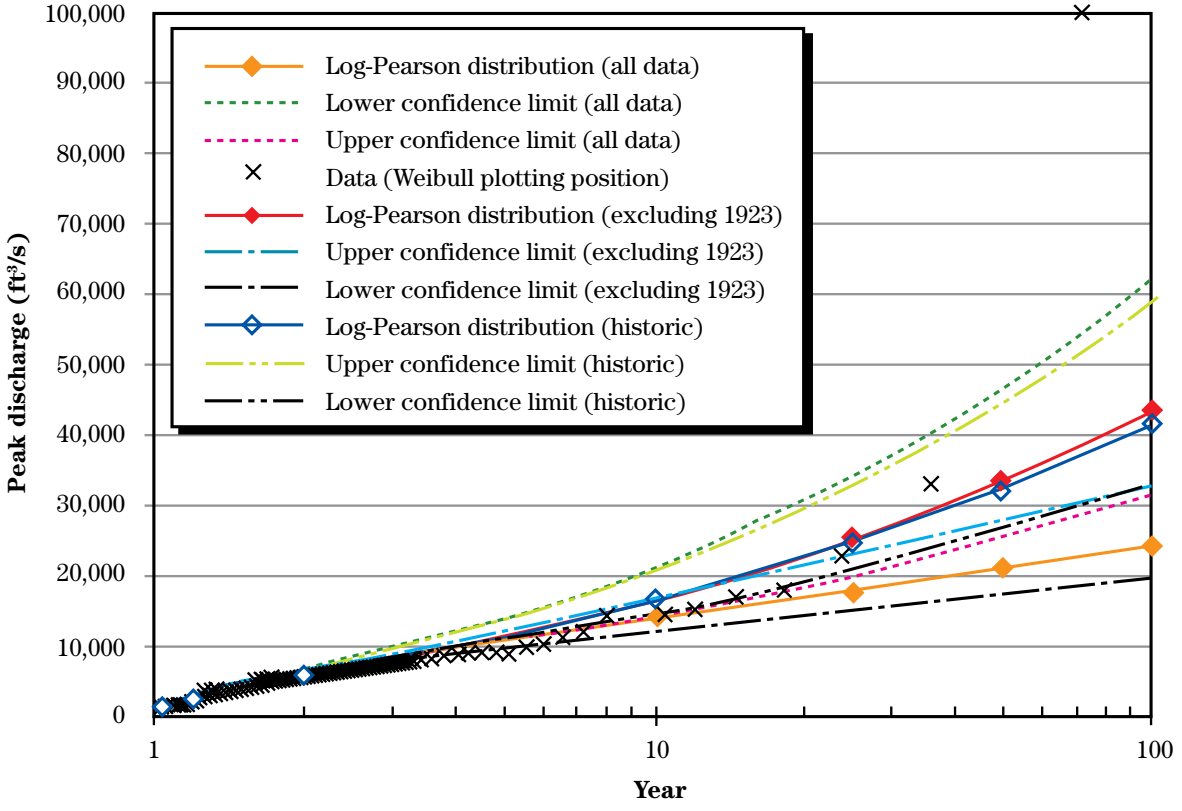


Table 5–7 Logarithmic data and Weibull plotting position values

Year	ln(Q)	Rank	Weibull (yr)
1923	11.513	1	72.0
1929	9.061	20	3.6
1930	8.304	53	1.4
1931	8.706	38	1.9
1932	8.175	58	1.2
1933	9.602	8	9.0
1934	7.560	67	1.1
1935	9.005	27	2.7
1936	9.131	17	4.2
1937	9.582	9	8.0
1938	8.652	40	1.8
1939	8.882	29	2.5
1940	8.828	32	2.3
1941	9.031	23	3.1
1942	8.531	47	1.5
1943	9.083	19	3.8
1944	9.278	12	6.0
1945	8.731	37	1.9
1946	8.652	41	1.8
1947	9.138	16	4.5
1948	9.140	15	4.8
1949	9.144	14	5.1
1950	7.871	62	1.2
1951	7.611	65	1.1
1952	9.636	6	12.0
1953	9.058	21	3.4
1954	9.184	13	5.5
1955	8.632	42	1.7
1956	8.882	30	2.4
1957	8.631	43	1.7
1958	8.497	48	1.5
1959	8.655	39	1.8
1960	8.732	36	2.0
1961	7.185	70	1.0
1962	10.043	3	24.0
1963	8.855	31	2.3
1964	9.616	7	10.3
1965	9.815	4	18.0
1966	8.294	54	1.3
1967	9.758	5	14.4
1968	9.057	22	3.3
1969	8.572	46	1.6
1970	9.094	18	4.0
1971	9.029	24	3.0
1972	8.962	28	2.6
1975	9.401	10	7.2
1976	8.589	45	1.6
1977	8.466	49	1.5
1978	10.404	2	36.0
1980	7.701	63	1.1
1981	7.678	64	1.1
1982	8.756	35	2.1
1983	7.962	60	1.2
1984	8.438	50	1.4
1985	7.251	69	1.0
1986	8.421	51	1.4
1987	9.341	11	6.5
1988	7.596	66	1.1
1989	6.685	71	1.0
1990	9.006	26	2.8
1991	8.605	44	1.6
1992	8.766	34	2.1
1993	8.816	33	2.2
1994	8.274	57	1.3
1995	9.018	25	2.9
1996	8.161	59	1.2
1997	8.364	52	1.4
1998	7.923	61	1.2
1999	8.284	55	1.3
2000	8.276	56	1.3
2001	7.307	68	1.1

Table 5–8 Frequency analysis data at gage 06324500 Powder River at Moorhead, MT

The basic statistics of the peak flow natural logarithms are provided:

Average =	8.7167	n =	71	
Standard deviation =	0.7722	z_c =	1.64485	(95% confidence limit)
Skew =	0.2883	a =	0.980674775	

Next, the log-Pearson K-Values are extracted from table 5–2, and the frequency values and confidence limits are computed:

Return period	**1.0526**	**1.25**	**2**	**5**	**10**	**25**	**50**	**100**	**200**
K-value	-1.559	-0.853	-0.048	0.825	1.308	1.845	2.205	2.536	2.845
ln (discharge)	7.513	8.058	8.680	9.354	9.727	10.142	10.419	10.675	10.914
Discharge (ft^3/s)	1,832	3,160	5,882	11,539	16,760	25,379	33,500	43,245	54,922
b	2.391	0.689	-0.036	0.642	1.673	3.367	4.824	6.391	8.057
$K_{CI,U}$	-1.293	-0.638	0.148	1.070	1.604	2.209	2.618	2.995	3.350
$K_{CI,L}$	-1.885	-1.101	-0.246	0.612	1.063	1.554	1.879	2.176	2.452
$\ln(Q_{CI,U})$	7.718	8.224	8.831	9.543	9.956	10.423	10.738	11.030	11.304
$\ln(Q_{CI,L})$	7.261	7.867	8.527	9.189	9.538	9.917	10.168	10.397	10.610
QCI,U (ft^3/s)	2,248	3,729	6,844	13,947	21,071	33,614	46,084	61,684	81,115
QCI,L (ft^3/s)	1,423	2,609	5,047	9,790	13,873	20,268	26,043	32,751	40,551

Table 5–9 Historic methodology computations

H = 79 Z = 1 n = 70 L = 0

From equation 5–15: systematic record weight, Ws = 1.114
From equation 5–16: historically adjusted average, $\tilde{X}$ = 8.713
From equation 5–17: historically adjusted standard deviation, $\tilde{S}$ = 0.765
From equation 5–18: historically adjusted skew, $\tilde{G}$ = 0.239
z_c =1.64485 (95% confidence limit) a = 0.980674775

Next, the log-Pearson K-Values are extracted from Table 5–5–2 and the frequency values and confidence limits are computed:

Return period	**1.0526**	**1.25**	**2**	**5**	**10**	**25**	**50**	**100**	**200**
K value	-1.573991189	-0.851162143	-0.039585477	0.827675714	1.304099048	1.830008811	2.179143811	2.499891431	2.799026432
ln (discharge)	7.509	8.062	8.682	9.346	9.710	10.112	10.379	10.624	10.853
Discharge (ft^3/s)	1,824	3,171	5,898	11,448	16,480	24,639	32,180	41,126	51,697
b	2.439	0.686	-0.037	0.646	1.662	3.310	4.710	6.211	7.796
$K_{CI,U}$	-1.306	-0.636	0.158	1.075	1.601	2.193	2.589	2.955	3.298
KCI,L	-1.904	-1.100	-0.239	0.613	1.059	1.539	1.855	2.143	2.411
ln(QCI,U)	7.714	8.227	8.834	9.534	9.937	10.389	10.693	10.973	11.235
ln(QCI,L)	7.257	7.871	8.530	9.182	9.522	9.890	10.131	10.351	10.556
QCI,U (ft^3/s)	2,239	3,739	6,861	13,827	20,682	32,516	44,036	58,261	75,707
QCI,L (ft^3/s)	1,418	2,621	5,064	9,718	13,659	19,730	25,113	31,302	38,409

Figure 5–4 Sheet 1 of log-Pearson spreadsheet output for USGS gage 0825150

OUTPUT TABLES (The spreadsheet is configured so that only the area in these boxes will be printed.)

NRCS Log-Pearson Frequency Analysis Spreadsheet, Version 2.1, 5/2003 *Page 1 of 3*

Project: *Example 5-5-3*
Streamgage: *# USGS 08251500 RIO GRANDE NEAR LOBATOS, CO.*
Date: *########* **Performed By:** *Steve Yochum; Hydrologist, Northern Plains Engineering Team*

Without Generalized Skew

Average:	7.8440
Standard Deviation:	0.95782872
Skew Coefficient(1):	-0.3948897
Length of systematic record:	102
Number of historic peaks:	0
Length of Data Record:	102
Length of Historic Record:(5)	----

With Generalized Skew

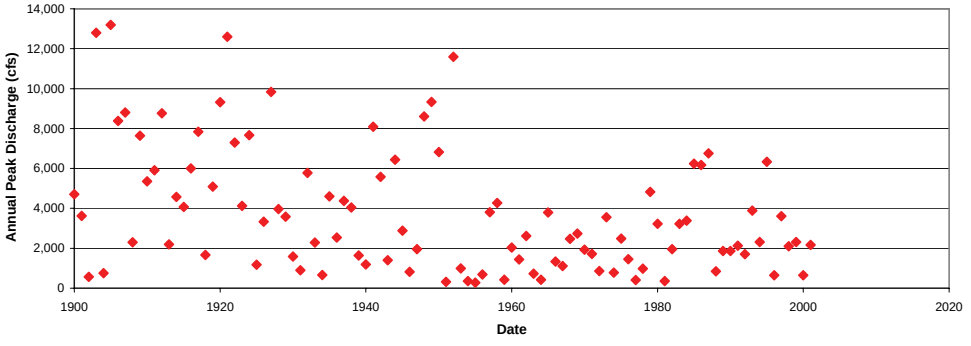
Generalized Skew Coefficient(3):	-0.0519
Variance of Generalized Skew(3):	0.2408
A:	-0.298409
B:	0.837329
station skew:	-0.394890
MSE Station Skew:	0.07195506
Weighted skew coefficient(1):	-0.3159687

Recurrence Interval(2) (years)	Percent Chance	K-Value	Ln(Q)	Peak(4) Discharge (cfs)	90% Confidence Interval Upper (cfs)	Lower (cfs)
200	0.5	2.206	9.9567	21,100	29,100	16,200
100	1	2.033	9.7911	17,900	24,200	13,900
50	2	1.837	9.6034	14,800	19,700	11,700
25	4	1.608	9.3841	11,900	15,400	9,560
10	10	1.232	9.0238	8,300	10,400	6,840
5	20	0.855	8.6628	5,780	7,030	4,870
2	50	0.065	7.9064	2,710	3,180	2,320
1.25	80	-0.816	7.0620	1,170	1,380	962
1.05	95	-1.749	6.1690	478	601	363
200	0.5	2.279	10.0270	22,600	31,500	17,300
100	1	2.092	9.8478	18,900	25,800	14,700
50	2	1.881	9.6457	15,500	20,600	12,200
25	4	1.637	9.4120	12,200	15,900	9,810
10	10	1.243	9.0343	8,390	10,500	6,910
5	20	0.853	8.6613	5,780	7,020	4,860
2	50	0.053	7.8943	2,680	3,140	2,290
1.25	80	-0.823	7.0560	1,160	1,380	956
1.05	95	-1.730	6.1871	486	611	370

(1) Station and generalized skews must be between -2.00 and +3.00 in this spreadsheet.
(2) Considering the relatively short length of most gage records, less frequent peak estimates need to be used with considerable care.
(3) Computed one of four ways (see "generalized skew coefficient" worksheet): Mean and variance (standard deviation^2) of station skews coefficients in region; skew isolines drawn on a map or regions; skew prediction equations; read from Plate 1 of Bulletin 17B (reproduced in this spreadsheet), with MSE Generalized Skew = 0.302.
(4) Results are automatically rounded to three significant figures, the dominant number of significant figures in the K-Value table.
(5) Historic frequency analysis assumes that intervening years reflect systematic record.

Comments: *(I) drainage area = 7700 mi^2, contributing drainage area (excluding closed basin) = 4760 mi^2.*
(ii) Generalized skew coefficient and variance computed by computing mean and variance of the station skews of the Upper Rio Grande basin.

Data Plot:



Peak Timing:

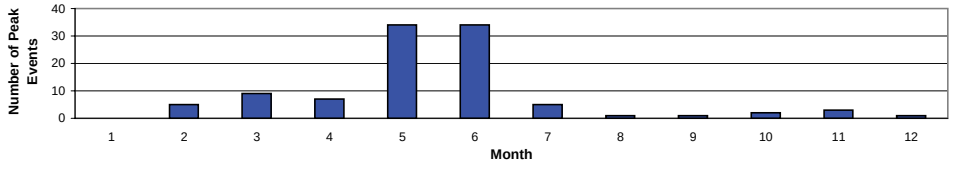


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Figure 5–5 Sheet 2 of log-Pearson spreadsheet output for USGS gage 08251500

NRCS Log-Pearson Frequency Analysis Spreadsheet, Version 2.1, 5/2003 *Page 2 of 3*

Project: *Example 5-5-3*
Streamgage: *# USGS 08251500 RIO GRANDE NEAR LOBATOS, CO.*
Date: *5/27/2003* **Performed By:** *Steve Yochum; Hydrologist, Northern Plains Engineering Team*

Input Data **Station ID:** *08251500* **Latitude, Longitude:** *37°04'42"* *105°45'22"*
Drainage Area (mi^2): *4760* **County:** *Conejos*
Number of low outliers eliminated: *0* **State:** *CO*

	Date	Discharge (cfs)	Historic?	Outlier?		Date	Discharge (cfs)	Historic?	Outlier?		Date	Discharge (cfs)	Historic?	Outlier?
1	05/30/1900	4,700	n	n	51	03/30/1950	6,820	n	n	101	29560	650	n	n
2	05/23/1901	3,620	n	n	52	02/19/1951	320	n	n	102	30103	2170	n	n
3	05/15/1902	565	n	n	53	05/08/1952	11,600	n	n	103	----	----	n	n
4	06/18/1903	12,800	n	n	54	05/30/1953	995	n	n	104	----	----	n	n
5	04/19/1904	751	n	n	55	02/13/1954	360	n	n	105	----	----	n	n
6	06/08/1905	13,200	n	n	56	03/11/1955	280	n	n	106	----	----	n	n
7	06/17/1906	8,380	n	n	57	06/05/1956	681	n	n	107	----	----	n	n
8	07/03/1907	8,800	n	n	58	07/31/1957	3,810	n	n	108	----	----	n	n
9	06/14/1908	2,300	n	n	59	05/29/1958	4,270	n	n	109	----	----	n	n
10	06/10/1909	7,640	n	n	60	03/02/1959	418	n	n	110	----	----	n	n
11	04/30/1910	5,360	n	n	61	06/12/1960	2,040	n	n	111	----	----	n	n
12	06/13/1911	5,910	n	n	62	05/02/1961	1,440	n	n	112	----	----	n	n
13	05/29/1912	8,770	n	n	63	04/22/1962	2,620	n	n	113	----	----	n	n
14	03/23/1913	2,200	n	n	64	11/10/1962	724	n	n	114	----	----	n	n
15	06/05/1914	4,580	n	n	65	11/11/1963	423	n	n	115	----	----	n	n
16	05/19/1915	4,070	n	n	66	06/22/1965	3,790	n	n	116	----	----	n	n
17	05/12/1916	6,000	n	n	67	05/11/1966	1,330	n	n	117	----	----	n	n
18	06/20/1917	7,840	n	n	68	08/13/1967	1,110	n	n	118	----	----	n	n
19	06/16/1918	1,670	n	n	69	06/01/1968	2,470	n	n	119	----	----	n	n
20	05/25/1919	5,090	n	n	70	06/19/1969	2,730	n	n	120	----	----	n	n
21	05/27/1920	9,320	n	n	71	09/18/1970	1,930	n	n	121	----	----	n	n
22	06/16/1921	12,600	n	n	72	03/30/1971	1,720	n	n	122	----	----	n	n
23	06/01/1922	7,300	n	n	73	03/16/1972	856	n	n	123	----	----	n	n
24	06/17/1923	4,120	n	n	74	05/23/1973	3,560	n	n	124	----	----	n	n
25	05/21/1924	7,670	n	n	75	04/01/1974	784	n	n	125	----	----	n	n
26	02/14/1925	1,180	n	n	76	06/18/1975	2,490	n	n	126	----	----	n	n
27	06/04/1926	3,330	n	n	77	05/31/1976	1,450	n	n	127	----	----	n	n
28	07/03/1927	9,830	n	n	78	03/22/1977	405	n	n	128	----	----	n	n
29	06/01/1928	3,960	n	n	79	07/01/1978	979	n	n	129	----	----	n	n
30	05/27/1929	3,580	n	n	80	06/10/1979	4,830	n	n	130	----	----	n	n
31	06/01/1930	1,590	n	n	81	06/13/1980	3,230	n	n	131	----	----	n	n
32	03/22/1931	900	n	n	82	12/05/1980	360	n	n	132	----	----	n	n
33	05/24/1932	5,780	n	n	83	06/01/1982	1,950	n	n	133	----	----	n	n
34	06/03/1933	2,290	n	n	84	06/29/1983	3,230	n	n	134	----	----	n	n
35	02/19/1934	663	n	n	85	05/31/1984	3,390	n	n	135	----	----	n	n
36	06/18/1935	4,600	n	n	86	06/13/1985	6,240	n	n	136	----	----	n	n
37	05/07/1936	2,540	n	n	87	06/11/1986	6,180	n	n	137	----	----	n	n
38	05/19/1937	4,370	n	n	88	05/19/1987	6,760	n	n	138	----	----	n	n
39	05/02/1938	4,040	n	n	89	04/10/1988	848	n	n	139	----	----	n	n
40	03/24/1939	1,640	n	n	90	04/11/1989	1,870	n	n	140	----	----	n	n
41	05/19/1940	1,190	n	n	91	05/10/1990	1,860	n	n	141	----	----	n	n
42	05/16/1941	8,090	n	n	92	05/23/1991	2,130	n	n	142	----	----	n	n
43	05/13/1942	5,580	n	n	93	04/15/1992	1,700	n	n	143	----	----	n	n
44	05/04/1943	1,400	n	n	94	05/30/1993	3,890	n	n	144	----	----	n	n
45	05/18/1944	6,440	n	n	95	06/03/1994	2,320	n	n	145	----	----	n	n
46	05/12/1945	2,880	n	n	96	07/05/1995	6,330	n	n	146	----	----	n	n
47	11/12/1945	822	n	n	97	02/20/1996	650	n	n	147	----	----	n	n
48	05/11/1947	1,960	n	n	98	06/05/1997	3,610	n	n	148	----	----	n	n
49	06/07/1948	8,600	n	n	99	10/15/1997	2,100	n	n	149	----	----	n	n
50	06/22/1949	9,330	n	n	100	06/20/1999	2,310	n	n	150	----	----	n	n

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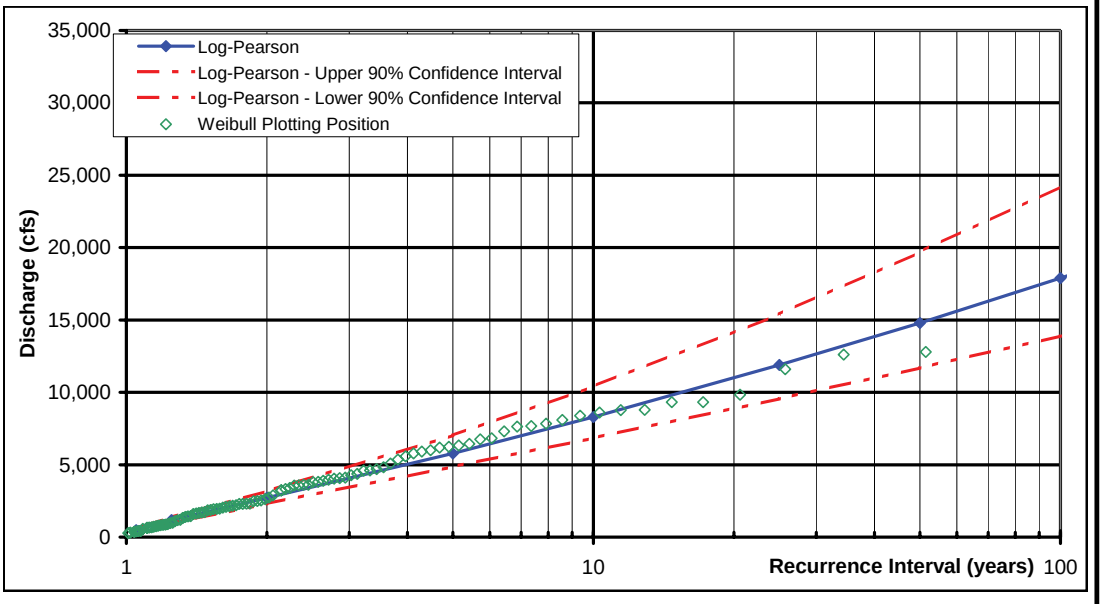
Figure 5–6 Sheet 3 of log-Pearson spreadsheet output for USGS gage 08251500

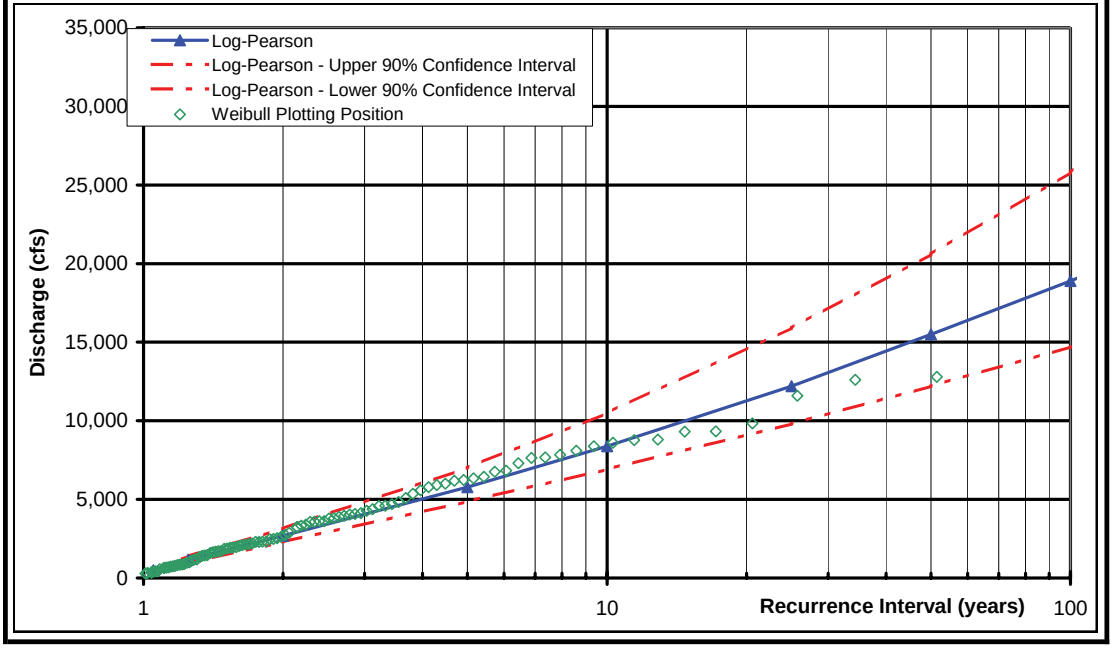
NRCS Log-Pearson Frequency Analysis Spreadsheet, Version 2.1, 5/2003 *Page 3 of 3*

Project: *Example 5-5-3*
Streamgage: *# USGS 08251500 RIO GRANDE NEAR LOBATOS, CO.*
Date: *########* **Performed By:** *Steve Yochum; Hydrologist, Northern Plains Engineering Team*

Discharge-Frequency, with Gage Skew
 # USGS 08251500 RIO GRANDE NEAR LOBATOS, CO



Discharge-Frequency, with Generalized Skew
 # USGS 08251500 RIO GRANDE NEAR LOBATOS, CO



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charges may also be transferred or correlated from gage data from a nearby stream with similar basin characteristics.

Several equations and techniques exist for data transfer. Equation 5–28 is a simple transfer equation:

$$Q_u = Q_g \left(\frac{A_u}{A_g} \right)^x \qquad \text{(eq. 5–28)}$$

where:

Q_u = flood discharge at the ungaged stream
Q_g = flood discharge at the gaged stream
A_u = area at the ungaged stream
A_g = area at the gaged stream
x = regional exponent for area ratio (typically from 0.5 to 1)

Equation 5–28 can be used to develop comparative estimates. The regional exponent is computed by plotting a graph of flows for the same return period and similar basins, and then determining the slope of the best fit line on log–log paper. Example 6 illustrates the calculation. Again, specific regional data are needed for each state, and each hydrologic region within the state.

Transposition of peak flow rates is adversely affected by large differences in watershed lag times, runoff generated from small area thunderstorms, large differences in drainage area size, and differences in soils and vegetative cover. For transfer relations to be effective, the following conditions should be met:

- The drainage area ratio between the gaged and the ungaged area should be two or less.
- The watershed at the gaged location and ungaged watershed must be in the same climatic and physiographic region.

The more deviation exists from these two conditions, the more it is recommended that calculated values be compared with other sources such as regional regression data and computer models.

Low-flow frequency analysis

A project design may require a low-flow assessment for biological design elements or requirements. For example, it may be necessary to know the flow depths and velocities during a defined critical spawning time in a designed channel. A reference similar to WRC Bulletin 17B is not available for low-flow frequency analysis. However, the same log-Pearson type III frequency distribution, used for peak flow analysis, is often used for low-flow analysis.

In the United States, annual minimum flows usually occur in late summer and early fall. The annual minimum average flow for a specified number of consecutive days (usually 7 days) is the typical data point. Computationally, the annual minimum 7-day flow may be found as the annual minimum value of 7-day means. The USGS provides access to daily mean flow values at stream gages across the United States at:

http://waterdata.usgs.gov/nwis

As is done in peak discharge analysis, the mean and standard deviations of the logarithms of the data values are calculated. Then log-Pearson type III frequency factors are applied to assign frequencies or probabilities to the low-flow magnitudes. The frequencies are nonexceedance probabilities. Equation 5–1 is still applicable, but the practitioner needs to be cautioned regarding the meaning of the statistics. For example, the 50-year, 7-day low flow has a 2 percent annual chance of not being exceeded. For comparison, the 50-year peak flow has a 2 percent annual chance of being exceeded.

Inclusion of a period of substantial drought helps ensure that a data sample is representative for low-flow conditions. It should also be noted that the effects of basin development are relatively greater for low flows than for high flows.

Transfer of low-flow frequency estimates to ungaged sites is difficult because of the geologic influence on low flows. If basin characteristics are very similar, drainage area ratios may be used to transfer low-flow data from gaged to ungaged sites. A few low-flow measurements at the ungaged site are good verification for the transferred data.

Example 6: Transfer method

Problem: The gage site has a drainage area of 10 square miles (fig. 5–7). The designer needs an estimate of the 100-year flow at a drainage area of 20 square miles.

Solution: The regression equation developed from the regional data for 100-year flow values, developed from frequency analysis of stream gages in the area, is as follows:

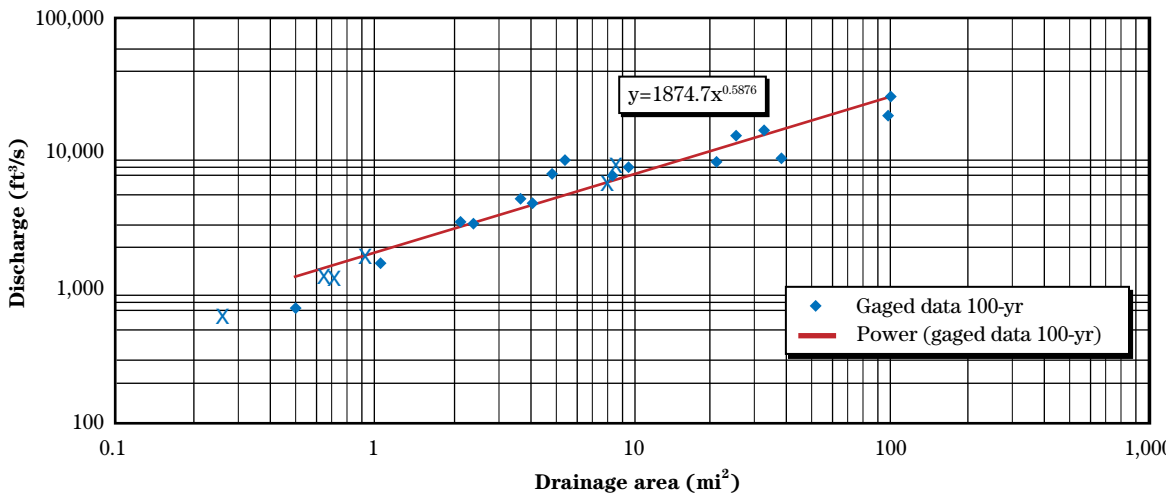
The value or intercept is the coefficient of the equation for a drainage area of 1 square mile (in log units = 0), and the intercept is 1,874.7.

The power of the equation is the slope of the line and is determined in log units as the discharge at 10 square miles, 7,253 cubic feet per second or 3.86 in log units. For a 1-square-mile drainage area, the discharge is 1,874.7 cubic feet per second and in log units is 3.27. The slope would be (3.86–3.27)/1 or 0.59.

Therefore, the 100-year flow at 20 square miles would be:

$$Q_u = 7{,}253\left(\frac{20}{10}\right)^{0.59} = 10{,}917 \text{ cubic feet per second}$$

Figure 5–7 100-yr discharges for the Rock Creek watershed in Montgomery County, MD



654.0505 Regional regression

Cost-effective designs for stream restoration, floodwater retarding structures, and many other conservation practices require peak streamflow frequency estimates. Peak streamflow frequency estimates represent peak discharges for return periods, generally ranging from 2 to 100 years. A regression equation for estimating peak discharges may be developed by statistically relating peak streamflow frequency and drainage basin characteristics for a geographic region of similar flood characteristics.

(a) Basic concepts

Regression forms

Regression is a method for developing a relationship between a dependent (Y) variable and one or more independent (X), predictor variables (NEH630.02, Hydrology). Regression assumptions are:

- No error exists in the independent variable; errors occur only in the dependent variable. Thus, regression is directional.
- Predictor variables are statistically independent.
- The observed values of the dependent variable are uncorrelated events.
- The population of the dependent variable is normally distributed about the regression line.
- A cause-and-effect relationship exists between predictor and dependent variables.

Regression is used to analyze hydrologic data because it provides an easy method for analyzing many factors simultaneously. The simplest form of the linear regression equation, with one predictor variable (X), is written as:

$$Y = a + bX \qquad \text{(eq. 5–29)}$$

where a and b are the intercept and slope regression coefficients. A more complicated form is the linear multiple regression equation, which relates a dependent variable and multiple predictor variables:

$$Y = b_0 + b_1X_1 + b_2X_2 \cdots + b_pX_p \qquad \text{(eq. 5–30)}$$

where:

Y	=	dependent variable, such as 100-year discharge
$b_0, b_1, b_2, \ldots, b_p$	=	partial regression coefficients
$X_1, X_2, \ldots, X_p$	=	independent (predictor) variables

Linear regression calculations are tedious by hand and are usually performed with the aid of programmed procedures on a computer. Example calculations may be found in NEH630.18.

Evaluating regression equations

One of the most commonly used measures of goodness of fit is the coefficient of determination, usually expressed as R^2. It is the dimensionless ratio of the explained variation in the dependent variable over the total variation of the dependent variable. A coefficient of determination of 1.0 indicates that the values of the dependent variable can be calculated exactly using the predictor variables in the given data set. The lower the R^2 value, the less direct the relation is and the wider the scatter in the data. Since this value is dimensionless, it can be used to compare goodness-of-fit of different regression equations. It does not provide a quantified expected variation. If a relationship is nonlinear, the regression coefficients will be dependent on the choice of independent variables, as well as on the curve fit relationship.

It should also be noted that a high degree of correlation (R^2 close to 1.0) does not necessarily mean that there is either causation or even a direct dependence between the variables. It only indicates that the given set of data can be predicted with the regression equation. In all circumstances, the reasonableness of the relationship between independent and dependent variables should be examined. Extremely high R^2 values (0.95 and above) can indicate bias in the data collection or an insufficient number of collected data points for the order of the calculated regression equation. For example, if only two data points are collected, a straight line regression equation between the two will have an R^2 value of 1.0.

Another measure of the quality of a regression equation is the standard error of estimate, typically expressed as $S_{Y,X}$. This is the root mean square of the estimates and is a measure of the scatter about the regression line of the independent variable. The standard error of estimate is not reflexive. It shows how

well the dependent variable correlates to the independent variable, but not vice versa. The standard error of estimate has similar properties to the standard deviation and can be thought of as the standard deviation of the residuals. A residual is the difference between the value predicted with the regression equation and the observed dependent variable. As the standard error of estimate approaches 0, the quality of the regression equation increases.

Step-type regressions can be used to evaluate the significance of each predictor variable in a regression equation. The significance of adding or deleting predictor variables is evaluated with an F-test. A computed F greater than a table F-value indicates significance (see NEH 630.18 for more details). For example, a step forward regression starts with the most important predictor as the only variable in the equation. The most important of the remaining predictors is added, and the F-value computed. If this predictor is significant, another of the remaining predictors is added, and the process repeated. When a predictor is not found significant, the previous equation, not including that predictor, is used for analysis.

(b) Regional analysis

Regional study helps assure consistency of estimates at different locations and provides means for estimating discharge-frequency values at locations where gaged data are not available. Also, flow discharge estimates at a gaged location can usually be improved by a study of gaged frequency characteristics throughout the region.

Simplified regional study method

Regional analysis allows the estimation of peak discharge magnitude and frequency for ungaged watersheds by using relationships from nearby gaged watersheds. NEH630.02, Hydrology, provides the regional analysis in its simplest form.

- Select nearby gaged watersheds that are climatically and physically similar to the ungaged watershed.
- Construct frequency lines of peak discharges for each gaged watershed.
- Plot peak discharges for selected frequencies of each gaged watershed against its drainage area. Use log-log paper for plotting. A simple regression (curve fitting) between log of drainage area (predictor variable) and log of discharge (dependent variable) aids in drawing a best fit straight line for each selected frequency.
- Construct the frequency line for the ungaged watershed as follows: enter the plot with the ungaged drainage area, find and plot the discharges on log-probability paper, and draw the frequency line through the points.

Use of regression equations

Regression equations are used to transfer flood characteristics from gaged to ungaged sites through use of watershed and climatic characteristics as predictor variables. The USGS has developed regional regression equations for each state and some territories, usually as part of cooperative studies with state departments of transportation (USGS 2002a, Report 02–4168).

General descriptions of techniques that USGS uses in developing regression equations follow. Frequency lines of peak discharges are developed at gaging stations following the recommendations of WRC Bulletin 17B (WRC 1981).

The regression equations generally take the form:

$$Q_T = aX^bY^cZ^d \qquad \text{(eq. 5–31)}$$

where:

Q_T = peak discharge of selected frequency; 100-year discharge (dependent variable)

X,Y,Z = watershed or climatic characteristics (predictor variables)

a,b,c,d = regression coefficients

With the log transform, the equation takes the form of equation 5–30. The most often used watershed and climatic characteristics are drainage area, main channel slope, and mean annual precipitation. Regression regions are generally determined by using major watershed boundaries and an analysis of the areal distribution of the residuals. As noted above, residuals are the differences between regression and observed flow estimates. For USGS regression equations, the region has already been predetermined for the end user. Regression equation use is illustrated in example 7.

Example 7: USGS regional equation (USGS Report 96–4307)

An ungaged watershed is located in Region 5, Texas. The watershed has a drainage area (A) of 13.2 square miles and stream slope (SL) of 71.3 feet per mile, determined with the aid of USGS 7.5 min quadrangle maps. The following regression equation applies for estimating the 100-year discharge:

$$Q_{100} = 295A^{1.01}\left(SL\right)^{0.405} = 22{,}500\ \text{ft}^3/\text{s}$$

Report 96–4307 gives a standard error of estimate of 78 percent. This means that there is roughly a two-thirds chance that the true 100-year discharge falls between 4,950 cubic feet per second and 40,050 cubic feet per second. Report 96–4307 also gives a means to calculate more exact confidence intervals (not shown here). An output report for the same ungaged watershed, generated by the USGS National Flood Frequency Program, follows:

```
National Flood Frequency Program
Version 3.0
Based on Water-Resources Investigations Report 02-4168
Equations from database NFFv3.mdb
Updated by kries 10/16/2002 at 3:51:06 PM new equation from WRIR 02-4140
Equations for Texas developed using English units

Site: MilldamTX, Texas
User: lgoertz
Date: Thursday, July 10, 2003 04:26 PM

Rural Estimate: Rural 1
  Basin Drainage Area: 13.2 mi2
  1 Region
  Region: Region_5_(A<32mi2_(51km2))
   Contributing_Drainage_Area = 13.2 mi2
   Stream_Slope = 71.3 ft/mi

Flood Peak Discharges, in cubic feet per second

             Recurrence    Peak,  Standard   Equivalent
Estimate     Interval, yrs cfs    Error, %   Years
____________ _____________  _____  ________   __________

Rural 1            2        919       75
                   5       2910       63
                  10       5180       66
                  25       9310       69
                  50      15900       72
                 100      22500       78
                 500      50600
```

Accuracy and limitations

The standard errors of estimate or prediction range from 30 to 60 percent for most regression equations. The largest standard errors generally are for equations developed for the western part of the Nation, where the variability of the flood records is greater, gaging stations are less dense, and flood records are generally shorter. Regression equations developed from gaged natural basins should only be used on natural basins to make regression estimates. A natural basin may be defined as a basin with less than 10 percent impervious cover and less than 10 percent of its drainage area controlled or manipulated to affect peak stream flow. Users should exercise caution in extrapolating flood estimates beyond the ranges of predictor variables used in developing the equations.

Regression equations are not as accurate as frequency analysis from gaged data. For design purposes in high risk situations, both regression equations and hydrologic modeling methods should be employed.

(c) Computational resources for regional regression analysis of peak flows

The USGS has developed and published regression equations for a variety of locations within the United States. These equations have been compiled into the National Flood Frequency (NFF) Program. A computer program, National Flood Frequency Program, version 3: A Computer Program for Estimating Magnitude and Frequency of Floods for Ungaged Sites provides access to this information. The following USGS Web site provides regional regressions for flood peaks developed for many regions throughout the United States:

http://water.usgs.gov/software/nff.html

Regional regression relationships for bankfull discharge

Bankfull discharge regional regression relationships can present some different issues to a designer than relationships for peak flows. However, the basics remain the same. Information on developing regional regression relationships for bankfull discharge is provided in NEH654 TS5.

654.0506 Flow duration

Flow duration is the percentage of time that a given flow was equaled or exceeded over a period of time. A flow-duration curve for stream flow represents the hydrograph of the average year (or season) with its flows arranged in order of magnitude. For example, the flow value in the average year to be exceeded 20 percent of the time may be read from the flow-duration curve for that location.

Flow-duration curves have been used in the analysis of sediment transport quantities, critical habitat functions, water quality management alternatives, and water availability. It is often important to determine how the proposed restoration project will perform with low or normal flows. While flow-duration curves are typically calculated for several (usually >10) years of homogeneous record, they can be developed for specific seasons since seasonal flow variations can have critical habitat importance. For example, a project goal may include a minimum flow depth during a critical spawning period for anadromous fish species and a lower minimum depth for resident fish species. The same techniques used to develop flow-duration curves for sediment analysis can also be used to assess and design for habitat conditions. An example is provided in figure 5–8.

The USGS has developed flow-duration curves for many gaged locations in the United States. These curves are normally available on request from the USGS. The construction procedure used by USGS is outlined in Searcy (1959). Procedures for developing flow-duration curves are also described in Hydrologic Frequency Analysis, EM 1110–2–1415 (USACE 1993a). Data are typically sorted by magnitude, and the percent of the time that each value is exceeded is calculated. Since the data points are typically daily averages, each point will not necessarily be independent. This is a relatively simple statistical analysis. Data bin ranges are developed, and the numbers of occurrences are counted for each bin. For example, the number of times the flow was between 500 and 1,000 cubic feet per second would be counted. Then the percent of occurrence is assembled as a cumulative distribution function to define the percent of time that the flow is above a certain discharge or level. Flow-duration analysis is performed by using daily average flow (or

other periods such as 3-day, 5-day, or weekly) during the period of interest. Historical and real time daily average flow data can be found at the following USGS Daily Flow Stream Gage Data Web site:

http://nwis.waterdata.usgs.gov/usa/nwis/discharge

The data can be stratified by seasons for this analysis, depending on study goals. The information can be for the entire period of record. However, if the watershed has undergone some significant change (as is typical in many stream restoration projects), it may be necessary to use only the record since the change has occurred. This is necessary to keep the data homogeneous.

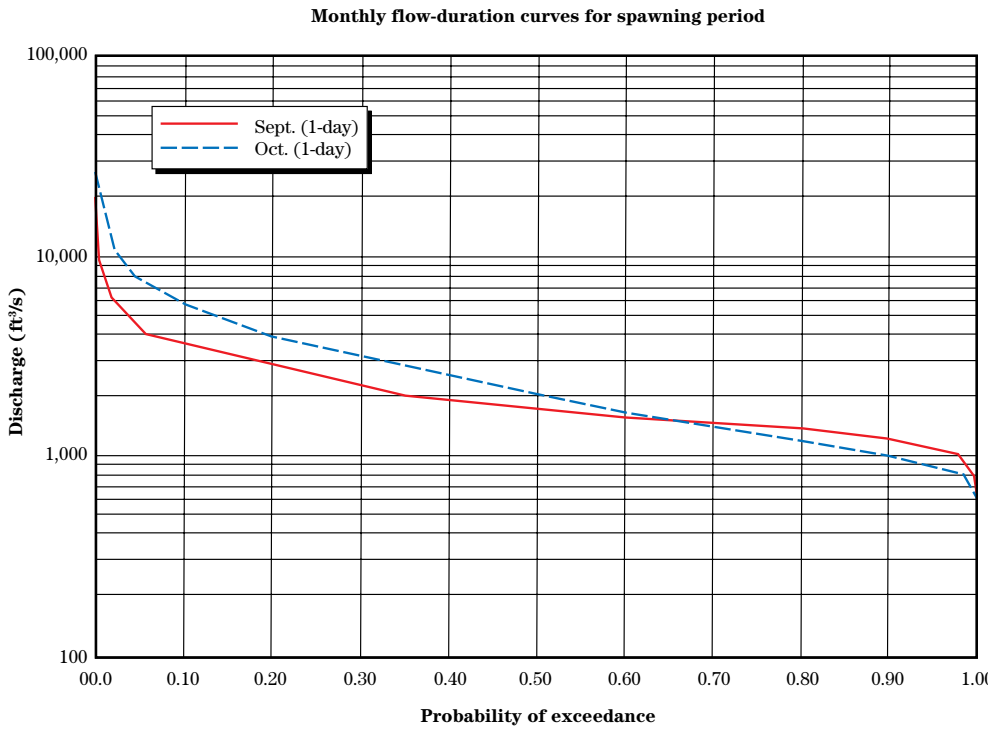
Transfer methods, as described earlier, can be used to transfer flow duration information from gaged sites to ungaged areas. However, these should have similar watershed characteristics, and the ratio of gaged to ungaged drainage area should be between 0.5 and 2.0 for reliable results. The accuracy of such a procedure is directly related to the similarity of the two sites. Typically, there is more error in transferring or estimating the ends of a flow-duration curve. Flow duration is dependent on watershed conditions. If regional flow-duration relations are to be developed, it is recommended that a measure of watershed conditions be included as an independent variable.

Two methods for estimating a flow-duration curve for ungaged sites are described by Biedenharn et al. (2000). They are the:

- drainage area flow-duration curve method
- regionalized-duration curve method

Graphs for the drainage area flow-duration curve method, for a specified recurrence interval discharge versus drainage area, are developed for a number of sites on the same stream or within hydrological similar portions of the same drainage basin. If data are reason-

Figure 5–8 Typical flow-duration curve



ably homogeneous, regression techniques should be used to generate curves of flow for selected percentile versus drainage area. By knowing the drainage area of the selected site(s), a flow-duration curve can be generated from the regression equations.

With the regionalized duration curve method, a nondimensional flow-duration curve is developed for a hydrologically similar gaged site by dividing discharge by bankfull discharge or by a specified recurrence interval discharge. Then a specified recurrence interval discharge is computed for the ungaged site using the aforementioned regression equations. Finally, the flow-duration curve for the ungaged site is derived by multiplying the dimensionless flows (Q/Q_2) from the nondimensional curve by the site Q_2. It should be noted that both methods simply provide an approximation to the true flow-duration curve for the site because perfect hydrologic similarity never occurs.

654.0507 Hydrologic models

There are many mathematical and computer hydrologic modeling systems available for predicting runoff from precipitation and snowmelt events that provide the volume and timing of water moving through the system. Models provide the ability to estimate existing, as well as future rainfall runoff patterns for a variety of conditions. Depending on the hydrologic model used, either single event peak flow or continuous multiple event modeling can be performed.

The accuracy of models is highly dependent on calibration data, which can often be difficult to acquire. However, if the issues that are to be addressed are comparative in nature rather than absolute, the importance of calibration is diminished. However, the results of a model study should fall between the USGS regional regression equation for the site and the upper bounds of one standard error of estimate. If the results of the model calibration are not within these bounds, after adjustment of the model parameters within reasonable limits, the reasons for the final answer and its derivation must be explained in the project documentation.

The level of accuracy required for a specific hydrologic analysis generally depends on the specific characteristics of each individual project. The selection of the appropriate methodology should be done with a firm understanding of the assumptions, accuracy, data requirements, and limitations of the approach. Brief statements on the use of the models are provided.

The rational method (rational formula) is one of the easiest models to implement. It can be used for drainage areas up to 80 hectares (200 acres). Use of the rational formula on larger drainage areas requires sound judgment to ensure reasonable results. The hydrologic assumptions underlying the rational formula include:

- constant and uniform rainfall over the entire basin
- a rainfall duration equal to the time of concentration

The rational method is not appropriate if:

- the basin has more than one main drainage channel

- the basin is divided so that hydrologic properties are significantly different in one section versus another
- the time of concentration is greater than 60 minutes
- storage is an important factor

The NRCS TR–55 method (USDA Soil Conservation Service (SCS) 1986) provides a manual method for computing peak discharges for drainage basins. The TR–55 method is segmental (flow time is computed by adding the travel times for the overland, shallow concentrated, and channel segments). TR–55 considers hydrologic parameters such as slope of the watershed and channel, channel roughness, water losses, rainfall intensity, soil type, land use, and time. TR–55 should be used with caution when the design is highly sensitive to the computed peak flow values. TR–55 also assumes that rainfall is uniform over the entire basin. Additional assumptions include:

- the basin is drained by a single main channel or by multiple channels with times of concentration that are nearly equal
- the weighted curve number should be greater than 40
- runoff from snowmelt or rain on frozen ground cannot be estimated using the procedures in TR–55
- the time of concentration should be between 0.1 and 10 hours
- storage in the drainage area is less than 5 percent of the runoff volume and does not affect the time of concentration
- a single composite curve number can accurately represent the watershed runoff characteristics

A computer program has been developed to automate the manual procedures in TR–55. The computer program developed in the Windows® environment is known as WinTR–55. The WinTR–55 computer program is available at the following Web site:

http://www.wcc.nrcs.usda.gov/hydro/hydro-tools-models-wintr55.html

The HEC–1/HEC–HMS models are rainfall-runoff models developed by the USACE Hydrologic Engineering Center (USACE 1981). These models can be used with basins of almost any size and complexity. HEC–1 is designed to simulate the surface runoff resulting from precipitation over a watershed by representing that watershed as an interconnected system of components. These components consist of surface runoff, stream channels, and reservoirs. Each component is represented by a set of parameters, which specify its characteristics, and the mathematical relations, which describe its physical processes. The end result of the HEC–1 modeling process is the computation of runoff hydrographs for the subbasins and stream channels. The program is composed of five basic sub models as illustrated in figure 5–9.

HEC–1 assumes that the rainfall is spatially uniform over each subbasin modeled. NRCS rainfall time distributions, loss methods, dimensionless unit hydrographs, and the lag equations often are used; however, careful consideration must be given to the assumptions and limitations underlying these methods. For example, the NRCS has published an upper limit on basin size for the NRCS lag equation of 800 hectares (2,000 acres, 3.1 mi^2) (NEH630.15). The upper limit on basin area for the NRCS Loss Method (runoff curve number) is not well established; however, a limit of 20 square miles has been suggested. These limitations may be overcome by subdivision of the watershed and appropriate routing. Various GIS packages can be used as an interface to HEC–1. These GIS techniques systematize the computation of the physiographic and hydrologic parameters required by HEC–1.

The WinTR–20 model is a rainfall-runoff model developed by the NRCS (USDA NRCS 2004). It can be used with basins of almost any size and complexity. WinTR–20 is designed to simulate the surface runoff resulting from precipitation over a watershed by representing that watershed as an interconnected system of components. These components consist of surface runoff stream channels and reservoirs. The program is composed of five submodels as illustrated in figure 5–9. Normally, it is assumed that the rainfall is uniform over each subbasin. However, that rainfall total can be varied for each subbasin. Actual or design temporal rainfall distributions can be used with standard dimensionless unit hydrographs that are part of the normal inputs. Several GIS computer programs

that can be used to develop the areal input values such as curve numbers are available. NRCS has developed a GIS computer program that provides geographical information in the proper format for WinTR–20 (USDA NRCS 2004). The program is available from the following Web site:

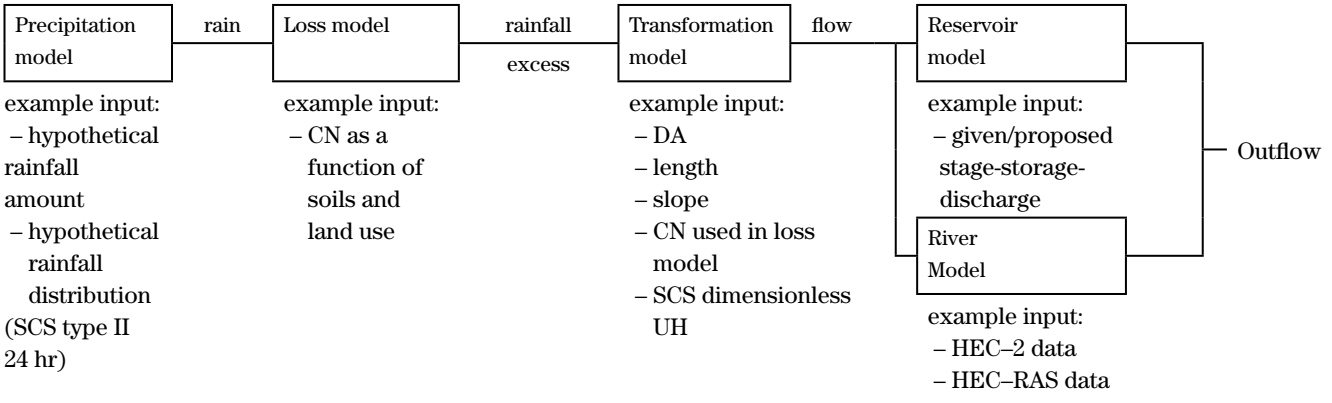
www.wcc.nrcs.usda.gov/hydro/

Use of these models is fairly common in ungaged systems, or in areas where land use and stormwater detention systems significantly alter the hydrograph. With the advent and collection of soil, vegetation, topography and land use types in GIS, model development and database management is a simpler process.

654.0508 Channel-forming discharge

Natural alluvial streams experience a wide range of discharges and adjust their shape and size during flow events that have sufficient energy to mobilize the channel boundary materials. Until the 1960s, it was widely assumed that floods of great magnitude, but low frequency, controlled channel form because of the nonlinear relationship between discharge and sediment transport capacity. Sediment transport increases exponentially with discharge. This view was challenged by Wolman and Miller (1960) who argued that in most streams, over an extended period of time, the total amount of sediment transported by a discharge of a given magnitude depends not only on its transport capacity, but also its frequency of occurrence. Thus, although extremely large events can produce spectacularly high sediment loads, they happen so infrequently and last such a short time that their overall contribution to the total sediment movement during a long period is relatively small. Small events also make a small contribution to the total sediment moved because their high frequency of occurrence is offset by their very low sediment transport capacity. It follows from this logic that flow of both moderate magnitude and moderate frequency is responsible for the greatest

Figure 5–9 Five basic submodels of a rainfall/runoff model



amount of sediment movement (Leopold, Wolman, and Miller 1964). However, recent studies have indicated that this concept may not hold true for all streams (Werrity 1997).

Channel-forming discharge concept

The channel-forming discharge concept is based on the idea that, for a given alluvial channel, there exists a single steady discharge that, given enough time, would produce channel dimensions equivalent to those produced by the natural hydrograph. This discharge is thought to dominate channel form and process. Estimates of channel-forming discharges are used to classify stream types, estimate channel dimensions, assess stability, and express hydraulic geometry relationships.

While many techniques and methodologies are used to estimate a channel-forming discharge in stable alluvial channels, all can be characterized as one of four main types. These are:

- discharge based on bankfull indices
- discharge based on drainage area
- discharge based on specified statistical recurrence intervals
- discharge based on an effective discharge calculation

Discharge based on bankfull indices

Channel-forming discharge based on bankfull indices is determined by visually inspecting the reach in question or surveys of this reach to locate morphological evidence of the bankfull stage. The discharge associated with this stage is then computed or estimated. Identifying relevant features that define the bankfull stage can be problematic (Williams 1978), particularly in dynamic, unstable channels (Simon, Dickerson, and Heins 2004). Many field indicators have been proposed and are briefly described in table 5–10.

Identifying bankfull stage from indicators is subjective. None of the bankfull indicators is applicable in all situations (Williams 1978). Many workers use a combination of the indices in an iterative fashion. However, even experienced observers may arrive at conflicting or misleading results, particularly for conditions outlined in table 5–11.

The field identification of bankfull indicators is particularly problematic in stream reaches that are unstable or threshold. If the project reach is not stable or alluvial, it may be possible to find indicators of bankfull stage in stable alluvial reaches upstream or downstream. However, since stream restoration is most often practiced in unstable watersheds, field determination of bankfull stage may be impractical or impossible (Copeland et al. 2001). An exception could be found in a stable and alluvial incised stream that has formed a new flood plain within the incised channel. In this case, the top of the high bank is now an abandoned flood plain or terrace, and there should be newly formed top-of-bank features within the older incised channel. However, it is important to remember that the new flood plain may not yet be fully formed; that is, the channel may not be stable and may still be aggrading. In addition, a new inset flood plain (sometimes referred to as incipient flood plain) may be restricted in width or height due to channel constraints. Measurements taken in such situations would give misleading values for the bankfull discharge.

When applying the estimate of bankfull stage from one reach to another, it is important to keep in mind that the location of the break between the channel and the flood plain is influenced by many factors, including (but not limited to) the following:

- climatic regime (humid vs. arid)
- geologic erosion conditions of the streambank materials (bedrock vs. unconsolidated material; coarse vs. fine textures; cohesive vs. noncohesive)
- stream slope
- hydrologic regime (perennial vs. intermittent versus ephemeral)
- sediment source, quantity and supply including distribution along the active channel and flood plain.
- stream confinement or width
- stream downcutting or incisement
- size and type of vegetation on the flood plain and within the channel
- controls on channel width and alignment such as riprap and bridge abutments

Table 5–10 Summary of bankfull indices

Bankfull indicator	Reference
Minimum width-to-depth ratio	Wolman (1955) Pickup and Warner (1976)
Highest elevation of channel bars	Wolman and Leopold (1957)
Elevation of middle bench in rivers with several overflow sections	Woodyer (1968)
Minimum width-to-depth ratio plus a discontinuity (vegetative and or physical) in the channel boundary	Wolman (1955)
Elevation of upper limit of sand-sized particles in boundary sediment	Leopold and Skibitzke (1967)
Elevation of low bench	Schumm (1960); Bray (1972)
Elevation of active flood plain	Wolman and Leopold (1957) Nixon (1959)
Lower limit of perennial vegetation	Schumm (1960)
Change in vegetation (herbs, grass, shrubs)	Leopold (1994)
A combination of: • elevation associated with the highest depositional features • break in bank slope • change in bank material • small benches and other inundation features • staining on rocks • exposed root hairs	Rosgen (1994)

Table 5–11 Summary of stream conditions that affect bankfull indices

Reach condition	Process	Effect on bankfull indices
Threshold	Sediment transport capacity of the reach exceeds the sediment supply, but the channel grade is stable	Bankfull indices may be relics of extreme flood events, and may indicate a bankfull flow that is too high
Degrading	The sediment transport capacity of the reach exceeds the sediment supply to the reach, and the channel grade is lowering	The former flood plain is in the process of becoming a terrace. As a result, bankfull indices may indicate a flow that is too high
Aggrading	The sediment transport capacity of the reach is less than the sediment supply	The existing flood plain or in channel deposits may indicate a flow that is too low
Recently experienced a large flow event	Erosion and/or deposition may have occurred on the bed and banks	Bankfull indices may be missing or may reflect the large flow event
Channelized	Sediment transport capacity may not be in balance with sediment supply. The channel may be aggrading or degrading. The reach may be functioning as a threshold channel	Bankfull indices may be relics of previous channel, artifacts of the construction effort, embryonic, or missing altogether

- controls on channel depth and slope such as drop structures, rock weirs, check dams, beaver dams, and cross vanes.

For example, the bankfull discharge measured from a reach with a narrow flood plain may be inappropriate for use on another reach of the same stream, which has a wide flood plain.

Once bankfull stages are estimated for a stream reach (generally over at least one meander wavelength or 10 channel widths); the bankfull discharge can be estimated. This is often done with either a resistance formula calculation such as Manning's equation or with a computer model such as HEC–RAS (Brunner 2002). Practitioners should keep in mind that the use of resistance equations such as Manning's equation or the Darcy-Weisbach equation, while rapid, are subject to the error inherent to the normal depth assumption. In addition, it should be noted that because stage is not a unique function of discharge in alluvial streams, some data scatter should be expected (Copeland et al. 2001). Uncertainty associated with stage-discharge relationships is addressed in more detail in standard manuals and texts (USACE 1996). Additional guidance on the identification of bankfull discharge indicators is provided in NEH654 TS5.

Discharge based on drainage area

Many relationships are available that correlate dominant discharge to drainage area. These offer a quick technique for assessing a dominant discharge. However, the practitioner should keep in mind that these relationships are basically best fit lines that are plotted through a data set. There is a distribution of valid bankfull discharge estimates that will fall both above and below the line. For example, figure 5–10 illustrates such a curve developed by Emmett (1975) for the Salmon River in Idaho. Although the regression line fits the data in a visually satisfactory fashion, it should be noted that for a drainage area of about 70 square miles, the bankfull discharge varied between about 300 and 900 cubic feet per second.

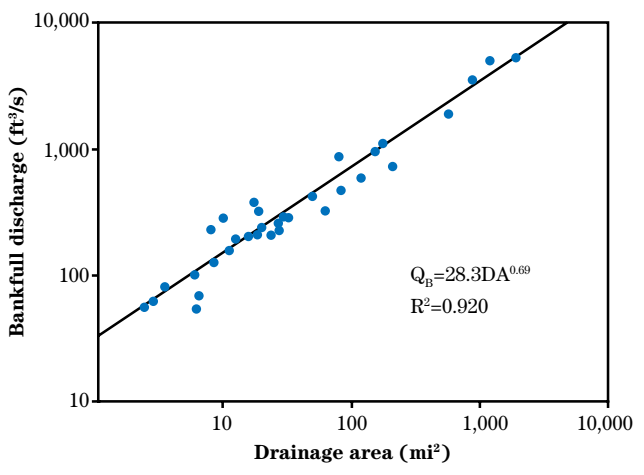
While drainage area is certainly an important factor in estimating streamflow, it is only one of many parameters affecting runoff. Caution should also be used when assessing the relevance of the relationship to watersheds in different physiographic areas or watersheds with different runoff characteristics. Finally, while drainage area is certainly an important factor to estimate streamflow, it is only one of many parameters affecting runoff.

Discharge based on a specific recurrence interval

Many practitioners have related the channel-forming discharge to a specific recurrence interval. The use of a recurrence interval to estimate the channel-forming discharge offers the advantage of being able to calculate a value using gage records, hydrologic modeling, or regional regression relations. Regression equations for estimating discharges with recurrence intervals from 2 to 100 years (Q_2 to Q_{100}) are available for the entire United States via *http://water.usgs.gov/software/nff.html*, as well as from many state and local organizations. This recurrence interval for channel-forming discharge is often assumed to correspond to fall between Q_1 and $Q_{2.5}$, with a mean of $Q_{1.5}$ (Leopold 1994).

However, there are many instances where the channel-forming discharge does not fall within the 1- to 2.5-year range. Williams (1978) showed that out of 35 flood plains he studied in the United States, the bankfull discharge (measured at top of bank) varied between the 1.01- and 32-year recurrence interval, and that only about a third of those streams had a bankfull discharge recurrence interval between 1 and 5 years.

Figure 5–10 Bankfull discharge as a function of drainage area for the Salmon River, ID



In a similar study, Pickup and Warner (1976) showed that bankfull recurrence intervals ranged from 4 to 10 years. The recurrence interval is usually calculated by determining the flow that corresponds to bankfull indices as addressed in the previous section. Therefore, the issues addressed that are associated with the reliable physical identification of bankfull discharge indices impact the calculation of the recurrence interval and may account for some of the discrepancies. Simon, Dickerson, and Heins (2004) used computations based on suspended-sediment transport to compute effective discharge for 10 gages on unstable sand-bed channels in Mississippi. The resulting values of effective discharge ranged from 0.56 to 2.72 of the $Q_{1.5}$, with a mean of 1.04 $Q_{1.5}$.

Nevertheless, the use of a specified recurrence interval is often used as a first approximation of channel-forming discharge. But, because of the noted discrepancies, field verification is generally recommended to ensure that the selected discharge reflects morphologically significant features.

Discharge based on an effective discharge calculation

The effective discharge is defined as the mean of the arithmetic discharge increment that transports the largest fraction of the annual sediment load over a period of years (Andrews 1980). The effective discharge incorporates the principle prescribed by Wolman and Miller (1960) that the channel-forming discharge is a function of both the magnitude of the event and its frequency of occurrence. An advantage of using the effective discharge is that it is a calculated value not subject to the problems associated with determining field indicators (Copeland et al. 2001). Effective discharge computation consists of three steps.

Step 1 The flow-duration curve is derived from available stream gage data.

Step 2 Sediment data or an appropriate sediment-transport function is used to construct a bed material sediment rating curve.

Step 3 The flow-duration curve and the bed material sediment rating curve are integrated to produce a sediment load histogram that displays sediment load as a function of discharge for the period of record. The histogram peak is the effective discharge increment.

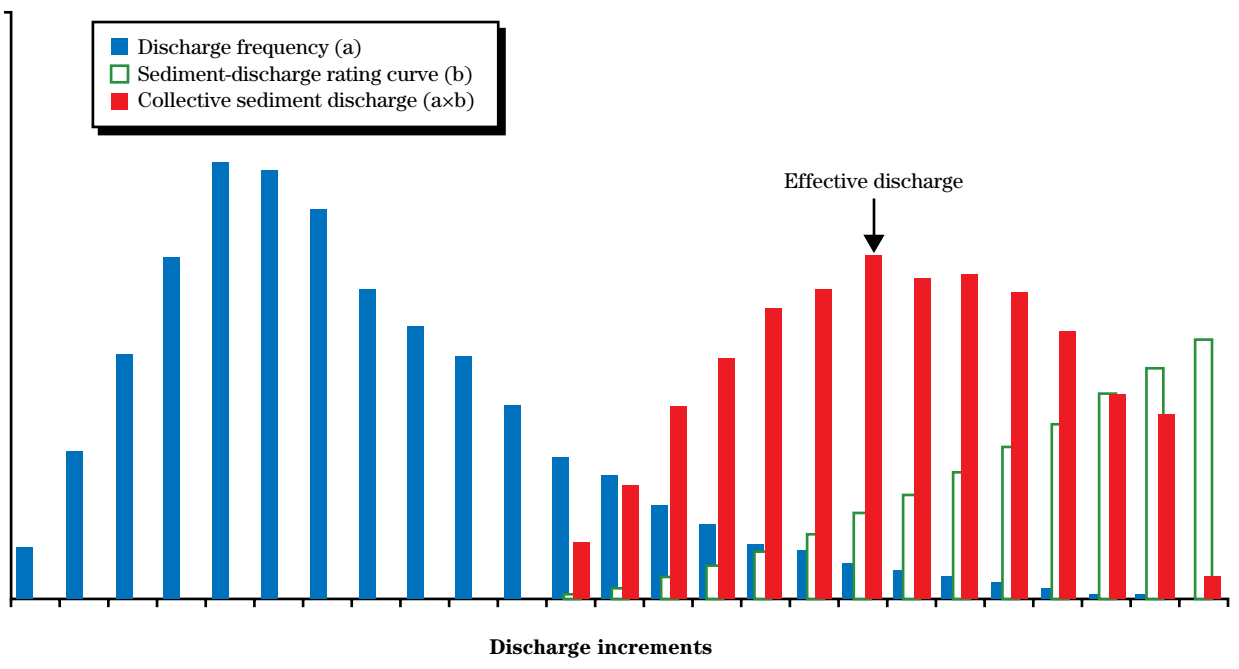
Specific instructions for calculating effective discharge can be found in the literature (Copeland et al. 2001; Biedenharn et al. 2000; and Thomas et al. 2000). Details of the procedure can influence the outcome, so study of these references is recommended. A graphical representation of the relationship between sediment transport, frequency of the transport, and the effective discharge is shown in figure 5–11. The peak of the effective discharge curve in figure 5–11 marks the discharge fraction that transports most of the material, and therefore, does the most work in forming the channel.

Effective discharge analyses may be performed for ungaged reaches by synthesizing a flow-duration curve and applying an appropriate sediment transport function to obtain a bed material sediment rating curve. Flow-duration curve synthesis may be done by plotting curves of discharge versus upstream drainage area for a given exceedance duration, using data from gages within the same watershed as the site of interest. A family of such curves may be created by varying the exceedance duration, and an appropriate flow-duration curve for the site of interest may be interpolated using its drainage area (Hey 1975). If flow-duration data are not available for adjacent gages, then regional information may be used after dividing discharge by either bankfull discharge or the 2-year discharge to produce a dimensionless ratio (Watson, Dubler, and Abt 1997). The dimensionless curve may be applied to the site of interest by multiplying by the base (Q_2 or bankfull Q) that is estimated using one of the aforementioned methods.

Since channel instability is the result of an imbalance in sediment supply and transport capacity, the greatest advantage of using effective discharge in restoration design lies in the fact that it requires quantification of the sediment transport capacity of a channel for a given hydrologic regime. Various channel geometries can be examined for their competence to transport the incoming sediment load, facilitating comparison of permutations of channel dimensions to optimize sediment transport efficiency within logistical constraints. This information is also useful when predicting the impact of alteration of watershed conditions with respect to sediment loads (upstream dam removal) or hydrology (urbanization) on channel stability (Copeland et al. 2001).

An important limitation of using an effective discharge analysis is that it is based on the assumption that the stream will transport the amount of sediment that it is hydraulically capable of moving, and it is this hydraulic capacity that forms the channel. In an urbanized watershed, once the urbanization is complete, the result is that the drainage area is partially covered so that the overland sediment yield reduced. In the Piedmont Region of Maryland, for example, many streams have degraded to bedrock or contain bed material that has been winnowed to a coarse gravel or cobble. These conditions, coupled with an increase in average annual flows, indicate that streams may have an excess sediment transport capacity. In this situation, the channel may be now operating as a threshold channel and the concept of effective discharge may not be relevant. Additional errors occur in effective discharge computations due to the assumption that sediment discharge is a continuous function of water discharge. Internal fluvial system thresholds or limitations on sediment supply may invalidate this assumption, leading to major errors at higher discharges (Nash 1994). An example calculation of effective discharge is provided in example 8.

Figure 5–11 Effective discharge calculation



Example 8: Effective discharge

Problem: Given the following flow-duration curve (fig. 5–12) and sediment transport rating curve (fig. 5–13), calculate the effective discharge:

Solution: The sediment transport rating curve was calculated from data collected during field surveys. The bed material gradation in the upstream supply reach was determined from the average of three volumetric bulk samples taken laterally across the stream. The cross-sectional geometry and slope were surveyed. Hydraulic parameters were calculated assuming normal depth. The Meyer-Peter Muller equation was chosen to make the sediment calculations because the bed was primarily gravel. The calculated bed material sediment transport rating curve is shown in figure 5–13.

The basic approach is to divide the natural range of streamflows during the period of record into a number of arithmetic classes, and then calculate the total bed material quantity transported by each class. This is achieved by multiplying the frequency of occurrence of each flow class by the median sediment load for that flow class. This can be accomplished using a spreadsheet or the USACE SAM program (Thomas, Copeland, and McComas 2003).

Table 5–12 represents output from the SAM program for the given conditions. The discharge increment with the largest increment of sediment transport is between 1,000 and 1,200 cubic feet per second. The effective discharge is then 1,000 cubic feet per second. The program also calculates the average annual sediment load, which is the sum of the sediment loads for each increment. In this case, the annual sediment load is 10,677 tons. A graphical representation of the effective discharge calculation is shown in figure 5–14.

Figure 5–13 Sediment transport rating curve calculated from bed material gradation collected upstream from the project reach and hydraulic parameters from surveyed cross section

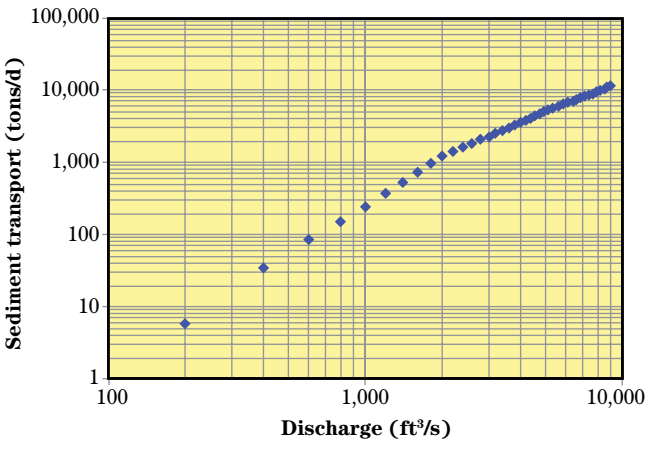


Figure 5–12 Flow-duration curve developed from 39 years of record at a USGS gage downstream from the project reach

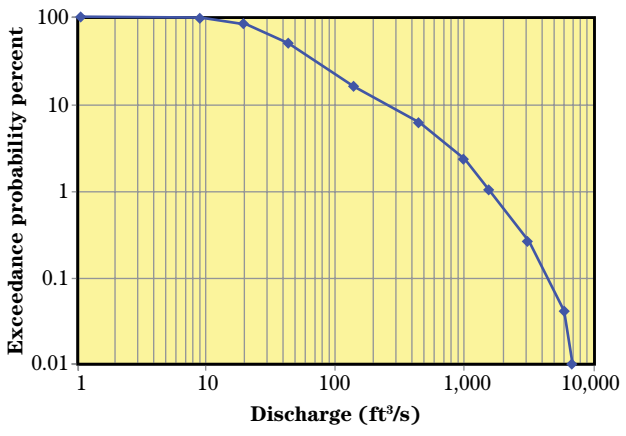


Figure 5–14 Effective discharge calculation

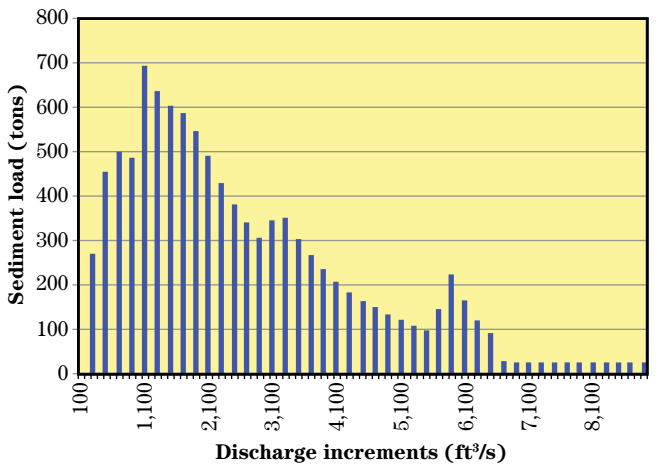


Table 5–12 Effective discharge calculation from SAM program

```
*****************************************************************************************
*                    SAMwin Software Registered to the US Army Corps of Engineers       *
*****************************************************************************************
*                                  SEDIMENT YIELD CALCULATIONS                          *
*                                          Version 1.0                                  *
*                      A Product of the Flood Control Channels Research Program         *
*            Coastal & Hydraulics Laboratory, USAE Engineer Research & Development Center *
*                                       in cooperation with                             *
*                          Owen Ayres & Associates, Inc., Ft. Collins, CO               *
*****************************************************************************************
TABLE 2.1 SEDIMENT DISCHARGE TABLE.
          Q,CFS    =          10.0     20.0     50.0     100.0     200.0
    QS,TONS/DAY    =   0.0      0.0         0.0                0.0       2.3

          Q,CFS    =          500.0    1100.0            2000.0    5000.0    10000.0
    QS,TONS/DAY    =   42.7    283.1            1074.6   4428.0    11178.0
TABLE 2.2 FLOW-DURATION TABLE
    #    CFS      %        #    CFS        %         #     CFS        %

    1       0.00    97.10  5      137.00     15.90     9    3090.00    0.25
    2      20.10    84.10  6      442.00      6.00    10    9000.00    0.00
    3      22.00    50.00  7      988.00      2.30
    4      44.20    50.00  8     1545.00      1.00

TABLE 2.3 INTEGRATION PARAMETERS FOR FLOW-DURATION OPTION
                        MINIMUM FLOW, CFS    =          0.00
                        MAXIMUM FLOW, CFS    =       9000.00
                 INTEGRATION INTERVAL, CFS    =         24.66
               NUMBER OF INTEGRATION STEPS    =           365
TABLE 2.7 DENSITY OF SEDIMENT DEPOSIT.
      IN LB/CUFT         =          93.00
      IN CY/TON          =           0.80
     TABLE 3.1 CALCULATED YIELDS
     SEDIMENT TRANSPORT FUNCTION USED -- MPM(1948),D50
    TIME PERIOD,           DAYS     =   354.415
    WATER YIELD,           ACFT     =   84445.,     Mean Daily Flow,     CFS     =  120.13
    SEDIMENT YIELD,        TONS     =   10677.,       Mean Daily Load,    T/D    =  30.
                           CUYD     =      8504.,     Mean Daily Conc,  mg/l     =  92.880
TABLE 3.2 DISTRIBUTION OF YIELD BY WATER DISCHARGE CLASS INTERVAL.
     NO. OF CLASSES    =       45      ,       CLASS INTERVAL =       200.00
     MINIMUM Q, CFS    =       0.00,   MAXIMUM Q,     CFS     =       9000.00
```

Table 5–12 Effective discharge calculation from SAM program—Continued

Class	Discharge ft^3/s	Sediment tons/d	Increment of water		Increment of sediment		
			acre-ft	%	%	tons	yd^3
	0	0					
1			12	0.01	0	0	0
	200	2					
2			14263	16.89	1.2	128	102
	400	21					
3			11422	13.53	3.0	320	255
	600	66					
4			8311	9.84	3.87	414	329
	800	132					
5			6288	7.45	4.19	448	356
	1000	225					
6			7033	8.33	6.18	660	526
	1200	344					
7			5136	6.08	5.6	598	476
	1400	485					
8			3993	4.73	5.2	555	442
	1600	653					
9			3293	3.9	5.0	534	425
	1800	849					
10			2633	3.12	4.59	490	390
	2000	1075					
11			2154	2.55	4.11	439	349
	2200	1245					
12			1795	2.13	3.6	384	306
	2400	1424					
13			1518	1.8	3.19	340	271
	2600	1612					
14			1301	1.54	2.85	304	242
	2800	1807					
15			1128	1.34	2.57	274	218
	3000	2011					
16			719	0.85	1.7	181	144
	3200	2222					
17			464	0.55	1.13	121	96
	3400	2440					
18			464	0.55	1.17	125	99
	3600	2665					
19			464	0.55	1.21	129	103

Table 5–12 Effective discharge calculation from SAM program—Continued

Class	Discharge ft^3/s	Sediment tons/d	Increment of water		Increment of sediment		
			acre-ft	%	%	tons	yd^3
	3800	2897					
20			464	0.55	1.24	132	105
	4000	3137					
21			464	0.55	1.27	136	108
	4200	3382					
22			464	0.55	1.31	140	111
	4400	3634					
23			464	0.55	1.34	143	114
	4600	3893					
24			464	0.55	1.37	147	117
	4800	4157					
25			464	0.55	1.41	150	119
	5000	4428					
26			464	0.55	1.43	153	122
	5200	4666					
27			464	0.55	1.45	155	123
	5400	4907					
28			464	0.55	1.47	157	125
	5600	5152					
29			464	0.55	1.48	159	126
	5800	5399					
30			464	0.55	1.5	160	128
	6000	5649					
31			464	0.55	1.52	162	129
	6200	5902					
32			464	0.55	1.54	164	131
	6400	6158					
33			464	0.55	1.55	166	132
	6600	6416					
34			464	0.55	1.57	167	133
	6800	6677					
35			464	0.55	1.58	169	135
	7000	6941					
36			464	0.55	1.6	171	136
	7200	7207					
37			464	0.55	1.61	172	137
	7400	7476					
38			464	0.55	1.63	174	138
	7600	7747					
39			464	0.55	1.64	175	140
	7800	8021					
40			464	0.55	1.66	177	141

Table 5–12 Effective discharge calculation from SAM program—Continued

Class	Discharge ft^3/s	Sediment tons/d	Increment of water		Increment of sediment		
			acre-ft	%	%	tons	yd^3
	8000	8297					
41			464	0.55	1.67	178	142
	8200	8575					
42			464	0.55	1.68	180	143
	8400	8855					
43			464	0.55	1.7	181	144
	8600	9138					
44			464	0.55	1.71	183	146
	8800	9423					
45			464	0.55	1.72	184	147
	9000	9710					
Total			84445	100	100	10677	8504

(a) Cautions and limitations

In addition to the previously noted limitations associated with the methods of estimation, other precautions should be applied to the entire nature of channel-forming discharge. The channel-forming discharge concept is based on the idea that there exists a single steady discharge that, given enough time, would produce channel dimensions equivalent to those produced by the natural long-term hydrograph. Although conceptually attractive, this definition is not necessarily physically feasible because riparian vegetation, bank stability, and even the bed configuration would be different in a natural stream than in a stream with a constant discharge (Copeland et al. 2001).

In addition, it is important to note that extreme events often have the capability to move a significant amount of sediment and cause major changes in channel cross section, profile, and planform. In streams that have experienced catastrophic events, the flow-frequency and sediment-transport relations may have changed or be changing with time as the channel adjusts. Results obtained using any technique may represent a condition that does not accurately depict present flow and sediment-transport conditions.

To design a stream restoration project with long-term stability, it is necessary to evaluate the full range of flows that will affect the channel. Therefore, stable channel design includes the evaluation of sediment transport capacity for a range of flows (not just the design discharge) to determine whether the project will aggrade or degrade, as well as meet other objectives for the restoration.

654.0509 Other sources of design flows

Other sources may include estimates of local flows. These can range from hydrologic models conducted as part of another study to historical records of extreme events. Regulatory or legislatively defined flows may be defined. Data may be available for irrigation releases, dam operations or navigation controls. It may be necessary to include an analysis of some or all of these flows. However, it is important to review this information to assess both the technical accuracy, as well as the assumptions made in their estimate. Additional calculations are often required.

654.0510 Conclusion

Rarely does the behavior of a channel under a single discharge adequately reflect the range of design conditions required for a stream restoration project. The design capacity of the channel should consider environmental objectives, as well as flood criteria. Often, habitat features are designed to narrow the channel during summer low flows to increase habitat during a biologically critical period. Project features are designed to withstand a significant flood event, normally a 10-year frequency discharge or larger. A realigned channel is normally designed to convey a flow selected for channel stability, normally larger than the 1-year frequency discharge. If the stream channel is realigned or reconstructed, a suitable design discharge must be selected. In many situations, this is the channel-forming discharge. A wide variety of sources and techniques exist for obtaining hydrologic data available to the designer. If a gage is available and the conditions applicable, a gage analysis is generally considered preferable, since it represents actual data for the stream. However, it is important to assess the applicability of the historic gage data to the current project conditions. For example, rapid increases of imperviousness in an urban watershed may have increased flows and resulted in stream instability. If this is the case, the historic gage data must not be used, because there is no realistic way to adjust the peak flow frequency to predevelopment conditions. Changes in rainfall-runoff characteristics may render historic gage data obsolete. Gage records provide an actual representation of the hydrologic behavior of a watershed. However, when a gage record is of short duration, or poor quality, or the results are judged to be inconsistent with field observations or sound judgment, then the analysis of the gage record should be supplemented with other methods.

Several state and local agencies have developed regional regression relations to estimate peak discharges at ungaged sites. These data can be readily applied, but care must be taken to assure that the regression relations include relevant parameters that can relate the unique characteristics of the study watershed to the data that were used to create the relations. Care must also be taken to make sure that the watershed parameters of the ungaged watershed being analyzed are within the watershed parameters used to develop the regression curves. It is also important to assess the relevance of the confidence limits of the estimates to the project analysis.

Hydrologic models provide the ability to estimate existing, as well as future rainfall runoff patterns for a variety of conditions. The use of models is preferred in cases where the watershed has changed. The accuracy of models is dependent on calibration data, which can be difficult to acquire. However, if the issues to be addressed are comparative in nature rather than absolute, the importance of calibration is diminished. The level of accuracy required for a specific hydrologic analysis generally depends on the specific characteristics of each individual project. The appropriate methodology should be selected with a firm understanding of the assumptions, accuracy, data requirements, and limitations of the approach. The designer should consult with a hydraulic engineer before deciding on which procedure should be used to obtain the needed flow data.

Channel-forming discharge can be estimated using a prescribed methodology. All methodologies for estimating channel-forming discharge present challenges. The practitioner should review the assumptions, data requirements and consider his or her experience when determining which technique to use. It is recommended that all available methods be used and cross checked against each other to reduce the uncertainty in the final estimate of the channel-forming flow.

United States
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**Natural
Resources
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Part 654 Stream Restoration Design National Engineering Handbook

Chapter 6 Stream Hydraulics

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Cover photo: Stream hydraulics focus on bankfull frequencies, velocities, and duration of flow, both for the current condition, as well as the condition anticipated with the project in place. Effects of vegetation are considered both in terms of protection of the bank materials, as well as on changes in hydraulic roughness.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 6 Stream Hydraulics

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Chapter 6 Stream Hydraulics

654.0600 Purpose

Human intervention in the stream environment, especially with projects intended to restore a stream ecosystem to some healthier state, must fully consider the stream system, stream geomorphology, stream ecology, stream hydraulics, and the science and mechanics of streamflow. This chapter provides working professionals with practical information about hydraulic parameters and associated computations. It provides example calculations, as well as information about the role of hydraulic engineers in the design process.

The hydraulic parameters used to evaluate and quantify streamflow are described in this chapter. The applicability of the various hydraulic parameters in planning and design in the stream environment is presented. The complexity of streamflow is addressed, as well as simplifying assumptions, their validity, and consequences. Guidance is provided for determining the level of analysis commensurate with a given project's goals and the associated hydraulic parameters. Finally, a range of analytical tools is described, the application of which depends on the complexity of the project.

Stream hydraulics is a complex subject, however, and this chapter does not provide exhaustive coverage of the topic. Readers are encouraged to supplement this information with the many good references that are available.

654.0601 Introduction

Stream hydraulics is the combination of science and engineering for determining streamflow behavior at specific locations for purposes including solving problems that generally originate with human impacts. A location of interest may be spatially limited, such as at a bridge, or on a larger scale such as a series of channel bends where the streambanks are eroding. Flood depth, as well as other hydraulic effects, may need to be determined over long stretches of the channel.

An understanding of flowing water forms the basis for much of the work done to restore streams. The discipline of hydrology involves the determination of flow rates or amounts, their origin, and their frequency. Hydraulics involves the mechanics of the flow and, given the great power of flowing water, its affect on bed, banks, and structures.

A stream is a natural system that constantly adjusts itself to its environment and participates in a cycle of action and reaction. These adjustments may be gradual, less noticeable, and long term, or they may be sudden and attention grabbing. The impacts causing a stream to react may be natural, such as a rare, intense rainfall, or human-induced, such as the straightening of a channel or filling of a wetland. However, the reaction of a stream to either kind of change may be more than localized. A stream adjusts its profile, slope, sinuosity, channel shape, flow velocity, and boundary roughness over long sections of its profile in response to such impacts. After an impact, a stream may restore a state of equilibrium in as little as a week, or it may take decades.

(a) Hydraulics as physics

Stream characteristics are derived from the basic physics of flowing water. Fluid mechanics is an old science with well-established physical relationships. Typically, simple empirical equations are used that do not account for all the variability that occurs in the flow. An example is Bernoulli's equation for balancing flow depth, velocity, and pressure. In this case, the flow must be considered steady. If it is important to assess how flow depth, velocity, and/or pressure

change over time, Bernoulli's equation by itself will not be sufficient.

The assumption that flow velocity is generally downstream in direction is also a common simplification in the analysis of streamflow. Real streams have many eddies where the flow circulates horizontally. Streams also have areas of upwelling, roiling, and vertical circulation. While designers commonly make use of an average velocity at a given cross section, the actual velocities in the plane of a cross section vary markedly from top to bottom, side to side, and in direction, varying with time and three-dimensional space.

Water surface profile analyses generally assume a constant flow elevation across a given cross section. Real streams, however, super-elevate their water surfaces in curved channel sections and may set up significant surface wave patterns that defy prediction. Finally, hydraulic analyses often assume that water flows against a fixed boundary. Real streams actually readjust their bed and banks constantly, move significant amounts of sediment, and transport unpredictable amounts of natural or humanmade debris.

It is, therefore, important to understand the limitations and restrictions of any equations before using them to obtain necessary information.

(b) Hydraulics as empiricism

Although thoroughly founded in physics, many hydraulic relationships require empirical coefficients to account for unmeasured or estimated processes. One of the parameters that has a significant influence on hydraulic calculations is surface roughness, in the form of Manning's n value, the Chézy C, or the Darcy-Weisbach friction factor. While the Darcy-Weisbach friction factor is generally considered to be more theoretically based, Manning's n is more commonly used for most stream design and restoration analysis. Roughness is a function of many stream physical properties including bed sediment size, vegetation, channel sinuosity, channel irregularity, and suspended sediment load. As a result, many of the estimates have inherent degrees of empiricism in their estimate.

Sediment transport also requires empirical input. Sediment particles vary in size and properties, from tiny silt particles that adhere to large boulders, sometimes redirecting a stream and sometimes transported downstream. Sediment transport is influenced by velocity vectors near the water/sediment boundary, and these bed velocities may not be well predicted by an average cross-sectional velocity. Many of the analytical sediment predictive techniques include many empirical estimates of specific parameters. More information on the analytical, as well as empirical approaches to sediment transport, is provided in other chapters of this handbook. More information on sedimentation analysis is provided in NEH654.09 and NEH654.13.

645.0602 Channel cross-sectional parameters

A variety of channel cross-sectional parameters are used in the hydraulic analysis of streams and rivers. It is important to measure and use these parameters consistently and accurately. A generalized cross section is shown in figure 6–1.

The flow depth is the distance between the channel bottom and the water surface. For rectangular channels, the depth is the same across an entire cross section, but it obviously varies in natural channels. Depth is often measured relative to the channel thalweg (or lowest point). Normal depth is the depth of flow in a uniform channel for which the water surface is normal or parallel to the channel profile and energy slope.

For a cross section aligned so that streamlines of flow are perpendicular, the flow area is the area of the cross section between bed and banks and water surface. For a rectangular channel, flow area is depth multiplied by top width. For a natural channel cross section, the area may be approximated with the sum of trapezoidal areas between cross-sectional points. The top width of a channel cross section at the water surface, typically designated as T, is a factor in the hydraulic depth.

The hydraulic depth is the ratio of the cross-sectional area of flow to the free water surface or top width. The hydraulic depth, d, is generally used either in computing the Froude number or in computing the section factor for critical depth. Since only one critical depth is possible for a given discharge in a channel, the section factor, Z, can be used to easily determine it (Chow 1959).

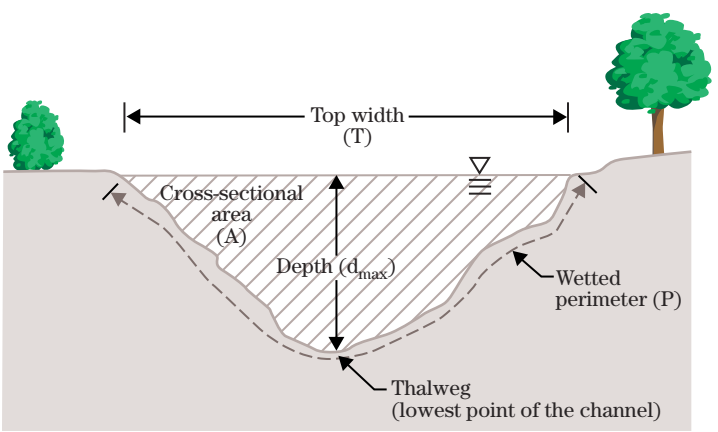
$$Z = \frac{A}{\sqrt{d}} \qquad \text{(eq. 6–1)}$$

$$Q_{critical} = Z\sqrt{g} \qquad \text{(eq. 6–2)}$$

For a cross section normal to the direction of flow, the wetted perimeter (typically designated P) is the length of cross-sectional boundary between water and bed and banks. The hydraulic radius is the ratio of the cross-sectional area of flow to the wetted perimeter or flow boundary. The hydraulic radius, R= A/P, is used in Manning's equation for calculation of normal depth discharge, as well as for calculation of shear velocity.

Velocity is a physics term for a change in distance during a time interval. Flow velocity refers to the areal extent of the flow (in a cross section) for which a velocity is specified. For example, an average velocity that applies to an entire cross-sectional area may be determined from V = Q/A or if the discharge is unknown, a uniform flow velocity may be determined from Manning's equation.

Figure 6–1 Channel cross-sectional parameters (per ft of channel length)



Another useful formulation is critical velocity, which is average flow velocity at critical depth, and is calculated from equation 6–3:

$$V_{cr} = \sqrt{gd_{cr}} \qquad \text{(eq. 6–3)}$$

where:
V_{cr} = critical velocity
g = gravitational acceleration
d_{cr} = critical depth

Determining the state of flow is a matter of determining whether the velocity is greater than critical velocity V_{cr} (supercritical flow) or less than critical velocity V_{cr} (subcritical flow).

Conveyance is a measure of the flow-carrying capacity of a cross section which is directly proportional to discharge. Conveyance, typically designated K, may be expressed from Manning's equation (without the slope term) as:

$$K = \frac{1.486}{n} AR^{\frac{2}{3}} \qquad \text{(eq. 6–4a)}$$

or

$$K = \frac{Q}{\sqrt{S}} \qquad \text{(eq. 6–4b)}$$

where:
A = flow area (ft^2)
R = hydraulic radius (ft)
Q = flow rate (ft^3/s)
S = slope, dimensionless

In backwater calculations, change in conveyance from cross section to cross section is a useful way to determine the adequacy of section spacing in a stream reach. Within a cross section, conveyance may be used to compare channel and overbank flow carrying capacity.

654.0603 Dimensionless ratios

Dimensionless ratios (also referred to as dimensionless numbers) are used to provide information on flow condition. The units of the variables used in the equation for a dimensionless ratio are such that they cancel. The two most commonly used ratios are Froude and Reynolds numbers. Being dimensionless allows their application to be made across a variety of scales.

(a) Froude number

The Froude number is a dimensionless ratio, relating inertial forces to gravitational forces. The Froude number represents the effect of gravity on the state of flow in a stream (Chow 1959). This useful number was derived by a nineteenth century English scientist, William Froude, who studied the resistance of ships being towed in water. He observed wave patterns along the hull of a moving ship and found that the same number of waves would occur as long as the ratio of the ship's speed to the square root of its length were the same. Applied in hydraulics, the length is replaced by hydraulic depth, as shown in equation 6–5.

$$F = \frac{V}{\sqrt{gd}} \qquad \text{(eq. 6–5)}$$

where:
V = velocity (ft/s)
g = acceleration due to gravity (32.2 ft/s^2)
d = flow depth (ft)

If the Froude number is less than one, gravitational forces dominate and the flow is subcritical, and if greater than one, inertial forces dominate and the flow is supercritical. The Froude number is used to determine the state of flow, since, for subcritical flow the boundary condition is downstream, and for supercritical flow it is upstream. When the Froude number equals one, the flow is at the critical state.

(b) Reynolds number

The Reynolds number is also a dimensionless ratio, relating the effect of viscosity to inertia, used to determine whether fluid flow is laminar or turbulent (Chow 1959). The Reynolds number relates inertial forces to

viscous forces and was derived by a nineteenth century English scientist, Osborne Reynolds, for use in wind tunnel experiments.

Inertia is represented in equation 6–6 by the product of velocity and hydraulic radius, divided by the kinematic viscosity of water, with units of length squared per time. For turbulent flow Re>2000, for laminar, Re<500, and values between these limits are identified as transitional.

$$Re = \frac{VR}{\nu} \qquad \text{(eq. 6–6)}$$

where:
V = velocity (ft/s)
R = hydraulic radius (ft)
ν = kinematic viscosity (ft^2/s)

For use in sediment transport analysis, the Reynolds number has been formulated to apply at the water-sediment boundary. In this case, the velocity is local to the boundary and termed shear velocity (V_*). Also, the length term is not the hydraulic radius, but roughness height, or the diameter of particles (D) forming the boundary. This boundary Reynolds number has also been called the bed Reynolds number or shear Reynolds number.

$$Re_{bed} = \frac{V_* D}{\nu} \qquad \text{(eq. 6–7)}$$

where:
V_* = boundary shear velocity (ft/s)
D = particle diameter
ν = kinematic viscosity (ft^2/s)

Because streamflow is almost exclusively turbulent, the Reynolds number is not needed as a flag of turbulence. The Reynolds number has value for sedimentation analyses in that drag coefficients have been empirically related to Reynolds number. Another important use in sedimentation involves incipient motion of sediment particles. Studies have related the bed Reynolds number to critical shear stress (the initiation point of sediment movement). Through the Shields diagram, for example, one can determine critical shear, given a bed Reynolds number. Additional information on this topic is provided in NEH654.13.

654.0604 Continuity

Open channel flow has a liquid surface that is open to the atmosphere. This boundary is not fixed by the physical boundaries of a closed conduit. Water is essentially an incompressible fluid, so it must increase or decrease its velocity and depth to adjust to the channel shape. If no water enters or leaves a stream (a simplification that can be made over short distances) the quantity of the flow will be the same from section to section. Since the flow is incompressible, the product of the velocity and cross-sectional area is a constant. This conservation of mass can be written as the continuity equation as follows:

$$Q = VA \qquad \text{(eq. 6–8)}$$

While the continuity equation can be used with any consistent set of units, it is normally expressed as:

Q = quantity of flow (ft^3/s)
A = cross-sectional area ($ft^{2)}$)
V = average velocity (ft/s)

624.0605 Energy

Energy, an abstract quantity basic to many areas of physics, is a property of a body or physical system that enables it to move against a force. It is an expression of work, which is force applied over a distance. Energy is the amount of work required to move a mass through a distance. Or, it is the amount of work a physical system is capable of doing, in changing from its actual state to some specified reference state.

Many useful concepts of energy exist, the primary one being that, in a closed system, the total energy is constant, the concept of conservation of energy. Water energy is comprised of a number of components, often called head and expressed as a vertical distance. The potential energy of water, or pressure head, is a result of its mass and the Earth's gravitational pull. The kinetic energy of water is related to its movement and is called the velocity head.

The Bernoulli equation (eq. 6–9) is an expression of the conservation of energy.

$$z_1 + y_1 + \alpha_1 \frac{V_1^2}{2g} = z_2 + y_2 + \alpha_2 \frac{V_2^2}{2g} + h_L \qquad \text{(eq. 6–9)}$$

This expression shows the interrelationship of these energy terms, between two cross sections (1 and 2). Each term represents a form of energy, with depth y representing potential energy, the velocity term V representing kinetic energy, and z, a potential energy term relating all to a common datum in a plane perpendicular to the direction of gravity. The head loss or h_L term is called a loss because any energy consumed between the two cross sections must be made up for by a change in height (or head). The head loss is the energy consumed by boundary friction, turbulence, eddies, or sediment transport. The velocity term represents velocity head and the depth term the pressure head.

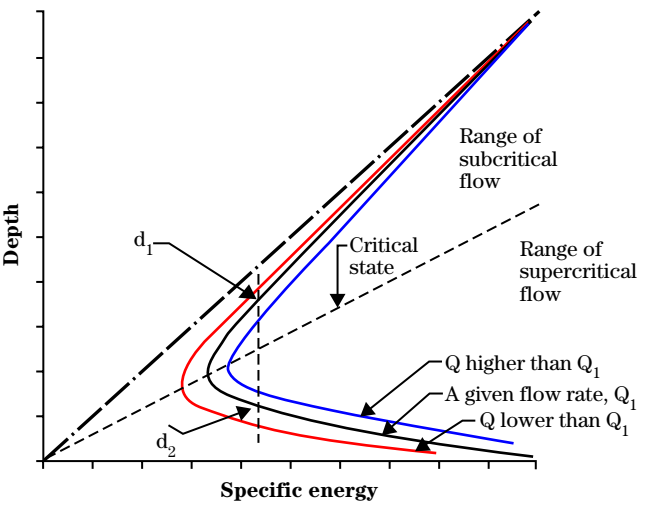
Although energy is a scalar quantity, without direction, the concept of energy as head has an orientation in the direction of gravity. Pressure, however, represents the magnitude of a force in the direction of whatever surface it impinges. So, as a channel slope steepens, the orientation of the pressure head is technically moving further from vertical. It is represented by the depth times the cosine of the slope angle. For most natural channels, the channel slope is sufficiently gradual for this angle to be small enough to be ignored. However, in slopes that are greater than 10 percent, this may become an issue that should be addressed.

Another assumption is that flow is always perpendicular to the cross sections. Finally, alpha (α) in the equation is the energy coefficient, and it varies with the uniformity of velocity vectors in the cross section. For a fairly uniform velocity, alpha may be taken to be one. If velocity varies markedly over the cross section, alpha may go as high as 1.1 in sections of sudden expansion or contraction (Chow 1959).

Specific energy is a particular concept in hydraulics defined as the energy per unit weight of water at a given cross section with respect to the channel bottom.

As shown in figure 6–2, specific energy can be helpful in visualizing flow states of a stream. The points d_1 and d_2 are alternate depths for the same energy level. Only one depth exists at the critical state, which is the lowest possible energy level for a given discharge. In natural streams, this is an unstable state since a very

Figure 6–2 Specific energy vs. depth of flow



small change in energy results in a relatively significant undulating change in depth. An understanding of flow energy is fundamental in hydraulic modeling.

The specific energy at any cross section for a channel of small slope (most natural channels) and $\alpha = 1$ is:

$$E = y + \frac{V^2}{2g} \qquad \text{(eq. 6–10)}$$

654.0606 Momentum

In basic physics, momentum is the mass of a body times its velocity and is a vector quantity, whereas energy is scalar, lacking a direction. In hydraulics, the use of this concept is due mainly to the implication of Newton's second law, that the resultant of all forces acting on a body causes a change in momentum. The momentum equation in hydraulics is similar in form to the energy equation and, when applied to many flow problems, can provide nearly identical results. However, knowledge of fundamental differences in the two concepts is critical to modeling certain hydraulic problems. Conceptually, the momentum approach should be thought of as involving forces on a mass of flowing water, instead of the energy state at a particular location. Friction losses in momentum relate to the force resistance met by that mass with its boundary, whereas in the energy concept, losses are due to internal energy dissipation (Chow 1959).

The momentum equation can have advantages in modeling flow over weirs, drops, hydraulic jumps, and junctions, where the predominate friction losses are due to external forces, rather than internal energy dissipation.

Interpreted for open channel, Newton's second law states that the rate of momentum change in this short section of channel equals the sum of the momentum of flow entering and leaving the section and the sum of the forces acting on the water in the section. Since momentum is mass times velocity, the rate of change of momentum is the mass rate of change times the velocity. The momentum equation may be written considering a small mass or slug of flowing water between two sections 1 and 2 and the principle of conservation of momentum.

$$\rho Q\left(\beta_2 V_2 - \beta_1 V_1\right) = P_1 - P_2 + W \sin\theta - F_{fr} \qquad \text{(eq. 6–11)}$$

The left side of the equation is the momentum entering and leaving, and the right side is the pressure force at each end of the mass, with $W\sin\theta$ being the weight of the mass, θ being the angle of the bottom slope of the channel, and F_{fr} being the resistance force of friction on the bed and banks.

654.0607 Specific force

Specific force is the horizontal force of flowing water per unit weight of water. It is derived from the momentum equation. A specific force curve looks similar to the specific energy curve. The critical depth occurs both at the minimum energy for a given discharge and also at the minimum specific force for a given discharge. This similarity shows how energy concepts and force or momentum concepts can be employed similarly in many hydraulic analyses, often with nearly identical results.

The designer should know what circumstances would cause the two approaches to diverge, however. Specific force concepts are applied over short horizontal reaches of channel, where the difference in external friction forces and force due to the weight of water are negligible. Examples are the flow over a broad-crested weir through a hydraulic jump or at junctions. One way to conceptualize why a momentum-based method, rather than an energy-based method, might be more applicable would be to energy changes in a hydraulic jump. Much energy is lost through turbulence caused by moving mass colliding with other mass that is not accounted for by energy principles alone.

An equation for specific force may be derived from the momentum equation. If the practitioner wishes to apply this equation to short sections of channel such as a weir or hydraulic jump, the frictional resistance forces, F_{fr} can be neglected. With a flat channel of low slope, θ approaches 0, then the last two terms in equation 6–12 can be dropped. As a result, equation 16–11 becomes:

$$\rho Q\left(\beta_2 V_2 - \beta_1 V_1\right) = P_1 - P_2 \qquad \text{(eq. 6–12)}$$

Assume also that the Boussinesq coefficient (β) is 1. From the fact that the pressure increases with depth to the maximum of ρgy at the channel bottom (y being depth, b being channel width, and ρ being fluid density), the overall pressure on the vertical flow area may be expressed as $1/2\rho gby^2$. The velocities may be expressed as Q/A. For a rectangular channel:

$$\rho Q\left(\frac{Q}{A_2} - \frac{Q}{A_1}\right) = \frac{\rho g}{2}\left(A_1 y_1 - A_2 y_2\right) \qquad \text{(eq. 6–13)}$$

that becomes:

$$\frac{2Q^2}{gA_1} + A_1 y_1 = \frac{2Q^2}{gA_2} + A_2 y_2 \qquad \text{(eq. 6–14)}$$

For a channel section of any other shape, the resultant pressure may be taken at the centroid of the flow area, at a depth, z, from the surface. Then the momentum formulation is:

$$\frac{Q^2}{gA_1} + A_1 z_1 = \frac{Q^2}{gA_2} + A_2 z_2 \qquad \text{(eq. 6–15)}$$

Either side of this equation is the definition of specific force, and the specific force is constant over a short stretch of channel such as a hydraulic jump. The first term represents change in momentum over time, and the second term the force of the water mass. As Chow (1959) explains, specific force is sometimes called force plus momentum or momentum flux.

654.0608 Stream power

Stream power is a geomorphology concept that is a measure of the available energy a stream has for moving sediment, rock, or woody material. For a cross section, the total stream power per unit length of channel may be formulated as:

$$\Omega = \gamma Q S_f = \gamma v w d S_f \qquad \text{(eq. 6–16)}$$

where:
γ = unit weight of water (lb/ft^3)
Q = discharge (ft^3/s)
S_f = energy slope (ft/ft)
v = velocity (ft/s)
w = channel width (ft)
d = hydraulic depth (ft)

English units are pounds per second per foot of channel length. A second formulation, unit stream power, is the stream power per unit of bed area:

$$\underline{\Omega} = \tau_0 v \qquad \text{(eq. 6–17)}$$

where:
τ = bed shear stress
v = average velocity

A third formulation relates stream power per unit weight of water:

$$\underline{\Omega} = S_f v \qquad \text{(eq. 6–18)}$$

where the terms are as previously defined.

654.0609 Hydraulic computations

(a) Uniform flow

Water flowing in an open channel typically gains kinetic energy as it flows from a higher elevation to a lower elevation. It loses energy with friction and obstructions. Uniform flow occurs when the gravitational forces that are pushing the flow along the channel are in balance with the frictional forces exerted by the wetted perimeter that are retarding the flow. For uniform flow to exist:

- Mean velocity is constant from section to section.
- Depth of flow is constant from section to section.
- Area of flow is constant from section to section.

Therefore, uniform flow can only truly occur in very long, straight, prismatic channels where the terminal velocity of the flow is achieved. In many cases, the flow only approaches uniform flow.

Since uniform flow occurs when the gravitational forces are exactly offset by the resistance forces, a resistance equation can be used to calculate a velocity. The most commonly used resistance equation is Manning's equation (eq. 6–19).

$$Q = \frac{1.486}{n} A R^{\frac{2}{3}} S \qquad \text{(eq. 6–19)}$$

given $Q = VA$

then
$$V = \frac{1.486}{n} R^{\frac{2}{3}} S^{\frac{1}{2}} \qquad \text{(eq. 6–20)}$$

where:
A = flow area (ft^2)
R = hydraulic radius (ft)
S = channel profile slope (ft/ft)
n = roughness coefficient

The 1.486 exponent is replaced by 1.0 if SI units are used. The flow area (A) and the hydraulic radius (R) relate how the flow interacts with the boundary.

A rough estimate of the flow capacity or average velocity at a natural cross section may be determined with Manning's equation. A designer may assume a roughly trapezoidal cross section, estimating bottom width, side slopes, and profile slope from topographic maps. The roughness coefficient is a significant factor, and its determination is described in NEH654.0609(c).

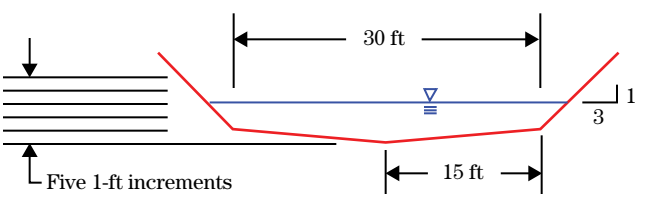
(b) Determining normal depth

Normal depth calculation is one of the most commonly used analyses in stream restoration assessment and design. Several spreadsheets, computer programs, and nomographs are available for use in calculating normal depth. In a natural channel, with a nonuniform cross section, reliability of the normal depth calculation is directly related to the reliability of the input data. Sound engineering judgment is required in the selection of a representative cross section. The cross section should be located in a uniform reach where flow is essentially parallel to the bank line (no reverse flow or eddies). This typically occurs at a crossing or riffle.

Determination of the average energy slope can be difficult. If the channel cross section and roughness are relatively uniform, surface slope can be used. Thalweg slopes and low-flow water surface slopes may not be representative of the energy slope at design flows. Slope estimates should be made over a significant length of the stream (a meander wavelength or 20 channel widths). Hydraulic roughness is estimated based on field observations and measurements.

In addition to normal depth for a given discharge, these same procedures may be used to estimate average velocities in the cross section. These calculations do not account for backwater in a channel reach. The following example calculation refers to the cross section shown in figure 6–3.

Figure 6–3 Problem cross section



Example problem: Normal depth rating curve calculation

Problem 1: Calculate a normal depth rating curve for each foot of depth up to 5 feet. Assume channel slope = 0.0015 and an n value = 0.03

Solution:

For

$$Q = \left(\frac{1.49}{n}\right) A R^{\frac{2}{3}} S^{0.5} \quad ,$$

the value

$$\left(\frac{1.49}{n}\right) S^{0.5} = 1.924 \quad .$$

A and P need to be determined.

$$R = \frac{A}{P}$$

$$A_1 = 30 \times \frac{1}{2} = 15 \text{ ft}^2$$

$$P_1 = 2\left(15^2 + 1^2\right)^{0.5} = 30.07 \text{ ft}$$

$$R_1 = 0.499 \text{ ft}$$

$$A_2 = 15 + (30 \times 1) + (1 \times 3) = 48 \text{ ft}^2$$

$$P_2 = 30.07 + 2\left(3^2 + 1^2\right)^{0.5} = 36.39 \text{ ft}$$

$$R_2 = 1.319 \text{ ft}$$

$$A_3 = 48 + (36 \times 1) + (1 \times 3) = 87 \text{ ft}^2$$

$$P_3 = 36.39 + 2\left(3^2 + 1^2\right)^{0.5} = 42.71 \text{ ft}$$

$$R_3 = 2.037 \text{ ft}$$

$$A_4 = 87 + (42 \times 1) + (1 \times 3) = 132 \text{ ft}^2$$

$$P_4 = 42.71 + 2\left(3^2 + 1^2\right)^{0.5} = 49.03 \text{ ft}$$

$$R_4 = 2.692 \text{ ft}$$

$$A_5 = 132 + (48 \times 1) + (1 \times 3) = 183 \text{ ft}^2$$

$$P_5 = 49.03 + 2\left(3^2 + 1^2\right)^{0.5} = 55.36 \text{ ft}$$

$$R_5 = 3.306 \text{ ft}$$

Solving for Q, then:

$$Q_1 = 1.924 \times 15 \times (0.499)^{0.667} = 18.1 \text{ ft}^3/\text{s} \quad (\text{at d} = 1 \text{ ft})$$

$$Q_2 = 1.924 \times 48 \times (1.319)^{0.667} = 111.1 \text{ ft}^3/\text{s} \quad (\text{at d} = 2 \text{ ft})$$

$$Q_3 = 1.924 \times 87 \times (2.037)^{0.667} = 269.0 \text{ ft}^3/\text{s} \quad (\text{at d} = 3 \text{ ft})$$

$$Q_4 = 1.924 \times 132 \times (2.692)^{0.667} = 491.6 \text{ ft}^3/\text{s} \quad (\text{at d} = 4 \text{ ft})$$

$$Q_5 = 1.924 \times 183 \times (3.306)^{0.667} = 781.7 \text{ ft}^3/\text{s} \quad (\text{at d} = 5 \text{ ft})$$

Problem 2: Determine the normal depth for a discharge of 350 cubic feet per second and the associated average velocity.

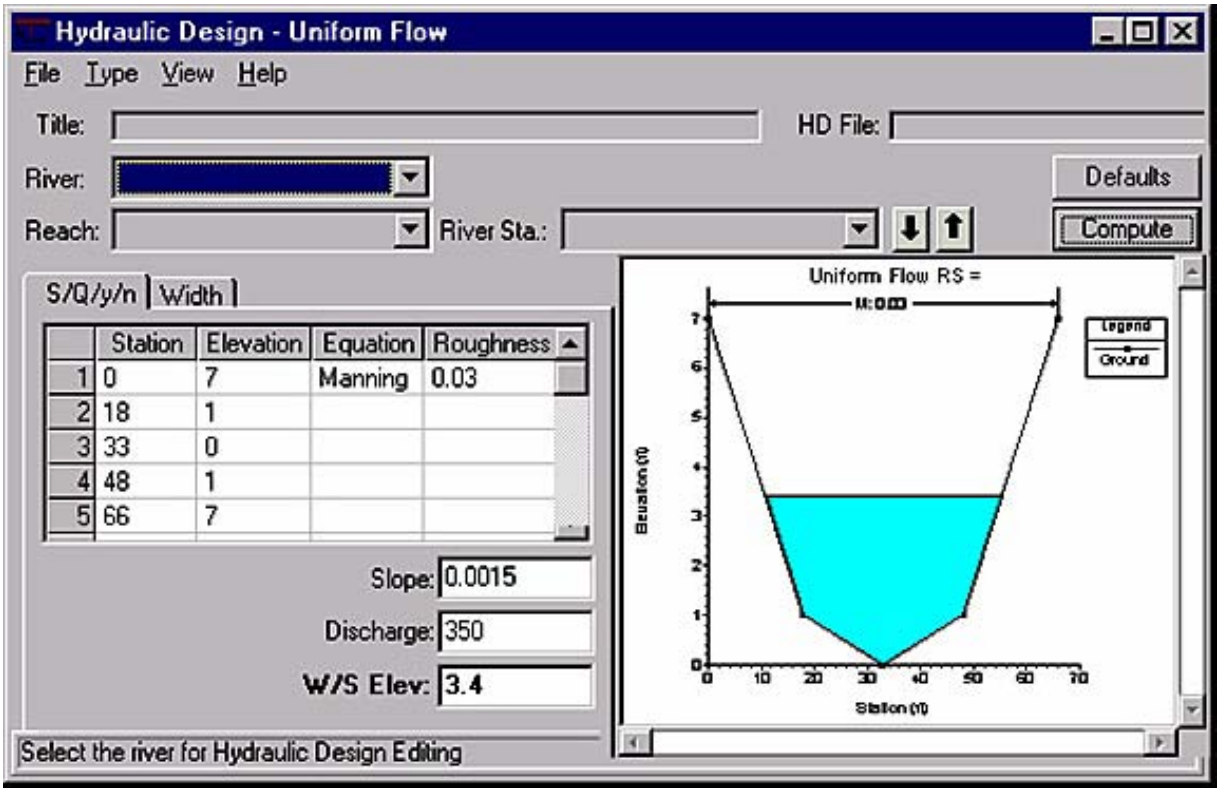
Solution: From the rating curve calculated above, the 350 cubic feet per second discharge in this problem will be between Q_3 and Q_4. A straight-line interpolation gives a depth of 3.4 feet.

For velocity, since $Q = VA$

$$V = \frac{350}{\left(3\left[(3.4 \times 3.4) + 8(3.4) - 4\right]\right)} = 3.36 \text{ ft/s}$$

Discussion:
The more complicated a section becomes, the more tedious is this hand calculation. Numerous computer programs, such as HEC–RAS (USACE 2001b), can perform normal depth calculations for a cross section of many coordinate points. A typical image from HEC–RAS is shown as figure 6–4.

Figure 6–4 HEC–RAS screen shot for uniform flow computation



(c) Determining roughness coefficient (*n* value)

The roughness coefficient, an empirical factor in Manning's equation, accounts for frictional resistance of the flow boundary. Estimating this flow resistance is not a simple matter. This parameter is used in computation of water surface profiles and estimation of normal depths and velocities.

Boundary friction factors must be chosen carefully, as hydraulic calculations are significantly influenced by the *n* choice. Factors affecting roughness include ground surface composition, vegetation, channel irregularity, channel alignment, aggradation or scouring, obstructions, size and shape of channel, stage and discharge, seasonal change, and sediment transport.

Significant guidance exists in the literature regarding roughness estimation. Chow (1959) discusses four general approaches for roughness determination. The U.S. Geological Survey (USGS) (Arcement and Schneider 1990) published an extensive step-by-step guide for determination of *n* values. NRCS guidance for channel *n* value determination is available from Faskin (1963). Finally, when observed flow data and stages are known, manual calculations or a computer program such as HEC–RAS may be used to determine *n* values.

With the many factors that impact roughness, and each stream combining different factors to different extents, no standard formula is available for use with measured information. As stated in Chow (1959):

> *...there is no exact method of selecting the n value. At the present stage of knowledge [1959], to select a value of n actually means to estimate the resistance to flow in a given channel, which is really a matter of intangibles. To veteran engineers, this means the exercise of sound engineering judgment and experience; for beginners, it can be no more than a guess, and different individuals will obtain different results.*

While there has been considerable research on estimating roughness coefficients since 1959, flood plain and channel *n* values are still challenging to determine. In practice, to a large extent the selection of Manning's *n* values remains judgement based.

Estimates of channel roughness may be made using photographs or tables provided by Chow (1959), Brater and King (1976), Faskin (1963), and Barnes (1967). NEH–5 supplement B, Hydraulics, can also be used to estimate roughness values. As roughness can change dramatically between surfaces within the same cross section, such as between channel and overbanks, a determination of a composite value for the cross section is necessary (Chow 1959). The choice of a channel compositing method is very important in stream restoration design where large differences exist in bank and bed roughness. While the following example uses the Lotter method, other methods, such as the equal velocity method and the conveyance method, can also be used.

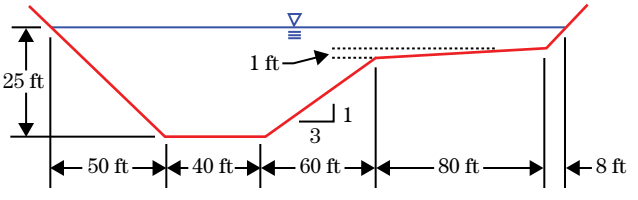
Example problem: Composite Manning's *n* value

Problem: Determine a composite *n* value for the cross section illustrated in figure 6–5 at the given depth of flow.

Assume that this channel is experiencing a 6,094 cubic feet per second flow, with 5,770 cubic feet per second in the main channel and the remainder on the right overbank. The mean velocity in the main channel is 2.3 feet per second and on the overbank, 0.55 foot per second. The channel slope is 0.00016, and a fairly regular profile of clay and silt is observed.

The channel is relatively straight and free of vegetation up to a stage of 10 feet. Above that level, both banks are lined with snags, shrubs, and overhanging trees. The right overbank is heavily timbered with standing trees up to 6 inches in diameter with significant forest litter. In stream work, the convention is that the

Figure 6–5 Cross-sectional dimensions



left bank is on the left when looking downstream. See figures 6–6 and 6–7 where the photos are taken at a lower stage (Barnes 1967).

Solution: To determine the composite Manning's n value, the inchannel and overbank n values must first be determined.

The solution will first estimate n values using reference materials, then this solution will compare this estimate with the value calculated from Manning's equation. Roughness estimates can be found in NEH–5, Hydraulics, supplement B by Cowan (1956). Arcement and Schneider (1990) extended this body of work. Both methods estimate a base n value for a straight, uniform, smooth channel in natural materials, then modifying values are added for channel irregularity, channel cross-sectional variation, obstructions, and vegetation. After these adjustments are totaled, an adjustment for meandering is also available.

For the channel below 10 feet, the bed material is silty clay. Arcement and Schneider (1990) show base n values for sand and gravel. For firm soil, their n value ranges from 0.025 to 0.032. Cowan (1956) shows a base n of 0.020 for earth channels. Richardson, Simons, and Lagasse (2001) shows 0.020 for alluvial silt and 0.025 for stiff clay. A reasonable assumption could be 0.024 for the channel below 10 feet of depth. For the remainder of the channel, above 10 feet of depth to top of bank at 20 feet, the effects of vegetation must be added in. The channel is then divided into three pieces: a lower channel, an upper channel, and a right overbank. Other breakdowns of this cross section are possible.

For the lower channel a base n value of 0.024 is assumed. Referring to Cowan (1956) in NEH 5, supplement B, a 0.005 can be added for minor irregularity and a 0.005 addition for a shifting cross section. This gives a total n value for the lower channel of 0.034.

Figure 6–6 Looking upstream from left bank

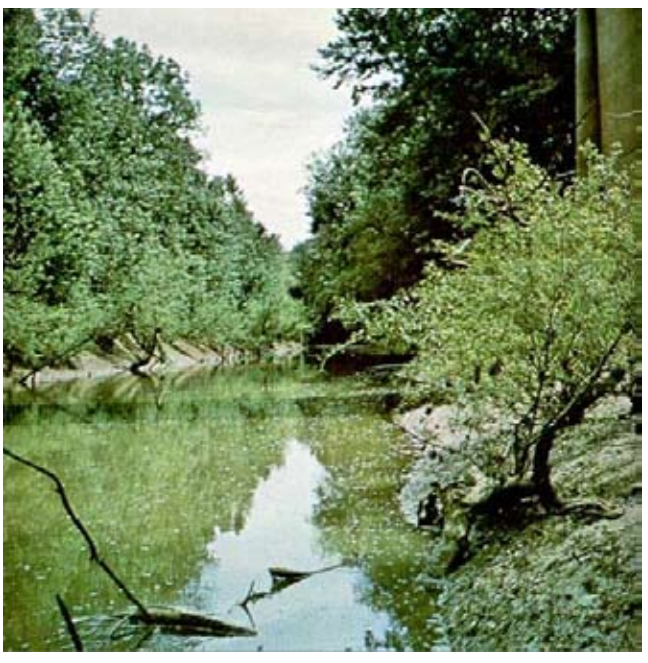
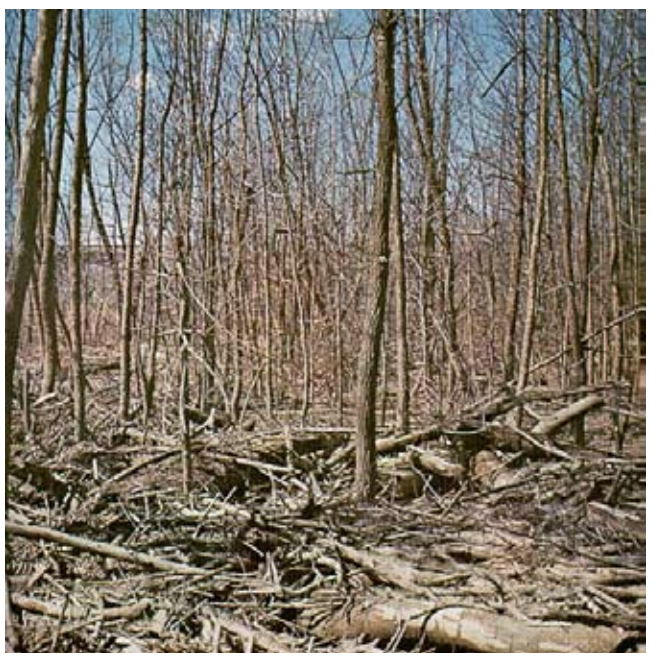


Figure 6–7 Looking downstream on right overbank



For the upper channel, the area above the lower 10 feet of flow depth and excluding the right overbank, the base n value is 0.024, a minor irregularity addition of 0.005, a 0.005 addition for a shifting cross section, a minor obstruction addition of 0.010, and a medium vegetation addition of 0.020 can be selected. This gives a total n value for the upper channel of 0.064.

For the overbank, a base n (from the overbank soil) is needed. Based on site-specific observations, it was found that the soil is slightly more coarse than that of the main channel, n = 0.027. Again from NEH 5, supplement B, Cowan (1956) a minor irregularity addition of 0.005, a shifting cross section addition of 0.005, an appreciable obstruction addition of 0.020, and a high vegetation addition of 0.030 can be selected. This gives a total n value for the overbank of 0.087.

To obtain composite roughness, use the method of Chow (1959), whereby a proportioning is done with wetted perimeter (P) and hydraulic radius (R):

$$n = \frac{PR^{\frac{5}{3}}}{\sum_{1}^{N}\left(\frac{P_N R_N^{\frac{5}{3}}}{n_N}\right)} \qquad \text{(eq. 6–21)}$$

As follows:

x–s part	P (ft)	A (ft²)	R (ft)	*n*
Lower channel	94	650	6.91	0.034
Upper channel	65	1875	28.8	0.064
Right overbank	89	376	4.22	0.087
Total channel	159	2,525	15.88	—
Total x–s	248	2,901	11.70	—

Using equation 6–21 the composite roughness is:

$$n = \frac{(248)(11.70)^{\frac{5}{3}}}{\frac{(94)(6.91)^{\frac{5}{3}}}{0.034} + \frac{(65)(28.8)^{\frac{5}{3}}}{0.064} + \frac{(89)(4.22)^{\frac{5}{3}}}{0.087}}$$

$$n = 0.042$$

This value can be compared to a value calculated with Manning's equation as follows.

$$n = \frac{1.486}{Q} AR^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$n_{chan} = \frac{1.486}{5770}(2525)(15.88)^{\frac{2}{3}}(.00016)^{\frac{1}{2}} = 0.052$$

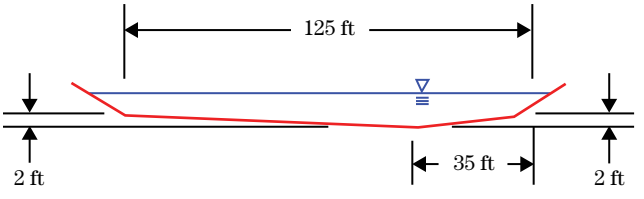
Discussion:
The difference in Manning's n initially appears to be cause for concern. However, it does illustrate three important points. First, this process is subjective, and two equally capable practitioners may arrive at different results. Second, Manning's equation is for uniform flow. Differences in measured and calculated n values should be attributed to the uncertainty in choosing appropriate values to account for various factors associated with roughness. Manning's equation by itself can provide an estimate, but it cannot precisely determine roughness when the flow is not uniform. Third, an uncertainty analysis is recommended for hydraulic analysis.

As documented in Barnes (1967), the USGS backwater calculations determined the channel n value to be 0.046 and the right overbank n value to be 0.097. In contrast to this example, Barnes calculated roughness using energy slope, rather than water surface slope and also included expansion and contraction losses.

Example problem: Manning's *n* value for a sand-bed channel

Problem: Determine the n value for a wide, sand channel with the following cross section (fig. 6–8). Assume a discharge of 4,100 cubic feet per second, a thalweg depth of 5 feet, 3:1 side slopes and a fairly straight, regular reach. Assume a slope of 0.0013 and a sandy bottom with a D_{50} of 0.3 millimeter.

Figure 6–8 Sand channel cross section



Solution: Roughness in sand channels is highly dependent on the channel bedforms, and bedforms are a function of stream power and the sand gradation. Arcement and Schneider (1990) show suggested n values for various D_{50} values with the footnote that they apply only for upper regime flows where grain roughness is predominant. For a D_{50} of 0.3 millimeter, this reference suggests a 0.017 n value. However, it is important to assess the regime of the flow. A figure from Simons and Richardson (1966) (also in Richardson, Simons, and Lagasse 2001 and Arcement and Schneider 1990) is shown as figure 6–9. Given stream power and median fall diameter, the flow regime may be estimated, as well as the expected bedform and roughness range.

Stream power may be calculated from where gamma is unit weight of water, Q is discharge, and S_f is the energy slope. Assuming the energy slope is nearly the same as the bed slope, then:

$$\Omega = \left(62.4\ \text{lb/ft}^3\right)\left(4100\ \text{ft}^3/\text{s}\right)\left(0.0013\right)$$
$$= 333\ \text{lb/s}$$
(per ft of channel length)

For figure 6–9, stream power per cross-sectional area is needed. The flow area for the given cross section is 554 ft^2, so the stream power is 0.60 pounds per second per square foot (per foot of channel length). Reading figure 6–9, with a D_{50} of 0.3 millimeter, the flow is in the upper regime, but close to the transition. This would support an n value of 0.017, particularly if bedforms are present.

Figure 6–9 Plot of flow regimes resulting from stream power vs. median fall diameter of sediment

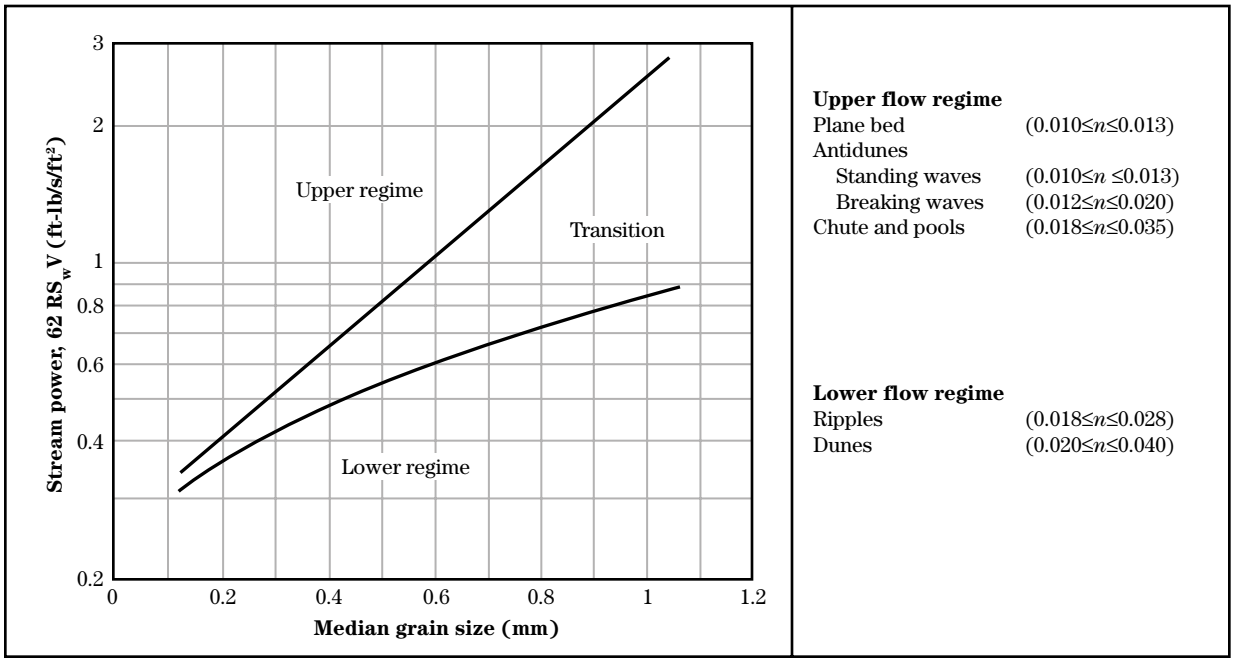


Figure 6–10 (Arcement and Schneider 1990) indicates the general bedforms for increasing stream power.

The anticipated bedform is a plane bed, and figure 6–9 suggests an n value between 0.010 and 0.013 for plane beds. The presence of breaking waves over antidunes would raise the roughness estimate to between 0.012 to 0.02. Finally, an estimate may be calculated with the Strickler formula (Chang 1988; Chow 1959) that relates n value to grain roughness. So, for a plane bed it should give a good estimate:

$$n = 0.0389\left(D_{50}\right)^{\frac{1}{6}} \quad \text{with } D_{50} \text{ in feet} \qquad \text{(eq. 6–22)}$$

or

$$n = 0.0474\left(D_{50}\right)^{\frac{1}{6}} \quad \text{with } D_{50} \text{ in meters} \qquad \text{(eq. 6–23)}$$

Since the D_{50} is 0.3 millimeter, the calculated n value is 0.012, which agrees with figure 6–9 results for plane beds. Arcement and Schneider (1990) show n = 0.012 for a D_{50} of 0.2 millimeter, and this calculation is close to the transition range. Considering all of the above, information supports a roughness selection between 0.013 to 0.017. If field observations support the plane bed assumption, a value from the low end of this range should be selected. If antidunes are present, a value from the high end of this range would be reasonable.

Example problem: Manning's *n* value for a gravel-bed channel

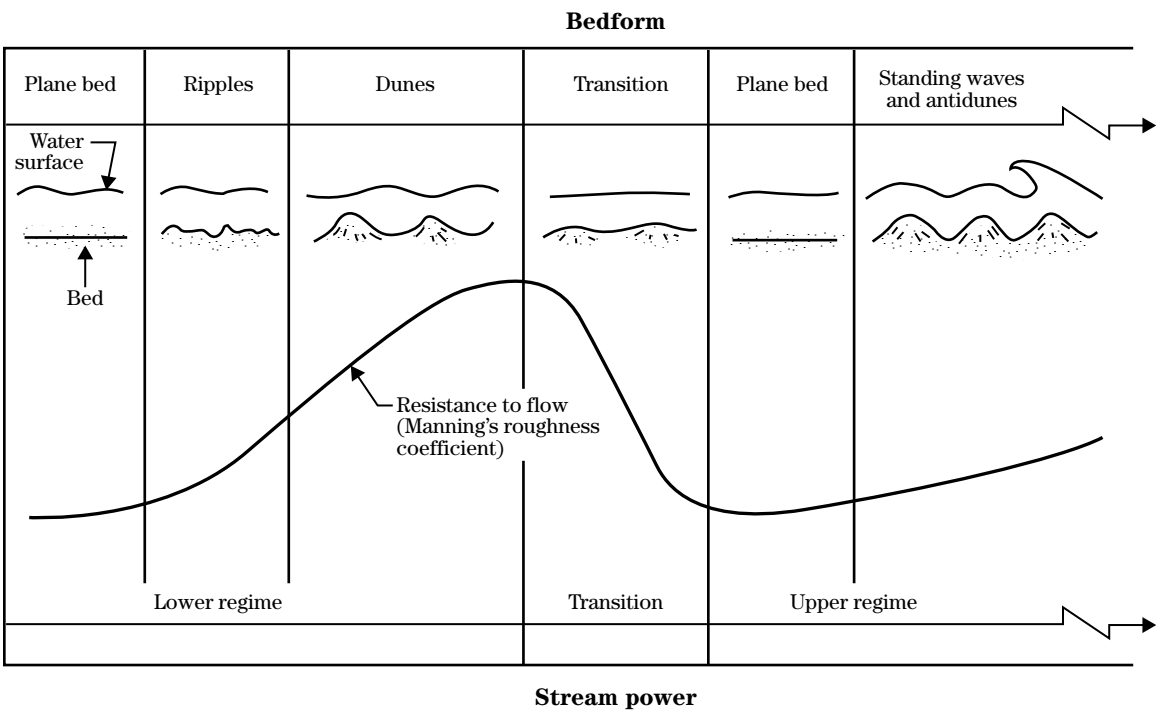
Problem: Determine the n value for a wide, gravel-bed channel with a D_{50} of 110 millimeters. Assume a fairly straight, regular reach. Assume minimal vegetation and bedform influence.

Solution: Since the grain roughness is predominant, the Strickler formula can be used.

$$n = 0.0474\left(D_{50}\right)^{\frac{1}{6}} \quad \text{for } D_{50} \text{ in meters}$$

This results in an estimated n value of 0.033. It should be noted that this estimate does not take into account many of the factors which influence roughness in natural channels. As a result, a estimate made with Strickler's equation is often only used as an initial, rough estimate or as a lower bound.

Figure 6–10 General bedforms for increasing stream power



(d) Friction factor

As with Manning's *n* value and the Chézy C, the friction factor, f, is a roughness coefficient in a velocity equation, namely, the Darcy-Weisbach equation. Originally developed for pipe flow, the equation adapted for flow in open channels is:

$$V = \left(\frac{8gRS}{f}\right)^{0.5} \text{ with f being dimensionless.} \qquad \text{(eq. 6–24)}$$

$$\text{Alternatively, } f = \left(\frac{8gRS}{V^2}\right) \qquad \text{(eq. 6–25)}$$

In 1963, the ASCE Task Committee on Friction Factors in Open Channels recommended the preferential use of the Darcy-Weisbach friction factor over Manning's *n* (Simons and Sentürk 1992). While Manning's equation remains the most used equation in practice, a comparison between the two is an illustrative exercise. The equation, applicable for steady uniform flow, is a balance of downstream gravitational force and upstream boundary resistance forces. The relationship between Manning's *n* and Chézy C is (Hey 1979, English units):

$$\left(\frac{8}{f}\right)^{0.5} = \frac{d^{\frac{1}{6}}}{ng^{0.5}} = \frac{C}{g^{0.5}} \qquad \text{(eq. 6–26)}$$

where:
d = hydraulic depth

To apply the velocity equation, the friction factor must be determined. As has often been discussed by researchers (Raudkivi 1990; Thorne, Hey, and Newson 2001), the vertical velocity profile can often be assumed to be logarithmic with distance from the bed. For sand and gravel channels, where the relative roughness (flow depth/bed-material size) exceeds 10, this relationship holds.

For use in gravel-bed streams, with width-to-depth ratios greater than about 15, Hey (1979) derived the following (see also Thorne, Hey, and Newson 2001):

$$\frac{1}{\sqrt{f}} = 2.03\log\frac{aR}{3.5D_{84}} \quad \text{(SI units)} \qquad \text{(eq. 6–27)}$$

or

$$\left(\frac{8}{f}\right)^{0.5} = 5.75\log\frac{aR}{3.5D_{84}} \quad \text{(English units)} \qquad \text{(eq. 6–28)}$$

where:
R = hydraulic radius
D_{84} = bed-material size for which 84 percent is smaller

The dimensionless a is given by (Thorne, Hey, and Newson 2001):

$$a = 11.1\left(\frac{R}{d_{max}}\right)^{-0.314} \qquad \text{(eq. 6–29)}$$

where:
d_{max} = maximum flow depth

The coefficient a varies from 11.1 to 13.46 and is a function of channel cross-sectional shape. For channels in which the width-to-depth ratio exceeds 2, the maximum flow depth is valid in the above equation. Otherwise, the value in the denominator should be the distance perpendicular from the bed surface to the point of maximum velocity. This formula for determining f may be used in gravel-bed riffle-pool streams in the riffle section, where flow is often assumed to be uniform. In general, the D_{84} is calculated based on a sample taken at the riffle section.

The Limerinos equation can also be used to determine the friction factor.

$$n = \frac{(0.0926)R^{\frac{1}{6}}}{\left(1.16 + 2.0\log\left(\frac{r}{D_{84}}\right)\right)} \qquad \text{(eq. 6–30)}$$

where:
R = hydraulic radius, in ft
D_{84} = particle diameter, in ft, that equals or exceeds that of 84 percent of the particles

This equation was developed from samples taken from 11 large United States rivers with bed materials ranging from small gravel to medium size boulders. This equation has been shown to work well on sand-bed streams with plane beds.

(e) Accounting for velocity distributions in water surface profiles

Actual velocities in a cross section are distributed from highest, generally in the center at a depth that is some small proportion beneath the surface, to much

lower values in overbanks and at flow boundaries (fig. 6–11). A velocity meter measures velocities related to the vertical flow area close to the instrument.

This elementary phenomenon is responsible for the fact that an average cross-sectional velocity cannot provide a precise measure of the kinetic energy of the flow; the alpha and beta coefficients therefore are needed as modifiers.

When the flow velocity in a cross section is not uniformly distributed, the kinetic energy of the flow, or velocity head, is generally greater than $V^2/2g$, where V is the average velocity. The true velocity head may be approximated by multiplying the velocity head by alpha (α), the energy coefficient. Chow (1959) stated that experiments generally place alpha between 1.03 and 1.36 for fairly straight prismatic channels. The nonuniformity of velocity distribution also influences momentum calculations (as momentum is a function of velocity).

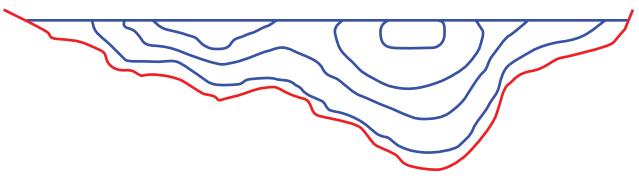
Beta (β) is the momentum coefficient that Chow indicates varies from 1.01 to 1.12 for fairly straight prismatic channels. Beta, also called the Boussinesq coefficient, is also described in Chow (1959). Both coefficients may be calculated by dividing the flow area into subareas of generally uniform velocity distribution.

$$\alpha \approx \frac{\sum v_i^3 A_i}{V^3 A_{total}} \qquad \text{(eq. 6–31)}$$

$$\beta \approx \frac{\sum v_i^2 A_i}{V^2 A_{total}} \qquad \text{(eq. 6–32)}$$

However, for natural channels, the calculation is better made using conveyance. HEC–RAS uses the following formulas:

Figure 6–11 Flow velocities for a typical cross section



$$\alpha \approx \frac{\sum\left(\frac{K_i^3}{A_i^2}\right)}{\frac{K_{total}^3}{A_{total}^2}} \qquad \text{(eq. 6–33)}$$

$$\beta \approx \frac{\sum\left(\frac{K_i^2}{A_i}\right)}{\frac{K_{total}^2}{A_{total}}} \qquad \text{(eq. 6–34)}$$

Every cross section is only a two-dimensional slice of a three-dimensional reality. Cross sections change along the stream profile, inevitably setting up transverse velocity vectors, and the flow is induced into a roughly spiral motion. This flow behavior leads to point bars, pools and riffles, meandering patterns, and flood plains. Further information on the velocity and shear in the design of streambank protection in bends is given in NEH654.14, Stabilization Techniques.

(f) Determining the water surface in curved channels

Water surface profiles as computed by HEC–RAS assume a level water surface in each cross section. This is not the case in a curved channel. However, the water surface calculated by HEC–RAS is valid along the centerline of the flow. Generally, HEC–RAS can account for the friction and eddy losses caused by a bend so that the water surface computed upstream would be correct. However, the super-elevated water surface in the bend itself must be calculated separately. The following formula is often used for estimating super-elevation in a water surface.

$$\Delta Z = \frac{bV^2}{gr_c} \qquad \text{(eq. 6–35)}$$

where:

V = average channel velocity (ft/s)
b = channel top width (ft)
g = gravitational acceleration (32.2 ft/s^2)
r_c = radius of curvature of the channel (ft)
ΔZ = super-elevation in ft from bank to bank, so the amount added to or subtracted from the centerline elevation would be half that. A factor of safety of 1.15 is generally applied.

In supercritical flow, curved channels are much more complicated due to wave patterns that propagate back and forth across the channel and downstream. With the disturbances reflecting from one side to the other, higher water surfaces can occur both on the inside and outside banks of a bend. Although a methodology for determining the super-elevation is developed by Chow (1959) for a regular curved channel with a constant width, it also approximates that for a natural channel.

Example problem: Super-elevation

Problem: A trapezoidal channel has a 30-foot bottom width, 1H:3V side slopes, and a radius of 100 feet. For a 500 cubic feet per second discharge, the depth is 4.12 feet, and the cross-sectional area is 174.5 square feet. Find the increase in water surface on the outside of the curve.

Solution: Calculate the velocity, from Q = VA:

$$V = \frac{Q}{A} = \frac{500}{174.5} = 2.87 \text{ ft/s}$$

top width is:

$$30 + (2 \times 3 \times 4.12) = 54.7 \text{ ft}$$

$$\Delta Z = \frac{bV^2}{gr_c} = \frac{(54.7)(2.87)^2}{(32.19)(100)} = 0.14 \text{ feet}$$

so, the increase in the flow depth on the outside of the curve is 0.07 feet, which is half of 0.14 feet.

(g) Transverse flow hydraulics and its geomorphologic effects

Frequently, the intent of channel design is to try to recreate or restore a natural condition, one that is geomorphologically sustainable. The hydraulic engineer needs to be aware of the mechanics of the flow and movable boundaries in channel curves. In a straight channel section, the task of determining boundary stress is easier than in curved reaches, as the direction of flow is more likely to be parallel to the banks. Shear force is dominant, and no significant additional force exists due to the momentum of flow impinging on the bank at some angle. In a curve, accounting for those angles of impinging flow is very important. The problem is three-dimensional, as previously mentioned, accounting for velocity distributions in water surface profiles, and flow in a curve sets up transverse velocity vectors and spiral motion. This phenomenon is completely natural and one of the driving mechanisms of geomorphology.

If a curving section of streambank is to be stabilized, some understanding of the nature of transverse (or secondary) flow is necessary. The task of streambank protection may be roughly divided into two major strategies: installation of measures that enable the bank to resist hydraulic forces at whatever angle they impinge or redirecting the flow so that the bank is no longer subject to damaging forces. Examples of the first would be planting vegetation on the banks or installing woody debris. The second strategy employs such measures as stream barbs, spur dikes, or longitudinal groins. Both of these strategies are covered extensively in NEH654.14 and related technical supplements in this handbook. However, particularly for curved channels, an examination of the hydraulic aspects upon which any streambank protection measure will succeed or fail is given here.

Even in straight channels, some flow spiraling can occur, and a moveable bed sets up transverse slopes that alternate direction along the bed profile. Figure 6–12 (Chang 1988) illustrates the behavior of spiral flow and the resulting transverse bed slopes.

In curved sections, the secondary current is not necessarily only one cell of circulation as shown in figure 6–13 (Chang 1988).

Chang (1988) provides the following equation for a hydraulically rough channel:

$$\tan\delta = 11\frac{d}{r} \qquad \text{(eq. 6–36)}$$

where:

δ = angle of the bottom current with channel centerline
d = depth at the location of interest in the section
r = radius of curvature to the location of d

The channel roughness is not considered to have a significant influence on the angle δ. Chang (1988) documents research that can enable the hydraulic engineer to calculate shear stress in the radial (or transverse) direction, the transverse bed slope a channel might be

Figure 6–12 Spiral flow characteristics for a typical reach

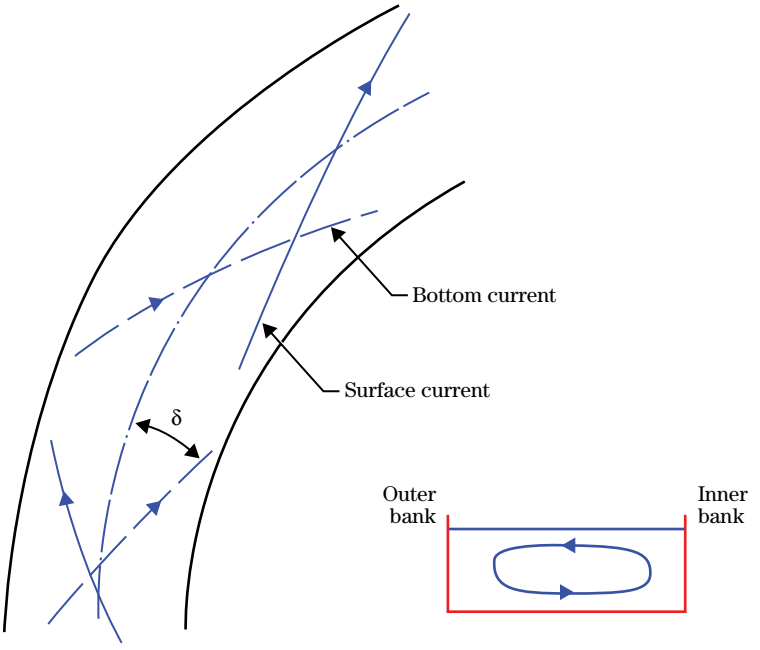
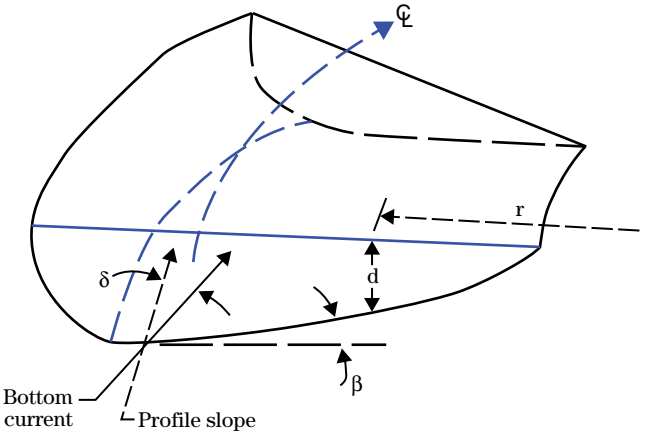


Figure 6–13 Flow characteristics for a typical reach



expected to acquire, and the sediment sorting expected along that transverse slope.

Chang (1988) provides the following two equations to calculate shear stress in the radial direction (both toward the inside of the curve, due to bottom current, and toward the outside due to surface current):

$$\tau_{0r} = -\rho d \frac{U^2}{r}\left[2\left(\frac{\sqrt{g}}{\kappa C}\right)^2 - 2\left(\frac{\sqrt{g}}{\kappa C}\right)^3\right] \qquad \text{(eq. 6–37)}$$

$$\tau_{0r} = \frac{1+m}{m^2+2m}\rho\frac{d}{r}U^2 \qquad \text{(eq. 6–38)}$$

$$m = \kappa\sqrt{\frac{8}{f}} \qquad \text{(eq. 6–39)}$$

where:
ρ = density of water
g = acceleration due to gravity
κ = the dimensionless von Kármán constant ($\kappa \approx 0.40$)
U = avg. cross-sectional velocity
C = Chézy resistance factor, defined below
d = depth at the location of interest
r = radius of curvature to that location
f = friction factor as defined below

The Chézy resistance factor is similar to Manning's *n* value in that it is an empirically derived coefficient serving as an index of boundary roughness. The following Ganguillet and Kutter formula (1869), as provided in Chow (1988), is a method of calculating Chézy C, given Kutter's n:

$$C = \frac{41.65 + \frac{0.00281}{S} + \frac{1.811}{n}}{1 + \left(41.65 + \frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}} \qquad \text{(eq. 6–40)}$$

where:
S = profile bed slope
R = hydraulic radius
n = Kutter's roughness

Chézy's C is related to Manning's *n* by the following equation in English units:

$$C = \frac{\left(1.486\ R^{\frac{1}{6}}\right)}{n} \qquad \text{(eq. 6–41)}$$

R = hydraulic radius (ft)

The Darcy-Weisbach friction factor, f, is described by Chow (1959) and for uniform or near uniform flow may be calculated using:

$$f = \frac{8gRS}{V^2} \qquad \text{(eq. 6–42)}$$

Both Chow (1959) and Chang (1988) describe the relationship of f to boundary Reynolds number. Chang provides three formulas, dependent on hydraulic smoothness, for channels in which form roughness is not a factor as follows.

$$f = \left(0.103 + 2\log R_{bed}\right)^{-.5} \quad \text{for} \quad \log\frac{R_{bed}k_s}{4R} < 0.5 \qquad \text{(eq. 6–43)}$$

(hydraulically smooth)
where:
R = hydraulic radius
R_{bed} = boundary Reynolds number
k_s = equivalent roughness or grain roughness, calculated from the following, one of several similar equations, Chang (1988):

$$k_s = 3D_{90} \qquad \text{(eq. 6–44)}$$

For the transition from hydraulically smooth to rough:

$$f = \left(\sum_{i=0}^{6} A_i\left(\log\frac{R_{bed}k_s}{4R}\right)^i + 2\log\frac{2R}{k_s}\right)^{-.5} \qquad \text{(eq. 6–45)}$$

for

$$0.5 \le \log\frac{R_{bed}k_s}{4R} \le 2.0$$

where the coefficients A_0 through A_6 are 1.3376, -4.3218, 19.454, -26.48, 16.509, -4.9407, and 0.57864, respectively.

For the hydraulically rough regime:

$$f = \left(1.74 + 2\log\frac{2R}{k_s}\right)^{-.5} \quad \text{for} \quad \log\frac{R_{bed}k_s}{4R} > 2.0 \qquad \text{(eq. 6–46)}$$

For gravel-bed rivers, Chang (1988) provides the following equation:

$$f = \left(0.248 + 2.36 \log \frac{d}{D_{50}}\right)^{-.5} \qquad \text{(eq. 6–47)}$$

where:
d = max depth of flow with units same as D_{50}

In figure 6–13, δ is the angle between the velocity vector of the bottom current and the centerline. Also of interest is the resultant angle of shear stress between the two components of shear, and longitudinal and radial. Chang (1988) gives that angle, δ , as:

$$\tan \delta' = \frac{2d}{\kappa^2 r}\left(1 - \frac{\sqrt{g}}{\kappa C}\right) \qquad \text{(eq. 6–48)}$$

where all variables have been previously defined.

Longitudinal shear stress at any point in the cross section is calculated with the following equation:

$$\tau_{0s} = \gamma d S_c \frac{r_c}{r} \qquad \text{(eq. 6–49)}$$

where the c subscript refers to the channel centerline.

The transverse bed slope (β) can be computed using:

$$\beta = \arctan\left(\tan \delta \tan \varphi\right) \qquad \text{(eq. 6–50)}$$

where:
δ = the angle shown in the above sketch
φ = the sediment angle of repose

This equation is valid when β is small compared to φ. This relationship is less accurate for channels with significant quantities of suspended sediment. Since φ is generally >30°, then β should be less than 10°. If φ >30°, then β becomes less valid as δ increases toward 20° or in tight curves.

Finally, Chang (1988) provides a formula for determining sediment sorting on the transverse slope:

$$D = \frac{3 \rho d S_c r_c}{2r\left(\rho_s - \rho\right)\tan \varphi} \qquad \text{(eq. 6–51)}$$

where:
D = median grain size
d = depth at that location
S_c = longitudinal profile slope along the centerline
r_c = radius of curvature to centerline
r = radius of curvature to location of d
ρ = densities of sediment and water

Example problem: Design radius

Problem: A roughly trapezoidal curved channel is being designed with a moveable boundary in dynamic equilibrium to carry a flow of 700 cubic feet per second. The channel profile slope is 0.0013, channel bottom width is 30 feet, with a transverse bed slope, β, of 10 percent, and 3H:1V side slopes. The bed material is rounded gravel, with a D_{50} of 0.30 inches, and *n* value of 0.035. Considering uniform flow and a maximum depth of 6 feet, calculate the design radius of curvature to the centerline, longitudinal and radial stress vectors at the centerline, and the resultant stress angle in the curve.

Solution:
Part 1—Design radius of curvature to the centerline

The angle of repose φ for 0.3-inch, rounded gravel is about 31 degrees. Assuming a constant transverse bed angle of 10 percent, tan β = 0.10, and the resulting angle of the bottom current would be:

$$\tan\beta = \tan \delta \tan \varphi \qquad \text{(eq. 6–52)}$$

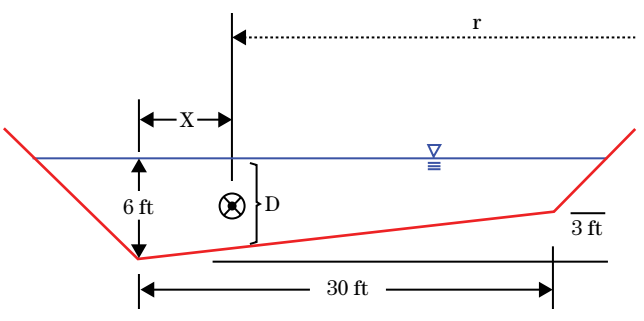
or

$$\delta = \arctan\left(\frac{\tan \beta}{\tan \varphi}\right) \qquad \text{(eq. 6–53)}$$

so, δ = 9.4 degrees

Consider the channel centerline to be horizontally located at the centroid of the flow cross section, as shown in figure 6–14.

Figure 6–14 Channel centerline at centroid of flow ⊗



To find X, the flow area left of the centroid must be equated to that on the right:

$$\frac{18\times 6}{2}+3X+\frac{30\times 3}{2}-\frac{(30-X)(3-0.1X)}{2}$$
$$=3(30-X)+\frac{9\times 3}{2}+\frac{(30-X)(3-0.1X)}{2}$$

Simplifying:

$$54+3X+45=90-3X+13.5+(30-X)(3-0.1X)$$

$$12X-0.1X^2=94.5$$

by trial and error, X = 8.5 feet.

The depth at the centerline is

$$6-(8.5)(0.10)=5.15\text{ ft}$$

given:

$\tan\delta=11\frac{d}{r}$, solving for radius of curvature, r = 342 ft

Part 2—Longitudinal and radial stress vectors

The longitudinal shear stress at the centerline is calculated with equation 6–49.

$$\tau_{0s}=\gamma dS_c\frac{r_c}{r}=62.4\times 5.15\times 0.0013=0.418\text{ lb/ft}^2$$

The total flow area is 202.5 square feet, wetted perimeter = 58.6 feet, so R = 3.46 feet. From Q = VA, the average velocity is 700/202.5 = 3.46 feet per second. The friction factor is:

$$f=\frac{8gRS}{V^2}=\frac{8\times 32.19\times 3.46\times 0.0013}{(3.46)^2}=0.097$$

The radial shear is calculated with equations 6–38 and 6–39.

$$\tau_{0r}=\frac{1+m}{m^2+2m}\rho\frac{d}{r}U^2$$

$$m=\kappa\sqrt{\frac{8}{f}}=0.4\sqrt{\frac{8}{0.097}}=3.36$$

$$\tau_{0r}=0.242\times 1.948\ \frac{\text{slugs}}{\text{ft}^3}\times\frac{\text{lb-s}^2/\text{ft}}{\text{slug}}\times\frac{5.15\text{ ft}}{342\text{ ft}}\times(3.46\text{ ft/s})^2$$
$$=0.085\text{ lb/ft}^2$$

Part 3—Resultant stress angle in the curve

The direction of the resultant stress vector between the longitudinal and radial components is calculated using equations 6–48 and 6–40.

$$\tan\delta'=\frac{2d}{\kappa^2 r}\left(1-\frac{\sqrt{g}}{\kappa C}\right)$$

where:

$$C=\frac{41.65+\frac{0.00281}{S}+\frac{1.811}{n}}{1+\left(41.65+\frac{0.00281}{S}\right)\frac{n}{\sqrt{R}}}$$

$$C=\frac{41.65+\frac{0.00281}{0.0013}+\frac{1.811}{0.035}}{1+\left(41.65+\frac{0.00281}{0.0013}\right)\frac{0.035}{\sqrt{3.46}}}=52.4$$

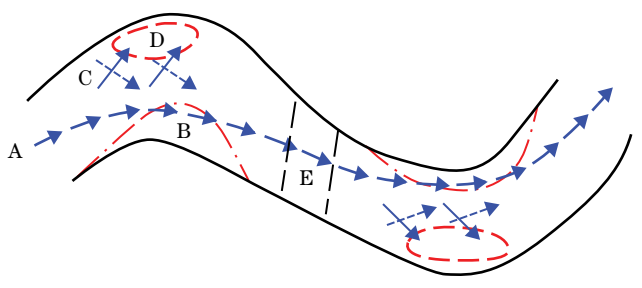
$$\tan\delta'=\frac{2\times 5.15}{(0.4)^2\times 342}\left(1-\frac{\sqrt{32.19}}{0.4\times 52.4}\right)=0.137$$

$$\delta'=8°$$

Hey (1979) addresses point bar development with a sketch similar to figure 6–15, showing how secondary currents, along with bed-load supply, impact the location of aggradation and degradation in a meander.

During bankfull flows, the strongest velocity vectors follow the course of the arrows starting at A in figure 6–15, cutting across the toe of point bars with the highest bed-load supply. At B, downstream of the bar apex, the shear stress and transport capacity drop, and aggradation occurs. Opposite the point bar at C, low bed load accompanies the incoming flow, and as surface

Figure 6–15 Point bar development



currents angle into the bank and undercurrents move away from the bank, a zone of downwelling results at point D. The low bed load gives the stream a scouring tendency. Toward the inflection point of the meander, flow with a low bed-load supply enters a contracted reach at E that is steeper and shallower, and regains its scouring capacity. Riffles form and, as the highest velocity vectors cut from one point bar toe to the toe of the next downstream bar, riffles are often skewed to the banks.

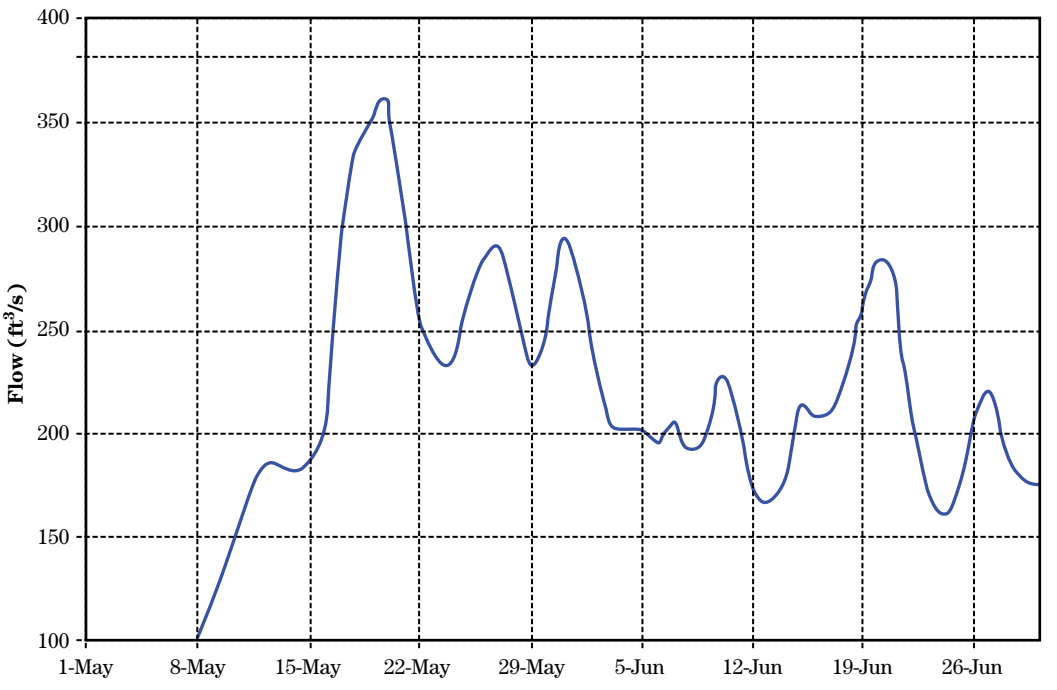
(h) Change in channel capacity

Natural channels will often incise in response to human impacts, such as watershed development, channel straightening, removal of vegetation, or overgrazing. The incision is a lowering of the channel bed, that in effect increases the channel size and capacity. Often, the overbank dries out due to a falling water table. This lowered water table can cause wetlands to shrink and adjacent productive lands to depend on irrigation. For projects in which overbank soil moisture is a concern, the duration of flow is often more important than the peak discharge. Inchannel flow can have a significant effect on overbank soil moisture if it is near bankfull for a sufficient duration.

Example problem: Change in overbank duration

Problem: A channel has, in the span of 10 years, incised by several feet and increased the bankfull flow area from 84 square feet to 107 square feet. The channel slope has increased from 0.0020 to 0.0025. The wetted perimeter increased from 29.4 feet to 42 feet. The vegetation has suffered to the extent that composite n value has decreased from 0.045 to 0.038. Approximate the change in duration of overbank flooding, given the season-long hydrograph in figure 6–16.

Figure 6–16 Seasonal hydrograph



Solution: Using a uniform flow assumption and Manning's equation, the original channel capacity was:

$$Q = \frac{1.486}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}}$$
$$= \frac{1.486}{0.045}(84)\left(\frac{84}{29.4}\right)^{\frac{2}{3}}(0.002)^{\frac{1}{2}}$$
$$= 250 \text{ ft}^3/\text{s}$$

With the changed hydraulic parameters:

$$Q = \frac{1.486}{0.038}(107)\left(\frac{107}{42}\right)^{\frac{2}{3}}(0.0025)^{\frac{1}{2}}$$
$$= 390 \text{ ft}^3/\text{s}$$

Looking at the hydrograph, then, the new channel condition fully contains the hydrograph, since the peak is less than 390 cubic feet per second: no days of overbank flooding occur. The previous channel capacity was 250 cubic feet per second, and overbank flow would have occurred four separate times for a total of about 16 days.

654.0610 Water surface profile calculations

The calculation of water surface profiles and associated hydraulic parameters is a common task of hydraulic engineers. In natural, gradually varied channels, velocity and depth change from cross section to cross section. However, the energy and mass are conserved. The energy and continuity equations can be used to step from a water surface elevation at one cross section to a water surface at another cross section that is a given distance upstream (subcritical) or downstream (supercritical). Programs, such as HEC–RAS, use the one dimensional energy equation, with energy losses due to friction evaluated with Manning's equation, to compute water surface profiles. Equation 6–9 becomes:

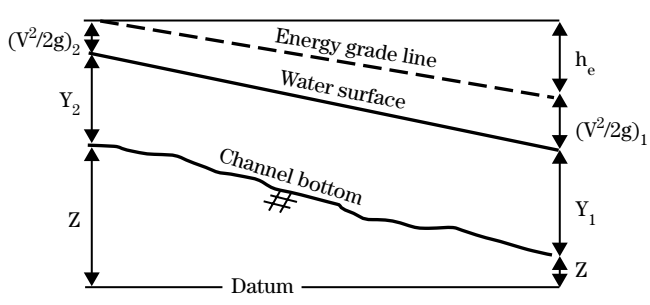
$$\left(\frac{V^2}{2g}\right)_2 + Y_2 + Z_2 = \left(\frac{V^2}{2g}\right)_1 + Y_1 + Z_1 + h_e \qquad \text{(eq. 6–54)}$$

This one dimensional energy equation can be restated as:

$$WS_2 = WS_1 + \frac{1}{2g}\left(\alpha_1 V_1^2 - \alpha_2 V_2^2\right) + h_e \qquad \text{(eq. 6–55)}$$

The water surface profile determination is accomplished with an iterative computational procedure called the standard step method. This is graphically illustrated in figure 6–17.

Figure 6–17 Standard step method



The energy loss includes friction losses (usually evaluated with Manning's equation) and losses associated with changes in cross-sectional areas and velocities. This is represented in equation 6–56:

$$h_e = L\overline{S_f} + C\left|\frac{\alpha V_2^2}{2g} - \frac{\alpha V_1^2}{2g}\right| \qquad \text{(eq. 6–56)}$$

Friction loss is evaluated as the product of the friction slope and the discharge weighted reach length. This is shown in equation 6–57:

$$L = \frac{L_{lob}\overline{Q_{lob}} + L_{ch}\overline{Q_{ch}} + L_{rob}\overline{Q_{rob}}}{\overline{Q_{lob}} + \overline{Q_{ch}} + \overline{Q_{rob}}} \qquad \text{(eq. 6–57)}$$

Example problem: Backwater from a log drop

Problem: Determine the maximum crest level of a log weir set all the way across the channel that would cause no backwater, and the crest level required to cause 1 foot of backwater just upstream of the weir (fig. 6–18). Assume a discharge of 491.5 cubic feet per second, depth of 4 feet, and uniform flow conditions without the weir.

Solution: To create no backwater, the log weir would have to pass the same discharge at the same water surface. The evaluation should be between the log crest (section 2) and a point (section 1) not very far upstream (fig. 6–19).

This can be evaluated using the energy approach with Bernoulli's equation. An assumption can be made that there is very little friction loss between the two points. The difference in the channel bottom elevation is also negligible over this short distance. So,

$$z_1 + y_1 + \alpha_1 \frac{V_1^2}{2g} = z_2 + y_2 + \alpha_2 \frac{V_2^2}{2g} + h_L$$

where:
h_L = head loss (assumed negligible) becomes:

$$y_1 + \frac{V_1^2}{2g} = y_2 + \frac{V_2^2}{2g} + D$$

where:
D = height of the log weir

If the flow is high enough, the log weir will be drowned out by the normal depth tail water and would not cause backwater. At lower discharges, the flow over the log will pass through critical depth (as shown in fig. 6–19).

At critical depth, the velocity head is equal to half the hydraulic depth:

$$\frac{V_{cr}^2}{2g} = \frac{d}{2} = \frac{A}{2T}$$

Figure 6–18 Problem determinations for determination of log weir

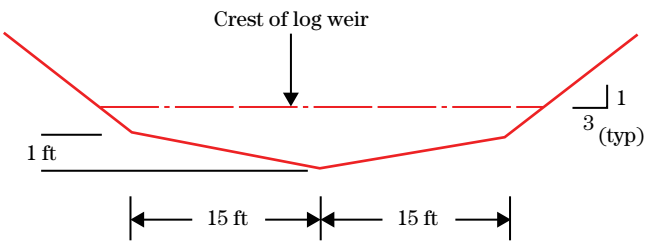
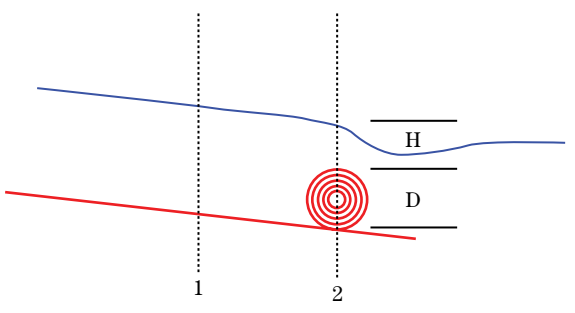


Figure 6–19 Profile for crest of log weir problem



Substituting back into Bernoulli's equation, since V_2 is V_{cr},

$$y_1 + \frac{V_1^2}{2g} = y_{cr} + \frac{A}{2T} + D$$

To determine whether the log causes backwater, compare the y_1 calculated to the flow depth without the log (4 ft). That is:

$$y_{cr} + \frac{A}{2T} + D - \frac{V_1^2}{2g} \qquad \text{less than 4}$$

where:
V_1 = velocity upstream of the weir

Using the critical depth formula (where d is hydraulic depth and T is top width) along with the continuity equation, Q = VA, the following can be derived:

$$\frac{V_{cr}^2}{2g} = \frac{d}{2} = \frac{A_{cr}}{2T}$$

$$A_{cr} = \frac{Q}{V_{cr}} \quad \text{and} \quad V_{cr}^2 = \frac{A_{cr}2g}{2T} = \frac{A_{cr}g}{T}$$

$$A_{cr}^2 = \frac{Q^2}{V_{cr}^2} \quad \text{so} \quad A_{cr}^2 = \frac{Q^2T}{A_{cr}g} \quad \text{and} \quad A_{cr} = \sqrt[3]{\frac{Q^2T}{g}}$$

To find the maximum log crest before backwater is created, a log crest must be chosen and checked with a trial and error approach. For this example, suppose D = 1. Choose a depth and calculate the flow area by the dimensions of the cross section. Then compare with the A_{cr}. When the two flow areas are the same, this is the critical depth for that Q (given as 491.5 ft^3/s).

Trial #	y_{cr}	T	A	A_{cr}
1	1.0	36.0	33.0	64.65
2	2.0	42.0	72.0	68.06
3	1.9	41.4	67.8	67.70

However, the velocity head must still be calculated to assure that there is no backwater. Note that the velocity head is negligible as long as the velocity is not too large. For example, a velocity of 5 feet per second results in a velocity head of 0.39 feet.

$$y_{cr} + \frac{A}{2T} + D - \frac{V_1^2}{2g} \qquad \text{less than 4}$$

If this velocity head term is neglected, then given y_{cr} = 1.9, T = 41.4, A = 67.8, the above formula solves as:

$$1.9 + \frac{67.8}{2 \times 41.4} + 1 = 3.72 \text{ ft}$$

Since this solution is less than the clear channel depth of 4, it may be possible to raise the weir.

(a) Steady versus unsteady flow

Many hydraulic parameters of interest in typical designs and assessment can be calculated by assuming a normal depth. Normal depth calculations are often based on a solution to Manning's equation. This approach is relatively simple, but only applicable in uniform flow conditions where the gravitational forces are exactly offset by the resistance forces. Manning's equation is an infinite slope model that assumes mean depth, velocity, and area are the same from cross section to cross section. It can only occur in long, straight, prismatic channels where the terminal velocity of the flow is achieved. This assumption cannot account for backwater conditions nor variable channel shape, roughness, and slope. Natural channels approach, but rarely achieve uniform, normal depth. Designs and assessments that depend on calculations based on normal depth must consider the affects of possible errors.

Even though flows in a stream are readily observable at any time, they are unsteady at every spatial and temporal scale. Typically, unsteady modeling results in variations in flow rate, velocity, and depth in space and time throughout the modeled reach. In most unsteady flow models, the discharges can vary within a model, and the boundary conditions are in terms of flow and stage with time. Unsteady flow calculations are often used to analyze a dam breach, inchannel storage, variable boundary conditions, rapidly rising hydrographs on flat slopes, irrigation withdrawals, tributary flow interaction, and locations where duration of flooding is an issue.

Unsteady flow models can be contrasted to steady flow models with no time component in the calculations. Steady flow models are typically much simpler to calibrate and execute than unsteady flow models. For most steady flow models, the depth and velocity

may change from section to section, but only one flow is allowed per section per model run. Since the flow is constant with respect to time, only one discharge is calculated for each section in a given steady flow model run. In addition, boundary conditions are held constant. These assumptions are often suitable for many analyses where the reach is short or the primary interest is an assessment of the peak hydraulic parameters for a given discharge.

In alluvial channels, the interaction of sediment with the flow can also have a profound affect since the amount and type of sediment load affects the energy balance of the flow. Equations of sediment motion (sediment continuity and sediment transport) are covered in NEH654.13.

(b) Backwater computational models

Computer programs are used to calculate water surface profiles, lateral velocity distributions, flow regimes, and scour potential. For projects that are likely to involve revisions to Federal Emergency Management Agency's (FEMA) Flood Insurance Rate Maps, selection of the hydraulic model should be coordinated carefully with FEMA. Following are some standard hydraulic models.

HEC–RAS
HEC–RAS (USACE 2001b) is the recommended computer program for performing hydraulic calculations for steady and unsteady, gradually varied (over distance), one-dimensional, open channel flow. HEC–RAS includes a culvert module that is consistent with HDS–5 and HY–8. The bridge hydraulics algorithms now include the WSPRO models. HEC–RAS applies conservation of momentum, as well as energy and mass, in its hydraulic analysis. HEC–RAS includes all the features inherent to HEC–2 and WSPRO, plus several friction slope methods, mixed flow regime support, automatic n value calibration, ice cover, quasi 2–D velocity distribution, and super-elevation around bends.

HEC–2
HEC–2 (USACE 1990b) performs hydraulic calculations for steady, gradually varied (over distance), one-dimensional, open channel flow. One of HEC–2's technical limitations is that the normal bridge routines and standard-step backwater computations use energy conservation only. Conservation of momentum is used only in the special bridge routines when bridge piers are involved.

WSPRO
The WSPRO computer program was developed by the USGS and is comparable to HEC–2, except for the fact that WSPRO had special subroutines for analysis of water surface profiles at bridge locations. All of these WSPRO subroutines have been incorporated into HEC–RAS. The current version of WSPRO is no longer being supported by USGS.

HY–22
HY–22 is a small tool kit of relatively simple computer programs for performing the hydraulic analyses described in the Urban Drainage Design Manual, Hydraulic Engineering Circular No. 22, U.S. Department of Transportation Federal Highway Administration (FHWA) (1996). HY–22 includes pavement drainage, open channel hydraulics, critical depth computation, computation of storage volume, and simple reservoir routing.

654.0611 Weir flow

Flow over a broad-crested weir is an application that can be analyzed with momentum principles. The momentum principle has certain advantages in application to problems involving high internal energy changes (Chow 1959). The pressure force due to the weight of water and the obstruction of the weir is important. The gravitational force vector in the direction of flow may be neglected for a mild channel slope and small distance between the uncontracted upstream section and the cross section at the weir. The friction forces on the wetted boundary in the short distance between the two sections may be neglected, as well.

Weir flow is calculated using:

$$Q = CLH^{\frac{3}{2}} \qquad \text{(eq. 6–58)}$$

where:

L = weir length (ft)

C = weir discharge coefficient (usually from 3.05 to 2.67)

H = approach head (ft)

The actual value of C depends on factors such as the roundedness of the upstream corner of the weir and the width and slope of the weir crest. Brater and King (1976) give C = 3.087 as a maximum value for broad-crested weirs with a vertical upstream face under any conditions, given that the upstream corner is so rounded as to prevent flow contraction and the slope of the crest is at least as great as the head loss on the weir due to friction. Under these conditions, flow over the weir occurs at critical depth. Inclining one or both faces of the broad-crested weir can also increase the C value, and Brater and King document experiments that obtain values of C as high as 3.8.

The flow velocity vectors for this equation are considered to be perpendicular to the crest; that is, the flow momentum is straight into the weir. If the weir is a lateral one or the main channel flow is parallel to the crest and the weir draws flow off to the side, the weir capacity would be less.

The major use of sharp-crested weirs is for flow measurement. Many different crest cross-sectional shapes exist, such as a V-notch, but the weir width is always thin and is perpendicular to the flow. The same discharge equation used for broad-crested weirs may be applied to horizontal, sharp-crested weirs, but the discharge coefficient, C, is highly dependent on the nappe conditions. The nappe is the sheet of water flowing or jetting over the weir. A fully aerated nappe has an air pocket at atmospheric pressure just downstream of the weir and below the sheet of flowing water. A weir with a fully aerated nappe has a higher discharge coefficient than one in which the nappe is partially (air pressure less than atmospheric) or fully submerged (no air pocket).

654.0612 Hydraulic jumps

Determining the strength and location of hydraulic jumps is important for designing energy dissipation structures and assessing the effectiveness of stream barbs or step-pool structures. The following equation is used to estimate energy dissipation at a hydraulic jump:

$$\Delta E = E_1 - E_2 = \frac{(y_2 - y_1)^2}{4y_1y_2} \qquad \text{(eq. 6–59)}$$

Energy is expressed in units of length (a head loss). The height of a jump for a channel of small slope can be estimated from:

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right) \qquad \text{(eq. 6–60)}$$

where:

y_1 = upstream depth
y_2 = downstream flow depth
F_1 = Froude number of the upstream flow. This equation is derived from the specific force formulation for a rectangular channel.

where:

$$\frac{2Q^2}{gA_1} + A_1y_1 = \frac{2Q^2}{gA_2} + A_2y_2 \qquad \text{(eq. 6–61)}$$

And the Froude number is:

$$F_1 = \frac{V_1}{\sqrt{gy_1}}$$

Substituting VA for Q and bd for A, where b is the channel bottom width, as well as making use of the definition of Froude number, this equation can be simplified to:

$$\frac{y_2^2}{y_1^2} - \left(2F_1^2 + 1\right) + \frac{2F_1^2y_1}{y_2} = 0 \qquad \text{(eq. 6–62)}$$

Knowing the depth of the approaching flow and its Froude number, the flow depth downstream of the jump can be calculated. Froude numbers can also be used to specify different types of jumps as shown in table 6–1 (Chow 1959).

The length of a well-defined hydraulic jump is the distance from the upstream face of the jump to the point on the surface just downstream of the roller. Chow indicates that it cannot be easily determined theoretically and is best estimated empirically. The U.S. Department of Interior Bureau of Reclamation performed numerous experiments and provides figure 6–20 for determining jump length based on upstream Froude number and upstream flow depth (Peterka 1984). L is jump length, y_1 is upstream depth, and the Froude number is that of the flow coming into the jump.

The location along the channel profile of the upstream beginning of the hydraulic jump can be generally determined from

$$\frac{y_2}{y_1} = \frac{1}{2}\left(\sqrt{1+8F_1^2} - 1\right) \qquad \text{(eq. 6–63)}$$

However, the jump length has a bearing on this estimate. For example, the location of a hydraulic jump formed by a broad-crested weir in the channel can be used to illustrate this situation (fig. 6–21). Downstream tailwater affects the location of the jump, moving it farther upstream and closer to the weir, as the tailwater is raised. A lower tailwater elevation produces a jump farther downstream. Increasing the height of the weir moves the jump upstream, whereas decreasing it moves the jump downstream.

Table 6–1 Froude numbers for types of hydraulic jumps

Froude	Jump type
<1	No jump, subcritical flow
1.0	No jump, critical flow
1.0–1.7	Undular jump: unsteady water surface
1.7–2.5	Weak jump: small rollers develop on surface
2.5–4.5	Oscillating jump: vertical flow jet produces surface waves that may travel long distances
4.5–9.0	Steady jump: best energy dissipation performance
>9	Strong jump: very high energy dissipation, but with surface waves sent downstream

Figure 6–20 Determination of jump length based on upstream Froude number

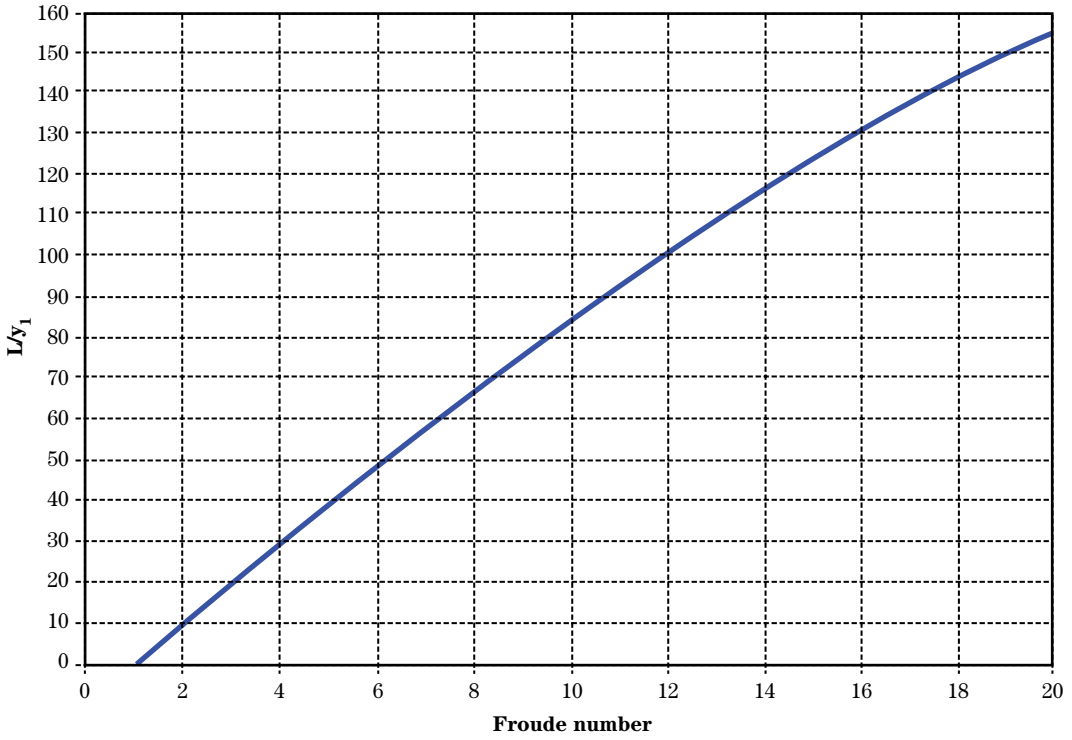
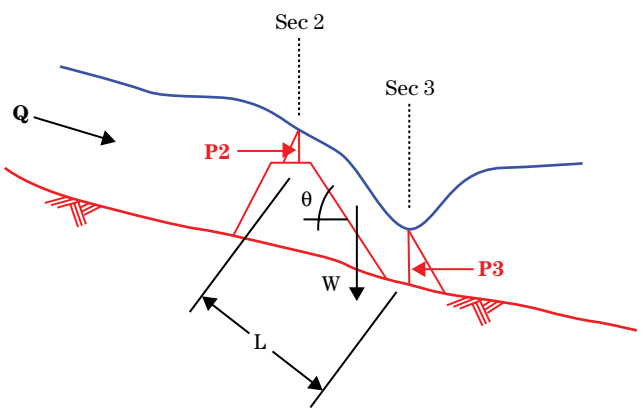


Figure 6–21 Parameters involved with modeling a hydraulic jump



However, the weir will not cause an hydraulic jump if it is drowned out by downstream tailwater. The downstream depth must be less than critical depth over the weir plus the weir height, or, using the definition of critical depth:

$$y_3 < \frac{(2y_1 + h)}{3} \qquad \text{(eq. 6–64)}$$

where:

y_1 = depth upstream of the weir
h = weir height
y_3 = tailwater depth downstream

654.0613 Channel routing

Channel routing is an important component of hydrologic modeling and assessments. Designers need to be able to estimate not only flow volumes, but also hydraulic parameters for many projects. Efforts to mathematically model and predict channel routing started with Jean Claude Saint-Venant in 1871. However, it is only with the advent of high speed computers that many of the techniques are readily available to most designers. The practitioner should keep in mind that even the most advanced computer models simplify natural system processes. It is, therefore, important for the modeler to understand the computational procedures used in the model being applied.

(a) Movement of a floodwave

Channel routing is the calculation of the hydraulic parameters of a floodwave as it moves through a channel. The overall movement is typically described with the concepts of celerity and attenuation. Floodwave celerity is the speed at which the floodwave moves down the channel and is primarily a function of the channel slope. The attenuation of a floodwave is the subsidence or flattening of the wave as it moves down the channel. Floodwave attenuation is directly related to the amount of inchannel or riparian storage available.

(b) Hydraulic and hydrologic routing

The movement of a floodwave is governed by the laws of fluid mechanics. The two equations for clear water flow are the conservation of mass, or the continuity equation, and the momentum equation. These two equations are referred to as the Saint-Venant equations. Traditional hydraulic routing involves a numerical solution to these equations as partial differential equations. Therefore, hydraulic routing is viewed as being more physically based than hydrologic routing.

Traditional hydrologic routing typically uses an algebraic solution to the continuity equation and a relationship between changes in storage in the reach and discharge at the outlet. Hydrologic routing is often based on analogies of stream channels and basins as a

set of storage reservoirs with appropriate properties. In fact, hydrologic routing equations are often referred to as storage routing equations. As a result, hydrologic modeling is inherently empirically based. Typical hydrologic routing equations include the Muskingum routing and the Reservoir (Puls) routing procedures.

(c) Saint-Venant equations

Most channel routing performed by computer modeling is based on some simplification of the Saint-Venant equations. These equations provide a very simple model of very complex processes. These equations are:

Continuity

$$A\frac{\partial V}{\partial x} + VB\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = q \qquad \text{(eq. 6–65)}$$

Momentum

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial y}{\partial t} \qquad \text{(eq. 6–66)}$$

where:

A = cross-sectional flow area
V = average velocity of water
x = distance along channel
B = water surface width
y = depth of water
t = time
q = lateral inflow per unit length of channel
S_f = friction slope
S_o = channel bed slope
g = gravitational acceleration

The solutions to the momentum and continuity equations concurrently define the propagation of a floodwave with respect to distance along the channel and time. Assumptions for these equations include:

- The momentum and continuity equations are shown for one-dimensional flow in the downstream direction. The natural variation in velocity with respect to depth is ignored. In addition, these equations do not directly address lateral or vertical stream flows that would require a more complex equation.
- Flow is gradually varied so that hydrostatic pressure prevails, and vertical accelerations can be ignored.
- The effects of boundary friction and turbulence can be treated with resistance laws, as they are in steady flow.
- Fluid is incompressible and has a constant density.

(d) Simplifications to the momentum equation

Depending on the relative importance of the various terms of the Momentum equation, it can be simplified for different applications as follow:

Steady uniform flow (kinematic wave approximation)	$S_f = S_o$
Steady nonuniform flow (diffusive wave approximation)	$S_f = S_o - \frac{\partial y}{\partial x}$
Steady nonuniform flow (Quasi-steady state dynamic wave approximation)	$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x}$
Steady nonuniforn flow (full dynamic wave approximation)	$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial y}{\partial t}$

Since simplification means that some aspect is being ignored, it is important for a modeler to understand the basis of the model being applied to answer a hydraulic or hydrologic question. Further discussion on application and limitations of some of routing approaches that are used in many computer programs follows.

- *Kinematic wave approximation*—The kinematic wave approximation assumes that the gravitational and frictional forces are in balance. The kinematic wave approximation works best when applied to steep (0.0019, 10 ft/mi or greater), well-defined channels, where the floodwave is gradually varied. Changes in depth and velocity with respect to time and distance are small in magnitude when compared to the bed slope of the channel. The approach is often applied in urban areas because the routing reaches are generally short and well defined (circular pipes, concrete lined channels). However, the equations do not allow for hydrograph diffusion, but only simple translation of the hydrograph in time.

The application of the kinematic wave equation is limited to flow conditions that do not demonstrate appreciable hydrograph attenuation. This may be an issue in wide channels, since attenuation increases with valley storage. The kinematic wave equations cannot handle backwater effects, since with a kinematic model flow disturbances can only propagate in the downstream direction.

- *Modified Puls reservoir routing*—This approach accounts for the difference of inflow as storage over some defined time period. This method is appropriate if lateral storage is the primary physical mechanism that affects the flood routing. This method disregards the equation of motion by focusing on continuity. It is closely related to level pool reservoir routing.
- *Muskingum river routing*—The Muskingum river routing method is based on two equations. The first is the continuity equation, and the second is a relationship of storage, inflow, and outflow of the reach. This method is based on a weighted function of the difference of inflow as storage over some defined time period. Typically, the coefficients of the Muskingum method are not directly related to physical channel properties and can only be determined from stream gage data.
- *Diffusive wave approximation*—The diffusion wave model is a significant improvement over the kinematic wave model because of the inclusion of the pressure differential term in the momentum equation. This term allows the diffusion model to describe the attenuation (diffusion effect) of the floodwave. It also allows the specification of a boundary condition at the downstream extremity of the routing reach to account for backwater effects. It also allows the specification of a boundary condition at the downstream extremity of the routing reach to account for backwater effects. Since it does not use the inertial terms (last two terms) from the full momentum equation, it is limited to slowly to moderately rising floodwaves in flat channels (Fread 1982). However, most natural floodwaves can be described with the diffusion form of the equations.
- *Muskingum-Cunge*—The theoretical development of the Muskingum-Cunge routing equation is based on the simplification of the convective diffusion equation. In the Muskingum-Cunge formulation, the amount of diffusion is controlled by forcing the numerical diffusion to match the physical diffusion represented by the convective diffusion equation. This approach accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The method includes the continuity equation and a relationship of storage, inflow, and outflow of the reach. The solution is independent of the user-specified computation interval. The coefficients of the Muskingum-Cunge method are based on data such as cross section and estimated Manning's n and are more physically based than the Muskingum method. Therefore, the Muskingum-Cunge method can be applied to ungaged streams. However, it cannot account for backwater effects, and the method begins to diverge from the full unsteady flow solution when very rapidly rising hydrographs are routed through flat channel sections.
- *Quasi-steady dynamic wave approximation*—The third simplification of the full dynamic wave equations is the quasi-steady dynamic wave approximation. In the case of flood routing, the last two terms in the momentum equation are often opposite in sign and tend to counteract each other. By including the convective acceleration term and not the local acceleration term, an error is introduced. This error is of greater magnitude than the error that results when both terms are excluded, as in the diffusion wave model. This approach is not often used in flood routing.
- *Dynamic wave equations*—The dynamic wave equations can be applied to a wide range of one-dimensional flow problems, such as dam break flood wave routing, tidal fluctuations, canal distribution, and forecasting water surface elevations and velocities in a river system during a flood. Solution of the full equations is normally accomplished with an explicit or implicit finite difference technique. The equations are solved for incremental times (dt) and incremental distances (dx) along the waterway.

654.0614 Hydraulics input into the stream design process

(a) Determining project scope and level of analysis

Hydraulic engineering contributions to stream design can be viewed as a three-dimensional process. The most important two dimensions are the type of project and the stage of the project.

The third dimension is the constraint of time and/or cost that is not strictly engineering related. The role of the hydraulic engineer in this third dimension is to apply the standards of professional engineering licensure. If time or cost prevents an analysis from meeting professional engineering standards, the engineer must inform project managers and act accordingly.

The level of detail required of a hydraulic analysis falls into one of three categories: rough estimation, standard engineering, and atypical complexity. Generally, the reconnaissance stage of a project requires a rough estimation level of detail, although many standard engineering procedures are not time consuming nor difficult to apply, so that often a reconnaissance stage can be supported with a greater level of detail. For the remaining project stages, standard engineering procedures are minimally required. However, depending on the project particulars, atypical complexity may be necessary.

Each project type, as identified in table 6–2, will have pertinent hydraulic parameters, computations, and an applicable level of detail. The scope of the hydraulic analysis is tied to the project, and each project type generally corresponds with a hydrology type as shown in table 6–3.

When providing hydraulic computations, the designer should also estimate uncertainties, be able to specify their source, and provide confidence limits. Engineering in a stream corridor requires field work. The greater the quantity or precision of results needed, the greater the amount of field data required. Time and human labor cost may be expected to rise accordingly.

Table 6–2 Project dimensions by type and stage of project

	Dimension 1
	Type of project
1	Flood prevention or flood level determination
2	Bed and bank scour prevention, streambank protection
3	New or relocated channel design
4	Design of structures (bridge, culvert, levee, drop, weir)
5	Habitat or vegetation enhancement
6	Flood plain reconnection

	Dimension 2
	Dimension of project
1	Reconnaissance
2	Planning
3	Design
4	Monitoring

Table 6–3 Scope of hydraulic analyses by project type

	Flow level	Major concerns	Project types
1	Low flow	Duration	5
2	Bankfull flow	Duration and frequency	2, 3, 6
3	Overbank flows	Frequency	1
4	Specific flow levels	Frequency	4

(b) Accounting for uncertainty and risk

The hydraulic engineer must always keep in mind the level of certainty inherent in data measurements, computational methods, and information provided by others. For example, the frequency of flows developed by a hydrologist is, at best, a statistical derivation with confidence limits. The hydraulic engineer can inspect the steepness of the frequency curve, as well as the confidence limits to determine the range of flows that should be associated with a given recurrence interval. If the hydrologist had no gage data from which to develop frequency information, the hydrology would probably be considered even less reliable.

As described in this chapter, numerous methods in hydraulic engineering were developed from empirical studies. The designer should know what situations are and are not applicable to a given methodology. Whenever simplified methods are employed, the designer should be aware of the sacrifice in confidence of results.

One typical response in the attempt to minimize the risk due to uncertainty is to use factors of safety and be conservative. However, it is critical that the designer apply factors of safety to the correct calculations and be conservative in the correct aspect of the analysis. To be conservative from a flood control perspective is to design a larger than necessary channel. However, if the goal of the project is to reconnect the flood plain, that designer's conservatism may lead to design failure.

The designer should keep track of each computational attempt to account for uncertainty. In each case, an adjustment should be justified by a description of the source of the uncertainty and reasoning regarding the magnitude of the adjustment. In many cases, a conventional factor of safety will have been established by the field of hydraulic engineering. Standard freeboard heights for channel design are also conventional.

Finally, the hydraulic engineer may wish to more fully document the impact of uncertainties by modeling what-if scenarios, considering extreme values of one or more parameters.

654.0615 Conclusion

This chapter provided an overview of the hydraulic engineering concepts involved in stream design. A number of typical hydraulic computations were provided as examples. This discussion can help all disciplines better understand the role of hydraulics in the stream design process.

United States
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**Natural
Resources
Conservation
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**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 7

Basic Principles of Channel Design

Issued August 2007

Cover photo: Where modification of the stream or channel is needed to achieve restoration goals and objectives, alteration of dimensions, pattern, and profile may be needed, resulting in designs ranging from minimal changes to large-scale rehabilitations.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 7 Basic Principles of Channel Design

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Chapter 7 Basic Principles of Channel Design

654.0700 Purpose

Channel design may involve the stabilization or re-alignment of an existing stream, or it may involve the creation of an entirely new channel. There are a wide variety of sources and techniques for designing stable channels that are available to the designer. These techniques may focus on a variety of open channel design work ranging from natural stream restoration to a strictly structural project. However, these techniques need to be applied to the appropriate conditions and stream types. The purpose of this chapter is to provide a framework for the designer to assess the use and application of several of the analysis and design techniques presented in subsequent chapters. This chapter provides some background which should be useful in the evaluation of these techniques to address specific goals, constraints and conditions. To provide a context for the different design techniques, a clear description of threshold and alluvial channels is presented in this chapter. In addition, a general description of channel design variables and approaches is presented. These broad, and occasionally overlapping, categories of stream types and design approaches can be used to evaluate the appropriateness of the design techniques for a specific objective and site.

654.0701 Overview of channel design

A stable channel is often defined as a channel where the planform, cross section, and longitudinal profile are sustainable over time. While channel migration may not always be acceptable due to project or site constraints, it is important to note that a natural channel can migrate and still be considered stable, in that its overall shape and cross-sectional area do not change appreciably. Design methodologies and approaches may be used to estimate the conditions that may result in such movements. Design features are also often employed to reduce the frequency and magnitude of these changes.

Another common goal for a channel restoration design is that long-term aggradation and/or degradation should be small enough to allow for economical channel maintenance. Ideally, a channel should be self-sustaining and not require any maintenance. Many design methodologies can be used to design a channel which is in balance with the incoming sediment load. However, it is also important for the designer to recognize that manmade, as well as natural channels may aggrade or degrade over time or in response to specific storm events. Sediment impact assessments can be used to quantify what storm events may result in a sediment disequilibrium and to quantify the expected aggradation, so that appropriate maintenance can be budgeted. Design features can also be employed to counteract a tendency for bed degradation.

A variety of applicable open channel analysis and design techniques are available to the designer. The approaches used in open channel design range from those that apply to a natural stream restoration, to those that are more applicable to a strictly structural project. The specifics and details regarding the use and application of several analysis and design techniques are presented in subsequent chapters. This chapter provides a framework in which to evaluate these techniques. While techniques may have the same general objective, the specifics of their applicability should be understood before one approach is chosen over another. Where there is uncertainty regarding the appropriate technique to use, it is recommended that the designer consider several applicable techniques and look for agreement on critical design elements.

Each technique presented and described in this handbook has advantages and disadvantages. One approach may require more certainty in specific background information than another. In other situations, one approach may result in a type of channel which may not satisfy a given ecological goal, while another may result in a more expensive, but potentially more ecologically beneficial project. In addition, different analysis and design techniques are more appropriate for use on specific stream types and systems than on others. For example, some of the techniques are appropriate only for fixed-bed systems, while others are appropriate for mobile-bed systems. While all of the presented techniques have been successfully used, there are many examples where they have been misapplied and have resulted in projects which performed less than ideally.

Many papers and descriptions compare and contrast the different design methods and approaches that are presented in this chapter. The purpose of this document is not to evaluate each of the techniques as being more suitable than others, but to present the user with sufficient information to understand the application of the individual techniques. It is left to the user to review and assess the applicability of each of the techniques to the project site.

654.0702 Channel types

The nature of the interaction of the flows and sediments with the channel boundary should be used in the selection of the appropriate design approach. Channels can be divided into two general categories based on the sediment load and the stability of the channel boundary during normal flow. These two categories are threshold and alluvial channels. The general design approaches for each are defined and contrasted in this chapter. In subsequent chapters, specific design techniques are presented and described. Since there is not always a sharp demarcation between these two very broad categories, transition channels are also described.

(a) Threshold channels

A threshold channel is defined as a channel in which channel boundary material has no significant movement during the design flow. The term threshold is used because the channel geometry is designed such that applied forces from the flow are below the threshold for movement of the boundary material.

A threshold type of channel or stream includes cases where the bed is composed of very coarse material or erosion resistant bedrock. Streams where the boundary materials are remnants of processes no longer active in the stream system may be threshold streams. Examples are streambeds formed by high runoff during the recession of glaciers or dam breaks and streams armored due to reduction in the upstream sediment supply and degradation. Photographs of examples of threshold channels are provided in figures 7–1 through 7–3.

Fine sediment may pass through threshold streams as throughput or wash load. Generally, wash load should not be considered part of the bed-material or sediment load for stability design purposes even if there are temporary deposits on the streambed at low flow. However, throughput or wash load may be an environmental issue.

Threshold channels do not have the ability to quickly adjust their geometry, as do alluvial channels, because the material forming the channel boundary is not erodible within the normal range of flows, and there is no significant exchange between the sediment in transport and the bed. At flows larger than the design flow or

Figure 7–1 Gabion-armored threshold channel

Figure 7–2 Grass-lined threshold channel

Figure 7–3 Bedrock threshold channel

during extreme events, threshold channels may become destabilized for short periods, with harmful morphological impacts. Since threshold channels do not adjust their dimensions to the natural runoff hydrograph, the concept of channel-forming discharge is generally not applicable.

The design goal of a threshold channel design technique is to produce a channel that has positional or engineering stability. As long as the flows in the channel are below the design discharge, the particles that make up the channel boundary are stable, and the section, plan, and profile of the channel should be essentially static over time. The use of threshold design does not necessarily imply the absence of sediment movement, but rather that the transport capacity is sufficiently large to carry the sediment load through the system without meaningful deposition at boundary stresses less than those required to erode (mobilize) the boundary. For this reason, threshold channels are often designed near the erosion threshold of the boundary during design flows to prevent deposition that would change channel characteristics.

The reader should note that in some literature, the term threshold channel refers to a channel that is at the threshold of movement. In this case, these channels are also referred to as incipient motion channels. This defines a situation where the particles in the channel boundary are at the initiation of motion, not some point below movement. However, as defined in this handbook, the boundary of a threshold channel is below this point for flows up to the design discharge, not directly at the threshold of motion.

(b) Alluvial channels

Alluvial streams and channels have bed and banks formed of material transported by the stream under present flow conditions. There is an exchange of material between the inflowing sediment load and the bed and banks of the stream. The sediment transported in an alluvial channel tends to be coarser and of a larger amount than that transported in a threshold channel. Examples of alluvial channels are shown in figures 7–4 through 7–6. Since natural alluvial channels adjust their width, depth, slope, and planform in response to changes in water or sediment discharge, an alluvial channel will not be as static as a threshold channel.

Figure 7–4 Sand-bed alluvial channel

Figure 7–5 Sand and gravel-bed alluvial channel

Figure 7–6 Gravel-bed alluvial channel

Alluvial channel designs require an analysis of channel stability. An alluvial stream is defined as stable when it has the ability to pass the incoming sediment load without significant degradation or aggradation, and when its width, depth, and slope are fairly consistent over time. The design goal of an alluvial channel design technique is often to produce a channel that has dynamic equilibrium or geomorphic stability. Bank erosion and bankline migration are natural processes and may continue in a stable channel. When bankline migration is deemed unacceptable, then engineering solutions must be employed to prevent bank erosion. Bank protection technology is not addressed in this chapter, but a review of issues and design considerations are in NEH654.14.

(c) Transition channels

A clear distinction between threshold and alluvial channels may not always be apparent. One reach of the stream may be alluvial, while another has the characteristics of a threshold channel. A threshold reach can be changed to an alluvial reach by flattening the slope. A stream may be alluvial at low discharges when there is an adequate sediment supply, and then act like a threshold channel at high discharges. Conversely, a channel may function as a threshold stream at low flows, but during very high discharge become mobile. An example is shown in figure 7–7. In these situations, it is often appropriate to apply both threshold and alluvial channel design techniques.

If an armor layer is present, a stream may be a threshold channel at low flows and on the rising limb of a flood hydrograph, but behave as an alluvial channel at high flows when the armor layer is mobilized, and on the falling limb of the flood hydrograph, when sediment is being deposited. Therefore, it is important to evaluate channels through their entire flow range to determine how they will react to natural inflow conditions and how their stability status may change as a function of discharge.

The armor layer of a gravel bed stream is shown in figure 7–8. Note the much finer subsurface bed material exposed when a few cobbles were removed from the armor layer. Armor layer thickness is typically equal to the D_{90} particle size of the subsurface material. Figure 7–9 shows an armor layer that had formed on the delta of a reservoir and then was destroyed when the water level was lowered.

Figure 7–7 Boulder-bed channel that could be a threshold channel or an alluvial channel, depending on the design discharge

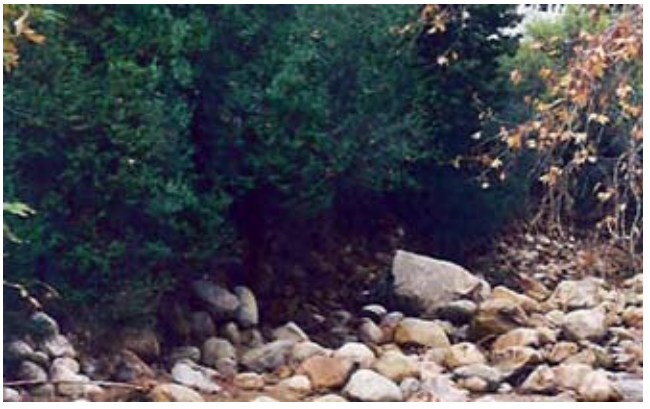

Figure 7–8 Subsurface layer exposed after removal of three cobbles from the armour layer

Figure 7–9 Degradation through armor layer

654.0703 Perennial, intermittent, and ephemeral streams

The flow conditions that a channel may experience through the year may also have an influence on the choice of the appropriate channel design technique. Both threshold and alluvial streams may be classified as perennial, intermittent, or ephemeral, depending on the duration of flow over the course of the year. Definitions of these terms are not precise. Following are stream definitions that have been used by the U.S. Geological Survey (USGS) since the early 1920s (Meinzer 1923):

Perennial—A stream that flows continuously. Perennial streams are generally associated with a water table in the localities through which they flow.

Intermittent or seasonal—A stream that flows only at certain times of the year when it receives water from springs or from some surface source such as melting snow in mountainous areas.

Ephemeral—A stream that flows only in direct response to precipitation, and whose channel is above the water table at all times.

A perennial stream is one that almost always has some flow. Osterkamp and Hedman (1982) provide a more definitive definition.

> *A perennial stream is a stream that exhibits a measurable surface discharge more than 80 percent of the time.*

Intermittent streams may be differentiated from ephemeral streams in that intermittent streams flow continuously for periods of at least 30 days. An intermittent stream flows only seasonally or sporadically. At times, the flow may infiltrate into the pores of the bed and flow only as ground water. An ephemeral stream generally flows only after a significant rainfall event. Channel processes and morphology are significantly affected by the fact that the discharge is intermittent.

The concept of channel-forming discharge is most applicable to perennial streams. Channel geometry in alluvial intermittent and ephemeral streams is typically

a remnant of the last major flow event, rather than a theoretical channel-forming discharge (fig. 7–10). In addition, in ephemeral streams, sediment transport most often occurs as a response to infrequent and flashy hydrologic events. These events cause temporal and spatial episodes of aggradation and degradation and a significantly variable sediment yield. Channel reaches under such flow conditions can be out of phase, and this episodic behavior suggests that ephemeral stream channels may be inherently unstable. Thus, the channel-forming discharge concept may not be applicable.

Figure 7–10 Remnant terraces in an ephemeral stream from previous high-flow events

654.0704 Channel design variables

Traditional channel design methods for fixed-boundary or threshold channels focus on efficient flow conveyance where water surface elevation and velocity are of primary importance. The independent hydraulic design variables are the design discharge and channel roughness. The dependent hydraulic design variables are width, depth, and slope. Channel roughness is a dependent variable if there is a choice of boundary materials. In channel design, these dependent variables are adjusted to achieve the desired hydraulic conditions. Attention is given to the hydraulic losses due to changes in the channel configuration and obstructions such as bridge piers and culverts. Hydraulic design can be accomplished using the energy or momentum equations, in conjunction with a resistance equation such as Manning's equation. The channel boundary is assumed to be immobile at the design discharge, and bed-material sediment inflow is negligible. Traditional methods are applicable for the design of flood control, drainage or irrigation channels lined with a nonerodible material, such as concrete or grass, and for earth channels and ditches with bank protection and little or no sediment inflow. Traditional methods can also be used for design and analysis of natural streams, where the stream boundary is immobile.

Channel design becomes more complicated in alluvial channels, where the bed is mobile and where bed-material sediment inflow is significant. In addition to water surface elevation, efficient transport of sediment becomes a focus in the hydraulic design of alluvial channels. Alluvial streams have the capability to adjust their channel geometry to efficiently transport sediment. The design process seeks to achieve a state of dynamic equilibrium by computing and selecting appropriate values for channel geometry. In some cases, site or project constraints make the ideal channel geometry infeasible. In such cases, erosion control features may be designed or sediment removal maintenance plans implemented.

The independent hydraulic design variables for an alluvial stream include the inflowing discharge hydrograph, bed-material gradation, streambank characteristics, and sediment inflow. The dependent

hydraulic design variables for an alluvial stream are width, depth, slope and planform. Hydraulic roughness is generally a function of the bed material, but bank roughness may be considered a dependent variable in some cases. These dependent variables must be selected so that the channel will pass the incoming sediment load without significant degradation or aggradation.

In addition to the energy or momentum equations and a hydraulic resistance equation, a sediment transport equation is needed to calculate appropriate hydraulic geometries. A geomorphic relationship from a reference reach or a selected hydraulic geometry relationship is also required. In some cases, where the existing channel is stable and watershed characteristics are not changing, channel dimensions can be based on a preexisting condition. The design is more challenging when the project reach is unstable due to straightening, channelization, or changing hydrologic or sediment inflow conditions, as is the case in most land use conversion areas. The characteristics of threshold and alluvial channels are summarized in table 7–1.

Table 7–1 Characteristics of threshold and alluvial channels

	Threshold channel	**Alluvial channel**
Channel boundary	Immobile at design discharge	Mobile
Bed-material sediment inflow	Usually small or negligible	Significant
Dependent variables	Width Depth Slope Roughness, if there is a choice of boundary materials	Width Depth Slope Planform Bank roughness Roughness due to obstructions or structures
Independent variables	Design discharge Channel roughness	Design hydrograph Channel-forming discharge Bed-material sediment inflow Bed material Streambank characteristics
Design equations	Energy Momentum Resistance	Energy Momentum Resistance Sediment transport Geomorphic relationship
Design goal with respect to channel stability	Pass the design discharge below the top of bank without mobilizing the boundary	Pass the incoming sediment load without significant aggradation or degradation or planform change

654.0705 Channel design methods and approaches

Channel design approaches can be broadly categorized by their applicability to threshold or alluvial channels. For threshold channels, the recommended design method will provide a stable channel boundary that will not unravel. This is accomplished for a design discharge and a specified channel boundary material. Channel cross-sectional dimensions and channel slope are selected, and velocities and/or shear stresses are calculated iteratively, using the energy or momentum equations and a hydraulic resistance equation, so that calculated values do not exceed acceptable critical values. Hydraulic design methods for threshold channels are well established and available from several sources. The most significant methods are reviewed in NEH654.08. Two methods are recommended for the hydraulic design of threshold channels: the allowable velocity method and the allowable shear stress method. In general, the allowable velocity method is most applicable when the channel will be lined with a variety of different materials, while the allowable shear stress method is often applied in the design of gravel-bed channels. Neither of these methods provides unique solutions for channel dimensions of width, depth, and slope. However, this limitation is not critical to the hydraulic design in terms of stability because the boundary is immobile.

For alluvial channels, hydraulic design methods require sediment transport analysis to ensure sediment continuity through the project reach. The recommended design methodology suggests analytical solutions of resistance and sediment transport equations, in combination with application of fluvial geomorphic principles. When possible, alluvial channels are sized for the channel-forming discharge.

The recommended design method generates a preliminary channel geometry that can transport the incoming water and sediment load for the selected channel-design discharge. Development of this preliminary or initial design geometry is based on a single discharge, the channel-forming discharge. The design philosophy for alluvial channels is to use appropriate fluvial geomorphic principles combined with analytical equations for flow resistance and sediment transport to solve for the dependent design variables of width, depth, slope, and planform. Geomorphic principles that can be used with the analytical equations include analogy methods, hydraulic geometry, and the extremal hypothesis. Project constraints often narrow the range of feasible solutions. Alluvial channel design techniques are addressed in more detail in NEH654.09.

The long-term stability of the preliminary channel design is evaluated using a flow-duration curve or a long-term hydrograph that includes the full range of discharges. Sediment impact analysis is described in NEH654.13. Design adjustments may then be made to the channel design based on issues related to stability, flood effects, and sedimentation. Characteristics of the hydraulic design philosophies for threshold and alluvial channels are shown in table 7–2.

Table 7–2 Hydraulic design philosophies

	Threshold channels	Alluvial channels
Design discharges	Maximum design discharge	Channel-forming discharge Flow-duration curve and/or long-term hydrograph
Design criteria	Critical velocity/shear stress	Continuity of sediment
Dependent variables	Width, depth, and slope (roughness if there is a choice of boundary material)	Width, depth, slope, planform, bank roughness, and roughness due to obstructions or structures
Design equations	Energy, momentum, and hydraulic resistance	Energy, momentum, hydraulic resistance, sediment transport, and geomorphic relationship

Some of the analysis, which is based on a threshold assumption, is also used in the design alluvial channel. Some of the degrees of movement that an alluvial channel may undergo may not be permissible. Hard or threshold design techniques may be used to restrict stream movement towards a road or a building, for example. Threshold methods are also used to design stream features such as toe protection, riffles, spurs, barbs, vanes, and deflector dikes. The use and design of these features are described in NEH654.14.

Threshold channels are designed so that the streambed is immobile for the full range of natural discharges, as long as these discharges are below the design flow. In alluvial channels, it is important to determine the discharge at which the streambed begins to move. This can be accomplished using the threshold criteria described in NEH654.08 and is especially important in a channel with an armor layer. Sediment transport capacity dramatically increases when the armor layer is disrupted or destroyed, and the coarse material becomes thoroughly mixed with the substrate material. Stability of vegetated or gravel banks can be determined using allowable velocity methods or shear stress methods. A mobile streambed is not necessarily unstable, but mobile beds require a higher level of analysis to determine stability, within the context of the limitations or requirements of the design.

(a) Analogy method

The analogy method is used to select channel dimensions and is based on the premise that conditions in a reference reach with similar characteristics and watershed conditions can be copied to the project reach. The method can be used for both threshold and alluvial channels, but if used for threshold channel design, bed stability in the project channel should be checked using threshold methods. For alluvial channels, the analogy method is used to select one of the primary dependent design variables of width, depth, or slope (preferably width). The design width is adapted from a selected reference reach, and the remaining two variables are calculated, using hydraulic resistance and sediment transport equations.

Planform can also be determined using the analogy method. The reference reach must be stable and alluvial and have the same channel-forming discharge as the project reach. A stable channel is one in which the stream's planform, cross section, and longitudinal profile are sustainable. Channel features may migrate laterally and longitudinally. The reference reach may be upstream or downstream from the project reach, or in a physiographically similar watershed. The bed and banks in the project and reference reaches must be composed of similar material, and there should be no significant hydrologic, hydraulic, or sediment differences in the reaches.

If a stable predisturbance width and planform can be identified, then the preexisting channel dimensions can be used with the analogy approach. This is feasible if historical width and planform can be determined from mapping, aerial photos, and/or soil borings. This technique is generally not applicable if the watershed water and sediment runoff characteristics or the base level have changed over time.

(b) Hydraulic geometry method

A suitable hydraulic geometry relationship can be used to select a value for one of the dependent variables for the channel-forming discharge. The hydraulic geometry method is similar to the analogy method, but it is more useful because a range of discharges is used. Hydraulic geometry theory is based on the concept that a river system tends to develop in a predictable way, producing an approximate equilibrium between the channel and the inflowing water and sediment (Leopold and Maddock 1953). The theory typically relates a dependent variable, such as width or slope, to an independent or driving variable such as channel-forming discharge or drainage area.

Hydraulic geometry relations are sometimes stratified according to bed-material size, bank vegetation, or bank material type. Rosgen (1998) suggests the use of stream classification as an appropriate tool for differentiating hydraulic geometry relations. Hydraulic geometry relationships are developed from field observations at stable and alluvial cross sections. These relationships were originally used as descriptors of geomorphic trends. Data scatter is expected about the developed curve, even in the same river reach (as described and shown in NEH654.0905. It is important to recognize that this scatter represents a valid range of stable channel configurations due to variables such as geology, vegetation, land use, sediment load and gradation, and runoff characteristics. The transfer of

hydraulic geometry relationships developed for one watershed to another watershed should be performed with extreme care. The two watersheds should be similar in historical land use, physiography, geology, hydrologic regime, precipitation, and vegetation.

Both the hydraulic geometry method and the analogy method depend on comparison to channels that are fully adjusted. Specifically, the reference reach, or a channel whose dimensions are used in a hydraulic geometry plot, are not evolving to a different form. If the watershed in which the channel to be designed is likely to change due to changes in water and sediment supply, this assumption can be problematic.

(c) Analytical method

Once one of the dependent design variables (preferably width) is determined using analogy or hydraulic geometry methods, the other two dependent design variables (depth and slope) should be calculated using an analytical, or computational, method. This is accomplished using one of several resistance and sediment transport equations available in the literature. If the resistance and sediment transport equations are solved simultaneously for a specified channel-forming discharge, a family of solutions can be calculated.

The analytical solution for depth and slope that matches the analogy or hydraulic geometry solution for width provides the three dependent design variables. The analytical family of solutions can also be used without the analogy or hydraulic geometry methods to determine the third dependent design variable. The wide range of possible solutions from the analytical calculations can be narrowed by the assigned project constraints. For example, a maximum width constraint might be imposed by right-of-way limits, and a maximum depth constraint might be imposed by flood control considerations. The valley slope would impose a maximum slope constraint. Another approach is to assume that the channel will form its geometry such that the minimum amount of energy is expended. This assumption will provide a unique solution at the minimum slope on the family of solutions.

Characteristics of the analogy, hydraulic geometry, and analytical design methods are summarized in table 7–3.

Table 7–3 Characteristics of analogy, geometry, and analytical hydraulic design methods

	Basis	Requirements	Recommended for determination of
Analogy	Channel dimensions from a reference reach can be transferred to another location	Reference reach must be stable and alluvial Reference reach must have same channel-forming discharge, valley slope, and similar bed and bank characteristics	Top width of channel-forming discharge channel and planform
Hydraulic geometry	Channel dimensions can be determined from regression relationships with independent variables	Regression curves must be developed from stable and alluvial reaches and from physiographically similar watersheds	Top width of channel-forming discharge channel and planform
Analytical	Depth and sediment transport can be calculated from physically based equations	Estimates of bed-material gradation and resistance coefficients must be obtained	Depth and slope

(d) Hybrid design techniques

Several techniques are available that include a combination of analytical, as well as analogy and hydraulic geometry design methods. Two of these techniques are presented in this handbook.

NEH654.10 presents a two-stage channel design approach for drainage ditches. This is a modification of many of the commonly used threshold design techniques to provide a floodway bench. The intent of this technique is to better mimic alluvial processes by providing a flood plain within the ditch.

NEH654.11 outlines a channel design technique based on the morphological and morphometric qualities of the Rosgen classification system. This approach is often referred to as the Rosgen design approach. The essence for this design approach is based upon measured morphological relations associated with bankfull flow, geomorphic valley type, and geomorphic stream type.

654.0706 Sediment impact assessment

The energy of flowing water constantly reconfigures the physical form of flood plain and stream habitats, primarily through modification of alluvial topography by fluvial action. However, to maintain an equilibrium of channel structure and function, especially in the context of riverine fisheries habitat, natural mechanisms that supply, transport, and deposit watershed materials must remain operative along the river continuum, from the basin to the reach-level scale. Alluvial and threshold channels maintain channel geometries that reflect the quantity of water and the size and characteristics of sediment delivered to them from their drainage basins. Maintenance of channel form and function requires that all of the mass and sizes of sediment supplied to the channel be transported in equilibrium, so that over the long term, the channel neither aggrades nor degrades.

A sediment impact assessment should be conducted for all projects involving changes to the existing channel or the creation of a new channel. This can be accomplished using visual or qualitative assessments for relatively simple projects or by using a numerical model that incorporates solution of the sediment continuity equation for more complex projects. The choice of the appropriate technique to assess the sediment impact of a proposed project includes an assessment of not only the project goals, type of channel, and watershed condition but also an assessment of the impact of project failure. Sediment impact assessments are described in more detail in NEH654.13.

654.0707 Conclusion

The following channel design chapters of this handbook present and describe several systematic hydraulic design methodologies and design techniques. The objective of each of these methodologies is to fit the channel design into the natural system within the physical constraints imposed by other project objectives and constraints. Some techniques are more appropriate for conditions where the design channel boundary is expected to be immobile at design flows, while others are more applicable to conditions where the design channel is expected to be in dynamic equilibrium with its sediment load.

Where appropriately applied, each of the presented design methodologies should be systematic; that is, when used by different engineers with the same project objectives, design results should be similar. However, since each technique is based on different assumptions and is applicable to different conditions, it should not be expected that all of the techniques will result in exactly the same design. The technique or approach that is selected should be appropriate not only for the project goals but also the nature of the sediment-flow exchange with the channel boundary. The physical principles upon which these approaches are based are outlined in the following chapters. The user should evaluate these to determine the applicability to the project specific site. Where there is uncertainty in the nature of the channel and the appropriateness of the design technique, many designers use several techniques and look for agreement on critical design elements.

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**Natural
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Conservation
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**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 8 Threshold Channel Design

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Cover photo: Threshold channels have erosion-resistant boundaries.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 8 Threshold Channel Design

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Chapter 8 Threshold Channel Design

654.0800 Purpose

Threshold channel design techniques are used for rigid boundary systems. In a threshold channel, movement of the channel boundary is minimal or nonexistent for stresses at or below the design flow condition. Therefore, the design approach for a threshold channel is to select a channel configuration where the stress applied during design conditions is below the allowable stress for the channel boundary. Many sources and techniques for designing stable threshold channels are available to the designer. This chapter provides an overview and description of some of the most common threshold channel design techniques. Examples have been provided to illustrate the methods.

654.0801 Introduction

A stable threshold channel has essentially rigid boundaries. The streambed is composed of very coarse material or erosion-resistant bedrock, clay soil, or grass lining. Streams where the boundary materials are remnants of processes no longer active in the stream system may be threshold streams. Examples are streambeds formed by high runoff during the recession of glaciers or dam breaks, streams armored due to degradation, and constructed channels where channel movement is unacceptable for the design flow.

A threshold channel is a channel in which movement of the channel boundary material is negligible during the design flow. The term threshold is used because the applied forces from the flow are below the threshold for movement of the boundary material. Therefore, the channel is assumed to be stable if the design stress is below the critical or recommended stress for the channel boundary. Design issues include assessing the limiting force and estimating the applied force. A requirement for a channel to be considered a threshold channel is that the sediment transport capacity must greatly exceed the inflowing sediment load so that there is no significant exchange of material between the sediment carried by the stream and the bed. Noncohesive material forming the channel boundary must be larger than what the normal range of flows can transport. For boundaries of cohesive materials, minor amounts of detached material can be transported through the system.

Threshold channels, therefore, transport no significant bed-material load. Fine sediment may pass through threshold streams as throughput. In general, this throughput sediment should not be considered part of the stream boundary for stability design purposes, even if there are intermittent small sediment deposits on the streambed at low flow.

An additional requirement for threshold channel design is to maintain a minimum velocity that is sufficient to transport the sediment load through the project reach. This sediment may consist of clays, silts, and fine sands. This is necessary to prevent aggradation in the threshold channel.

Threshold channels differ from movable bed or alluvial channels which show interaction between the incoming sediment load, flow, and channel boundary. In an alluvial channel, the bed and banks are formed from material that is transported by the stream under present flow conditions. The incoming sediment load and bed and bank material of an alluvial channel interact and exchange under design or normal flow conditions. Essentially, the configuration of a threshold channel is fixed under design conditions. An alluvial channel is free to change its shape, pattern, and planform in response to short- or long-term variations in flow and sediment. The design of alluvial channels is addressed in detail in NEH654.09.

Approaches that fall into four general categories for the design of threshold channels are addressed in this chapter. These approaches are the permissible velocity approach, allowable shear stress approach, and allowable tractive power approach. The grass-lined channel design approach, which is a specific case of either the permissible velocity or allowable shear stress approach, is also described. Table 8–1 provides general guidance for selecting the most appropriate design technique. This is a general guide, and there are certainly exceptions. For example, the allowable velocity technique, being the most historical, has been applied more broadly than indicated in table 8–1. Where there is uncertainty regarding the appropriate technique, it is recommended that the designer use several of the most appropriate techniques and look for agreement on critical design elements.

654.0802 Design discharges

Threshold channel design methods are appropriate where sediment inflow is negligible and the proposed channel boundary is to be immobile, even at high flows. Threshold channels do not have the freedom to adjust their geometry under normal flow conditions. Therefore, channel-forming discharge is not necessarily a critical factor in determining channel dimensions in a threshold channel. Design flows are traditionally based, at least in part, on programs and policy decisions.

As described in NEH654.07, the classification of a stream as alluvial or threshold may not be clear. One reach of the stream may be alluvial, while another may have the characteristics of a threshold channel. A threshold stream reach can be changed to an alluvial reach by flattening the slope to induce aggradation or increasing the slope so that the boundary material becomes mobile. At flows larger than the design flow or during extreme events, threshold channels may develop a movable boundary. It is important to evaluate channels through their entire flow range to determine how they will react to natural inflow conditions.

Design of a stream project may involve a hybrid approach. For example, project goals may require that the planform is rigid, while the cross section can vary. In this situation, a design approach might be to

Table 8–1 General guidance for selecting the most appropriate channel design technique

Technique	Significant sediment load and movable channel boundaries	Boundary material smaller than sand size	Boundary material larger than sand size	Boundary material does not act as discrete particles	No baseflow in channel. Climate can support permanent vegetation
Allowable velocity		X			
Allowable shear stress			X		
Tractive power				X	
Grass lined/tractive stress					X
Alluvial channel design techniques	X				

stabilize the grade and toe of a stream in place, and allow the upper bank to adjust naturally. Threshold channel design approaches, such as the use of riprap (NEH654.14), are also used to size stream features such as toe protection, riffles, stream barbs, and deflector dikes.

654.0803 Allowable velocity method

The allowable or permissible velocity approach is typically used with channels that are lined with grass, sand, or earth. Limiting forces for soil bioengineering and manufactured protective linings can also be expressed as permissible velocities.

To design a threshold channel using the allowable velocity method, average channel velocity is calculated for the proposed channel and compared to published allowable velocities for the boundary material. The average channel velocity in the design channel can be determined using a normal depth equation or a computer backwater model. Increased velocities at bends can be accounted for, using applicable charts and equations. Allowable velocities have been determined for a large variety of boundary materials and are provided in many texts and manuals. These tables have primarily been applied to the design of irrigation and drainage canals and were developed from data in relatively straight, uniform channels with depths less than 3 feet. It is common practice to apply allowable velocity data in meandering, nonuniform channels with depths greater than 3 feet, but such application should be done with caution. Allowable velocities can be increased or decreased to account for such irregularities as meandering alignments and increased sediment concentrations, using applicable charts. Allowable velocities are somewhat less than critical velocities so that a factor of safety is included in the values presented.

(a) Calculate average velocity

The first step in applying the allowable velocity design approach is to calculate the average velocity of the existing or proposed channel. Computing the average channel velocity requires a design discharge, cross section, planform alignment, average energy slope, and flow resistance data. If the design channel is a compound channel, it may be necessary to divide the channel into panels and calculate velocities for each panel. In channels with bends, the velocity on the outside of the bend may be significantly higher than the average velocity. Velocity can be calculated using normal depth assumptions or by a more rigorous backwater analysis if a gradually varied flow assumption is more appropriate.

A normal depth calculation is easier than a backwater analysis and can be accomplished using a flow resistance equation such as Manning's. The normal depth assumption is applicable for uniform flow conditions where energy slope, cross-sectional shape, and roughness are relatively constant in the applicable reach. In a natural channel, with a nonuniform cross section, reliability of the normal depth calculation is directly related to the reliability of the input data. Sound engineering judgment is required in the selection of a representative cross section. The cross section should be located in a uniform reach where flow is essentially parallel to the bank line with no reverse flow or eddies. This typically occurs at a crossing or riffle. Determination of the average energy slope can be difficult. If the channel cross section and roughness are relatively uniform, water surface slope can be used. Thalweg slopes and low-flow water surface slopes may not be representative of the energy slope at design flows. Slope estimates should be made over a significant length of the stream (a meander wavelength or 20 channel widths).

A computer program such as the U.S. Army Corps of Engineers (USACE) HEC–RAS can be used to perform these velocity calculations. Such programs allow the designer to account for nonuniform sections and for backwater conditions that may occur behind a bridge or at a constriction. The calculation of hydraulic parameters for both existing and proposed channels is critically important to design. A more complete treatment of the subject is provided in NEH654.06.

Minimum radius of curvature

Caution is recommended in applying this approach on channels with sharp bends. Section 16 of the National Engineering Handbook (U.S. Department of Agriculture (USDA) Soil Conservation Service (SCS) 1971) provides guidance for minimum radius of curvature for drainage ditches with very flat topography (slopes less than 0.00114). Table 8–2 provides guidance for channels in stable soil without bank protection. Conditions outside the range of table 8–2 and in erodible soils require use of the more detailed analysis provided in this chapter. The curved channel may require bank protection.

Maximum velocity in bends

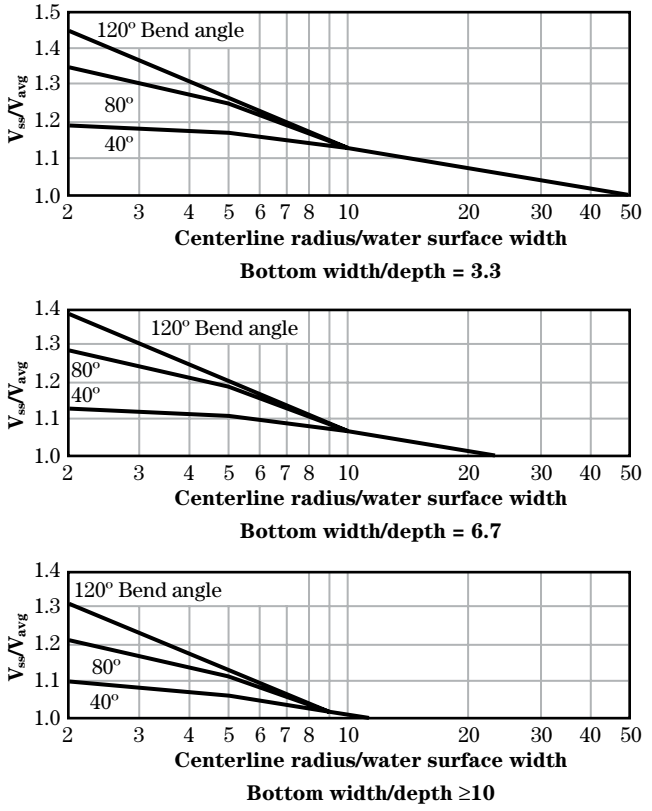
Adjustments to the calculated average channel velocity that account for flow concentration around bends is provided as part of the USACE riprap design method (USACE 1991b.) The method is based on a large body of laboratory data and has been compared to available prototype data (Maynord 1988). The method is applicable to side slopes of 1V:1.5H or flatter. The method calculates a characteristic velocity for side slopes,

Table 8–2 Suggested minimum radius of curvature in stable soils without bank protection

Type of ditch	Slope	Minimum radius of curvature (ft)	Minimum radius of curvature (m)	Approximate degree of curve (degrees)
Small ditches with maximum top width 15 ft (4.6 m)	<0.00057	300	90	19
	0.00057 to 0.00114	400	120	14
Medium-sized ditches with top width 15 to 35 ft (4.6–10.7 m)	<0.00057	500	150	11
	0.00057 to 0.00114	600	180	10
Large ditches with top width >35 ft (10.7 m)	<0.00057	600	180	10
	0.00057 to 0.00114	800	240	7

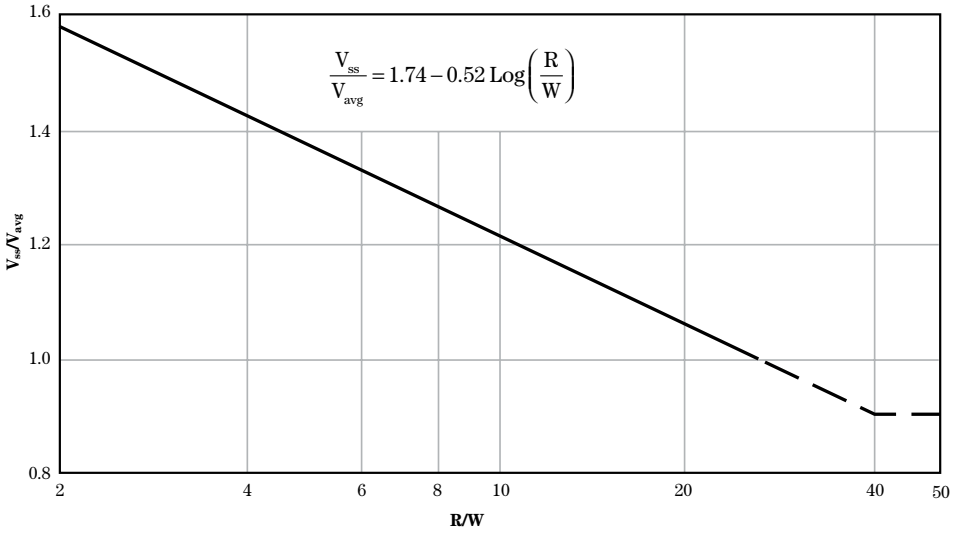
V_{ss}, which is the depth-averaged local velocity over the side slope at a point 20 percent of the slope length from the toe of the slope. This has been determined to be the part of the side slope that experiences the maximum flow velocity. The ratio V_{ss}/V_{avg}, where V_{avg} is the average channel velocity at the upstream end of the bend, has been determined to be a function of the ratio of the of centerline radius of curvature, R, and the water surface width, W. Figure 8–1 illustrates the relationship for natural channels. Figure 8–2 illustrates the relationship for trapezoidal channels. The data for trapezoidal channels shown in figure 8–2 are based on numerical model calculations described in Bernard (1993). The primary factors affecting velocity distribution in riprap lined bendways are R/W, bend angle, and aspect ratio (bottom width-to-depth). V_{avg}, R, and W should be based on main channel flow only and should not include overbank areas.

Figure 8–2 Design velocities for trapezoidal channels



Notes: V_{ss} is depth-averaged velocity at 20% of slope length up from toe, maximum value in bend. Curves based on STREMR model (Bernard 1993), V_{avg} = 6 ft/s, 1V:3H side slopes. n = 0.038, 15 ft depth

Figure 8–1 Design velocities for natural channels. Note: V_{ss} is depth-averaged velocity at 20% of slope length from toe



(b) Determine allowable velocity

The design velocity of the existing or proposed channel must be compared to the allowable velocity for the channel boundary. The allowable velocity is the greatest mean velocity that will not cause the channel boundary to erode. Since the allowable velocity is a design parameter that has a factor of safety, it is somewhat less than the critical velocity (the velocity at incipient motion of the boundary material).

The allowable velocity can be approximated from tables that relate boundary material to allowable velocity, but tabular estimates should be tempered by experience and judgment. In general, older channels have higher allowable velocities because the channel boundary typically becomes stabilized with the deposition of colloidal material in the interstices. Also, a deeper channel will typically have a higher allowable velocity than shallow channels because erosion is a function of the bottom velocity. Bottom velocities in deep channels are less than bottom velocities in shallow channels with the same mean velocity.

Fortier and Scobey (1926) presented a table of maximum permissible velocities for earthen irrigation canals with no vegetation or structural protection. Their work was compiled based on a questionnaire given to a number of experienced irrigation engineers and was recommended for use in 1926 by the Special Committee on Irrigation Research of the American Society of Civil Engineers. This compilation is presented in table 8–3.

USACE (1991b) provides allowable velocity criteria for nonscouring flood control channels in table 8–4.

Theoretical objections to use of average velocity as an erosion criterion can be overcome by using depth as a second independent variable. An example of a velocity-depth-grain size chart from the USACE (1991b) is shown in figure 8–3. This particular chart is intended to correspond to a small degree of bed movement, rather than no movement. Values given in this chart are for approximate guidance only.

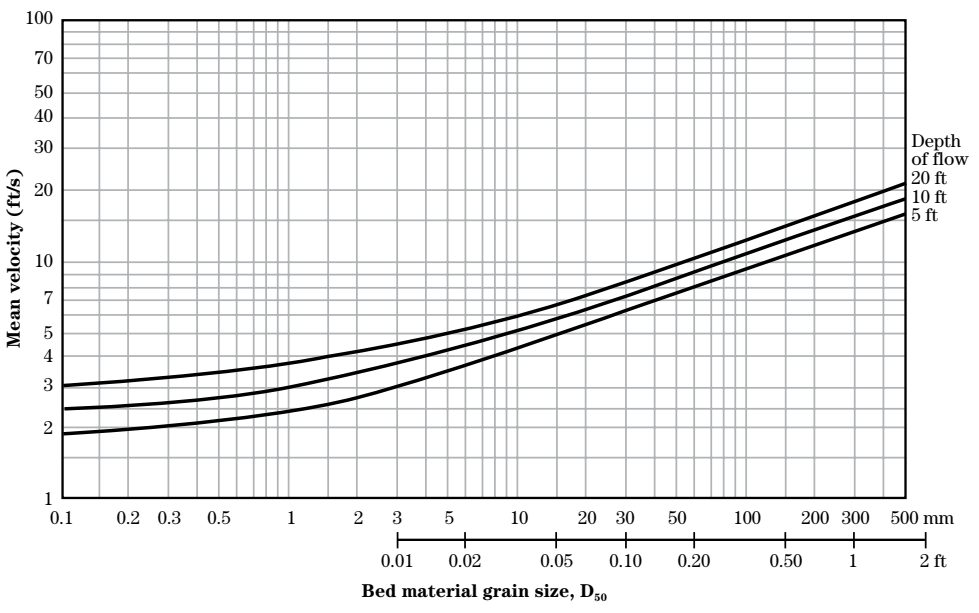
Table 8–3 Maximum permissible canal velocities

	Mean velocity, for straight canals of small slope, after aging with flow depths less than 3 ft (0.9 m)					
	Clear water, no detritus		Water transporting colloidal silts		Water transporting noncolloidal silts, sands, gravels, or rock fragments	
Original material excavated for canals	**ft/s**	**m/s**	**ft/s**	**m/s**	**ft/s**	**m/s**
Fine sand (noncolloidal)	1.5	0.46	2.5	0.76	1.5	0.46
Sandy loam (noncolloidal)	1.75	0.53	2.5	0.76	2.0	0.61
Silt loam (noncolloidal)	2.0	0.61	3.0	0.91	2.0	0.61
Alluvial silt (noncolloidal)	2.0	0.61	3.5	1.07	2.0	0.61
Ordinary firm loam	2.5	0.76	3.5	1.07	2.25	0.69
Volcanic ash	2.5	0.76	3.5	1.07	2.0	0.61
Stiff clay (very colloidal)	3.75	1.14	5.0	1.52	3.0	0.91
Alluvial silt (colloidal)	3.75	1.14	5.0	1.52	3.0	0.91
Shales and hardpans	6.0	1.83	6.0	1.83	5.0	1.52
Fine gravel	2.5	0.76	5.0	1.52	3.75	1.14
Graded, loam to cobbles (when noncolloidal)	3.75	1.14	5.0	1.52	5.0	1.52
Graded silt to cobbles (when colloidal)	4.0	1.22	5.5	1.68	5.0	1.52
Coarse gravel (noncolloidal)	4.0	1.22	6.0	1.83	6.5	1.98
Cobbles and shingles	5.0	1.52	5.5	1.68	6.5	1.98

Table 8–4 Allowable velocities

Channel material	Mean channel velocity (ft/s)	(m/s)
Fine sand	2.0	0.61
Coarse sand	4.0	1.22
Fine gravel	6.0	1.83
Earth		
Sandy silt	2.0	0.61
Silt clay	3.5	1.07
Clay	6.0	1.83
Grass-lined earth (slopes <5%)		
Bermudagrass		
Sandy silt	6.0	1.83
Silt clay	8.0	2.44
Kentucky bluegrass		
Sandy silt	5.0	1.52
Silt clay	7.0	2.13
Poor rock (usually sedimentary)	10.0	3.05
Soft sandstone	8.0	2.44
Soft shale	3.5	1.07
Good rock (usually igneous or hard metamorphic)	20.0	6.08

Figure 8–3 Allowable velocity-depth grain chart



(c) Soil Conservation Service allowable velocity approach

Basic allowable velocities may be determined from figure 8–4 (USDA SCS 1977). In this figure, allowable velocities are a function of sediment concentration, grain diameter for noncohesive boundary material, and plasticity index and soil characteristics for cohesive boundary material. Adjustments are given in figure 8–4 to the basic allowable velocity to account for frequency of design flow, alignment, bank slope, depth of flow, and sediment concentration for both discrete particles and cohesive soils. These design charts were compiled from the data of Fortier and Scobey (1926), Lane (1955a), and the Union of Soviet Socialist Republic (USSR) (1936). Soil materials are classified using the Unified Soil Classification System.

Procedure for application of allowable velocity method (USDA SCS 1977)

Step 1 Determine the hydraulics of the system. This includes hydrologic determinations, as well as the stage-discharge relationships for the channel considered.

Step 2 Determine the soil properties of the bed and banks of the design reach and of the channel upstream.

Step 3 Determine the concentration of the suspended sediment load entering the reach. This is best accomplished by measurements. Channels with suspended sediment concentrations less than 1,000 parts per million are considered sediment free for this analysis, in that the sediment load is not sufficient to decrease the energy of the stream flow. Sediment-free flows are, therefore, considered to have no effect on channel stability. Channels with suspended sediment concentrations greater than 20,000 parts per million are considered to be sediment laden. Sediment-laden flows are considered to enhance stream stability by filling boundary interstices with cohesive material. If a significant portion of the inflowing sediment load is bed-material load, it is likely that the channel is alluvial, and threshold design methods are not applicable.

Step 4 Check to see if the allowable velocity procedure is applicable using table 8–1.

Step 5 Determine the basic average allowable velocities for the channel from one or more of the available design guidelines (tables 8–3, 8–4, fig. 8–4 (USDA SCS 1977; Federal Interagency Stream Restoration Working Group (FISRWG) 1998)).

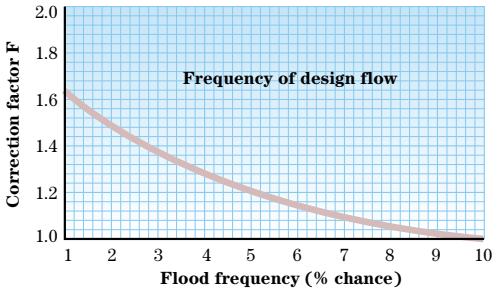
Step 6 Multiply the basic allowable velocity by the appropriate correction factors (fig. 8–4).

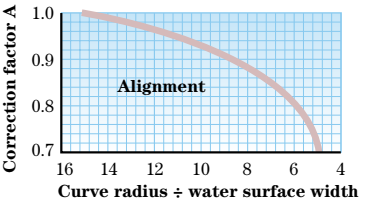
Step 7 Compare the design velocities with the allowable velocities. If the allowable velocities are greater than the design velocities, the design is satisfactory. Otherwise, three options are available:

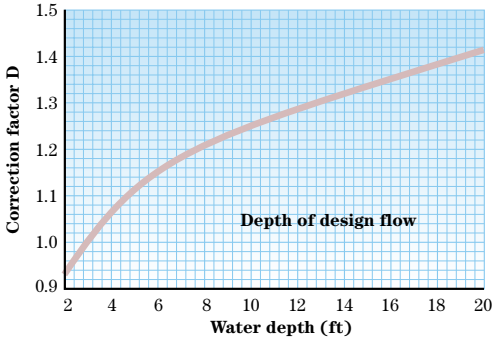
- Redesign the channel to reduce velocity.
- Provide structural measures (riprap, grade control) to prevent erosion.
- Consider a mobile boundary condition and evaluate the channel using appropriate sediment transport theory and programs.

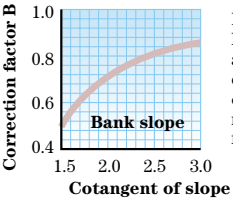
Design of Open Channels, TR–25 (USDA SCS 1977) contains several examples to guide the user through the allowable velocity approach.

Figure 8–4 Allowable velocities for unprotected earth channels

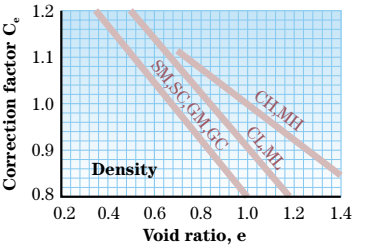




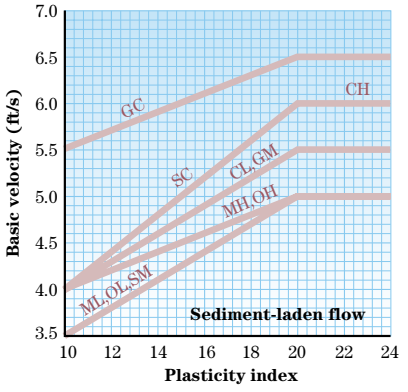


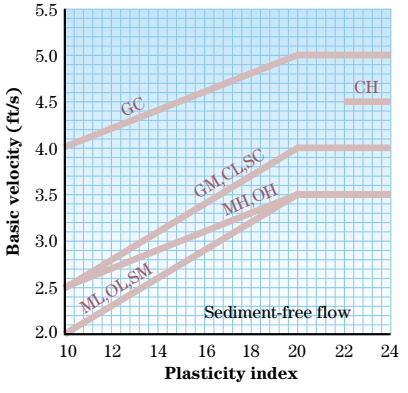


Notes:
In no case should the allowable velocity be exceeded when the 10% chance discharge occurs, regardless of the design flow frequency.

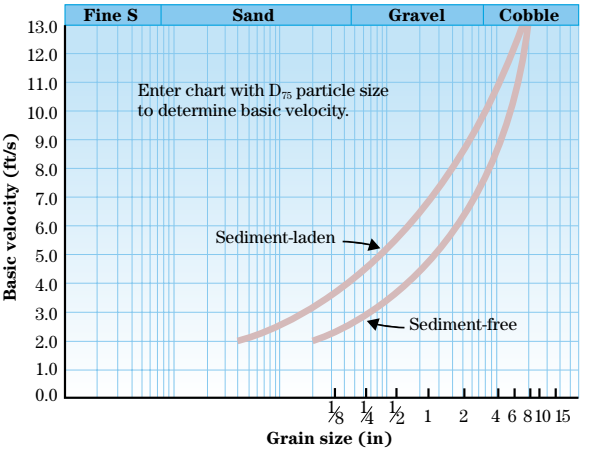


Basic velocities for coherent earth materials, v_b





Basic velocity for discrete particles of earth materials, v_b



Allowable velocities for unprotected earth channels	
Channel boundary materials	Allowable velocity
Discrete particles	
Sediment-laden flow	
D_{75} >0.4mm	Basic velocity chart value x D x A x B
D_{75} <0.4mm	2.0 ft/s
Sediment-free flow	
D_{75} >0.2mm	Basic velocity chart value x D x A x B
D_{75} <0.2mm	2.0 ft/s
Coherent earth materials	
PI >10	Basic velocity chart value x D x A x F x C_e
PI <10	2.0 ft/s

Source: USDA Soil Conservation Service, 1977

654.0804 Allowable shear stress approach

The allowable shear approach (sometimes referred to as the tractive stress approach) is typically used with channels that are lined with rock, gravel, or cobbles. Limiting forces for soil bioengineering and manufactured protective linings can also be expressed as allowable shear, as well.

To design a threshold channel using the allowable shear stress approach, the average applied grain bed shear stress is compared to the allowable shear stress for the boundary material. The applied grain bed shear stress can be calculated from the hydraulic parameters determined for the design channel and the characteristics of the channel boundary material. The hydraulic parameters are calculated using the same methods as in the allowable velocity approach. For noncohesive soils, the average allowable shear stress can be calculated using a critical shear stress approach and then adding a factor of safety or by using an empirical equation with a factor of safety included. For cohesive particles, the electrochemical bonds related primarily to clay mineralogy, are the most significant sediment properties that determine allowable shear stress. Although some empirical data are available, laboratory tests to determine allowable shear stress for a specific cohesive soil are preferred.

(a) Calculate applied shear stress

The first step in applying this approach is to calculate the hydraulics of the study reach. The total average shear stress on the boundary can be approximated from equation 8–1, using any consistent units of measurement:

$$\tau_o = \gamma RS \qquad \text{(eq. 8–1)}$$

where:

τ_o = total bed shear stress (lb/ft^2 or N/m^2)
γ = specific weight of water (lb/ft^3 or N/m^3)
R = hydraulic radius (ft or m)
S = energy slope, dimensionless

In wide channels where the width is more than 10 times the depth, R is generally taken to be equal to the depth. Spatial and temporal variation may result in a higher or lower point value for shear stress. The equation approximates average bed shear stress.

The shear stress can also be expressed as a function of the velocity and the ratio of hydraulic radius and boundary roughness. Keulegan (1938) presented such a formula.

$$\tau = \frac{\rho V^2}{\left(\frac{1}{\kappa}\ln\frac{R}{k_s} + 6.25\right)^2} \qquad \text{(eq. 8–2)}$$

where:

V = depth-averaged velocity, ft/s or m/s
ρ = density of water, lb-s^2/ft^4(slugs/ft^3) or kg/m^2
κ = von Karman's constant (usually taken to be 0.4)
k_s = roughness height, ft or m

Actual shear stress values should be calculated for the banks, as well as for the bed of a trapezoidal earth channel. Maximum stresses occur near the center of the bed and at a point on the bank about a third up from the bottom. The designer should note that computer programs such as HEC–RAS may only provide average boundary shear stress in the output. For most trapezoidal sections and depths of flow, bed stress values are somewhat higher than bank stress. Figures 8–5 and 8–6 provide actual shear stress values for the bed and sides of straight trapezoidal channels in coarse grained soil materials.

Grain shear stress

The total applied bed shear stress may be divided into that acting on the grains and that acting on the bedforms. Entrainment and sediment transport are a function only of the grain shear stress; therefore, the grain shear stress is the segment of interest for threshold design. Einstein (1950) determined that the grain shear stress could best be determined by separating total bed shear stress into a grain component and a form component, which are additive. The equation for total bed shear stress is:

$$\tau_o = \tau' + \tau'' = \gamma RS \qquad \text{(eq. 8–3)}$$

where:

τ' = grain shear stress (shear resulting from size of the material on the bed)
τ'' = form shear stress (shear resulting from bed irregularities due to bedforms)

Figure 8–5 Applied maximum shear stress, τ_b, on bed of straight trapezoidal channels relative to an infinitely wide channel, τ_∞

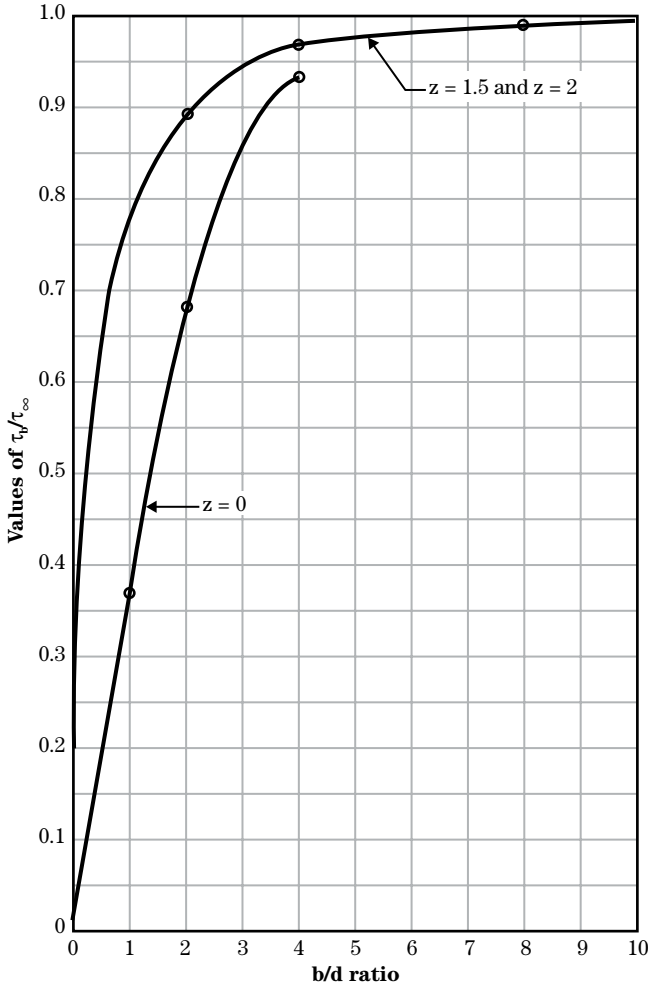
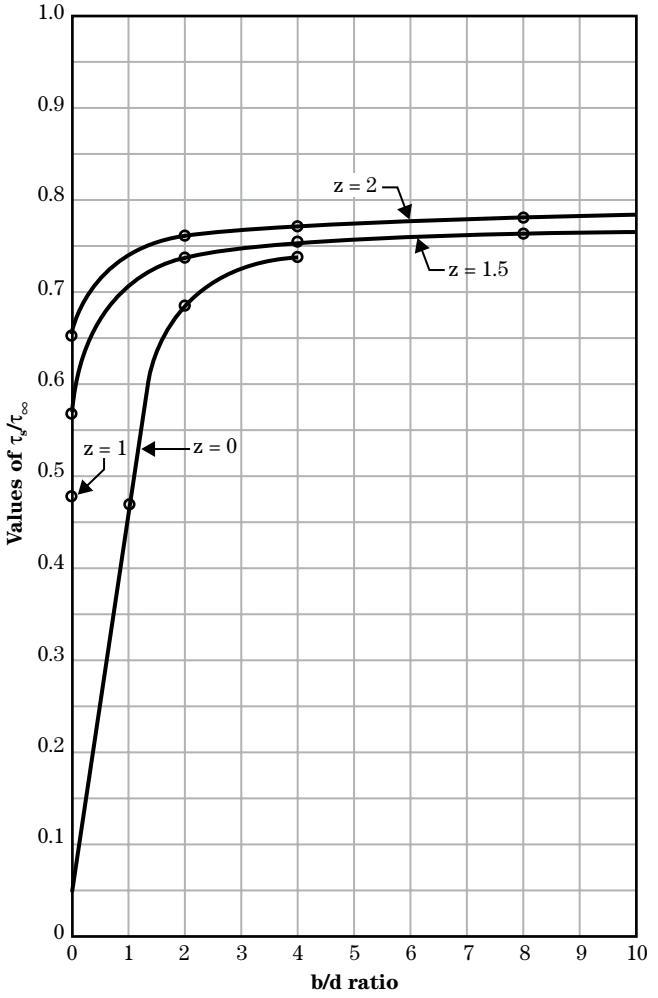


Figure 8–6 Applied maximum shear stress, τ_s, on sides of trapezoidal channels relative to an infinitely wide channel, τ_∞



Note:
b = bottom width
d = depth
z = side slope, zH:1V
τ_∞ = shear stress on a straight, infinitely wide channel
τ_b = applied shear stress on a channel bed
τ_s = applied shear stress on the side of a channel

Einstein also suggested that the hydraulic radius could be divided into grain and form components that are additive. The equations for grain and form shear stress then become:

$$\tau' = \gamma R' S \qquad \text{(eq. 8–4)}$$

$$\tau'' = \gamma R'' S \qquad \text{(eq. 8–5)}$$

where:
R' = hydraulic radii associated with the grain roughness
R'' = hydraulic radii associated with the form roughness

These hydraulic radii are conceptual parameters, useful for computational purposes and have no tangible reality. The total bed shear stress can be expressed as:

$$\tau_0 = \gamma R' S + \gamma R'' S \qquad \text{(eq. 8–6)}$$

Slope and the specific weight of water are constant so that the solution is to solve for one of the *R* components. The grain shear stress can be calculated with the Limerinos equation, using any consistent units of measurements.

$$\frac{V}{U'_*} = 3.28 + 5.66 \log_{10} \frac{R'}{D_{84}} \qquad \text{(eq. 8–7)}$$

$$U'_* = \sqrt{gR'S} \qquad \text{(eq. 8–8)}$$

where:
V = average velocity (ft/s or m/s)
U'_* = grain shear velocity (ft/s or m/s)
D_{84} = particle size for which 84% of the sediment mixture is finer (ft or m)
g = acceleration of gravity (ft/s^2 or m/s^2)

Limerinos (1970) developed his equation using data from gravel-bed streams. Limerinos' hydraulic radii ranged between 1 and 6 feet; D_{84} ranged between 1.5 and 250 millimeters. This equation was confirmed for plane bed sand-bed streams by Burkham and Dawdy (1976). The equation can be solved iteratively for R' and τ', when average velocity, slope, and D_{84} are known.

Whenever the streambanks contribute significantly to the total channel roughness, the applied shear stress to the banks must be accounted for. This is accomplished using the sidewall correction procedure, which separates total roughness into bed and bank roughness and conceptually divides the cross-sectional area into additive components. The procedure is based on the assumption that the average velocity and energy gradient are the same in all segments of the cross section.

$$A_{total} = A_b + A_w \qquad \text{(eq. 8–9)}$$

$$A_{total} = P_b R_b + P_w R_w \qquad \text{(eq. 8–10)}$$

where:
A = cross-sectional area (ft^2 or m^2)
P = perimeter (ft or m)

Subscripts b and w are associated with the bed and wall (or banks), respectively. Note that the hydraulic radius is not additive with this formulation, as it was with R' and R''. Using Manning's equation, with a known average velocity, slope, and roughness coefficient, the hydraulic radius associated with the banks can be calculated:

$$\frac{V}{CME\, S^{\frac{1}{2}}} = \frac{R^{\frac{2}{3}}}{n} = \frac{R_w^{\frac{2}{3}}}{n_w} \qquad \text{(eq. 8–11)}$$

$$R_w = \left(n_w \frac{V}{CME\, S^{\frac{1}{2}}} \right)^{\frac{3}{2}} \qquad \text{(eq. 8–12)}$$

where:
CME =1.486 in English units and 1.0 in SI units

Total hydraulic radius and shear stress, considering grain, form, and bank roughness, can be expressed by equations 8–13 and 8–14:

$$R_{total} = \frac{P_b (R' + R'') + P_w R_w}{P_{total}} \qquad \text{(eq. 8–13)}$$

$$\tau_{total} = \gamma S \left(\frac{P_b (R' + R'') + P_w R_w}{P_{total}} \right) \qquad \text{(eq. 8–14)}$$

Lane's tractive force method

Lane (1952) developed an analytical design approach for calculation of the applied grain shear stress and the shear distribution in trapezoidal channels. The tractive force, or applied shear force, is the force that the water exerts on the wetted perimeter of a channel

due to the motion of the water. Lane determined that in most irrigation canals, the tractive force near the middle of the channel closely approaches

$$\gamma d S_o$$

where:
γ = specific weight of water
d = depth
S_o = bed slope assuming uniform flow

He also determined that the maximum tractive force on the side slopes was approximately 0.75 $\gamma d S_o$. Lane also found that the side slopes of the channel affected the maximum allowable shear stress. He developed an adjustment factor, K, to account for the side slope effects. Detailed information on the tractive force approach is found in Design of Open Channels, TR–25 (USDA SCS 1977) and Chow (1959). A summary of the method follows.

When the boundary of the channel consists of coarse-grained discrete particles, Lane (1952) determined that the grain roughness, n_s, could be determined as a function of the D_{75} of the boundary material. Applied grain shear stress can then be calculated using Manning's equation. The D_{75} range for which Lane found this relationship to be applicable was between 0.25 inches (6.35 mm) and 5.0 inches (127 mm). This is similar to determining the grain shear stress using the Limerinos equation.

$$n_s = \frac{D_{75}^{\frac{1}{6}}}{39} \text{ with } D_{75} \text{ expressed in inches} \qquad \text{(eq. 8–15)}$$

$$n_s = \frac{D_{75}^{\frac{1}{6}}}{66.9} \text{ with } D_{75} \text{ expressed in millimeters} \qquad \text{(eq. 8–16)}$$

The grain roughness is combined with other roughness elements to determine the total Manning's roughness coefficient, n. The friction slope associated with grain roughness, S_t, can then be calculated using equation 8–17:

$$S_t = \left(\frac{n_s}{n}\right)^2 S_e \qquad \text{(eq. 8–17)}$$

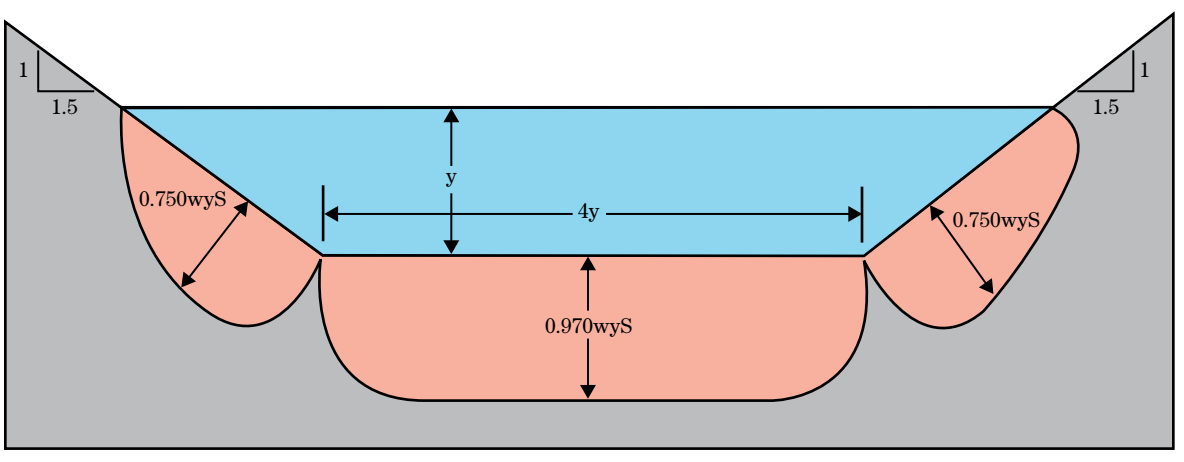
where:
S_e = total friction slope determined from Manning's equation

The applied shear stress acting on the grains in an infinitely wide channel is then calculated from equation 8–18.

$$\tau_\infty = \gamma d S_t \qquad \text{(eq. 8–18)}$$

In open channels, the applied shear stresses are not distributed uniformly along the perimeter as is shown in figure 8–7 (Lane 1952). Laboratory experiments and field observations have indicated that in trapezoidal channels the stresses are very small near the water surface and corners of the channel. In straight chan-

Figure 8–7 Lateral distribution of shear stress in a trapezoidal channel



w=specific weight of water, y=depth, and S=slope

nels, the maximum shear stress occurs on the bed near the center of the channel. The maximum shear stress on the banks occurs about a third the way up the bank from the bed. Figures 8–5 and 8–6 can be used to determine the shear stress distribution in a trapezoidal channel, relative to the applied shear stress in an infinitely wide channel with the same depth of flow and energy slope (USDA SCS 1977).

The magnitude of applied shear stresses is not uniform in turbulent flow. Calculations using traditional equations provide an average value of shear stress. In design, therefore, a factor of safety is typically applied to account for this fluctuation. This fluctuation may also be addressed in certain design approaches using probability methods presented later in this chapter.

Applied shear stress on curved reaches

Curved channels have higher maximum shear stresses than straight channels. Maximum stress occurs on the inside bank in the upstream portion of the curve and on the outer bank in the downstream portion of the curve. The smaller the radius of curvature, the more the stress increases along the curved reach. Maximum applied shear stress in a channel with a single curve also occurs on the inside bank in the upstream portion of the curve and near the outer bank downstream from the curve. Compounding of curves in a channel complicates the flow pattern and causes a compounding of the maximum applied shear stress. Figure 8–8 gives values of maximum applied shear stress based on judgment coupled with very limited experimental data (USDA SCS 1977). It does not show the effect of depth of flow and length of curve, and its use is only justified until more accurate information is obtained. Figure 8–9, with a similar degree of accuracy, gives the maximum applied shear stresses at various distances downstream from the curve (USDA SCS 1977). The designer should note that these adjustments are similar to rules of thumb.

(b) Calculate allowable shear stress

The applied shear stress must be compared to the allowable shear stress. Shear stress at initiation of motion can be calculated from an empirically derived relationship between dimensionless shear stress (Shields parameter), τ^*, and grain Reynolds number, R^*. The dimensionless shear stress is defined as the ratio of the critical shear stress (shear stress at the initiation of particle motion) and product of the grain diameter and the submerged specific weight of the particle. The grain Reynolds number is defined as the ratio of the product of shear velocity and grain diameter to kinematic viscosity. Shields parameter and grain Reynolds number are dimensionless and can be used with any consistent units of measurement. The relationship between τ^* and R^* represents an average curve drawn through scattered data points that were determined experimentally from flumes or rivers. Therefore, a wide range in recommended values exists for the Shields parameter, depending on how the experiment was conducted and the nature of the bed material being evaluated.

Once τ^* has been assigned, the critical shear stress for a particle having a diameter, D, is calculated from equation 8–19.

$$\tau_c = \tau^*(\gamma_s - \gamma)D \qquad \text{(eq. 8–19)}$$

where:

τ^* = Shields parameter, dimensionless
R^* = grain Reynolds number = u^*d/ν, dimensionless
τ_c = critical shear stress (lb/ft^2 or N/m^2)
γ_s = specific weight of sediment (lb/ft^3 or N/m^3)
γ = specific weight of water (lb/ft^3 or N/m^3)
D = particle diameter (ft or m)
u^* = shear velocity = $(gRS)^{1/2}$ (ft/s or m/s)
ν = kinematic viscosity of the fluid (ft^2/s or m^2/s)
g = acceleration of gravity (ft/s^2 or m/s^2)

Shields (1936) obtained his critical values for τ^* experimentally using uniform bed material and measuring sediment transport at decreasing levels of bed shear stress, and then extrapolating to zero transport. The Shields curve is shown in figure 8–10 (USACE 1995c). Shields' data suggest that τ^* varies with R^* until the grain Reynolds number exceeds 400. At larger values of R^*, τ^* is independent of R^* and is commonly taken to be 0.06. The Shields curve may be expressed as an equation, useful for computer programming and spreadsheet analysis.

$$\tau^* = 0.22\beta + 0.06 \times 10^{-7.7\beta} \qquad \text{(eq. 8–20)}$$

$$\beta = \left(\frac{1}{\nu}\sqrt{\left(\frac{\gamma_s - \gamma}{\gamma}\right)gD^3}\right)^{-0.6} \qquad \text{(eq. 8–21)}$$

The Shields diagram is the classic method for determining critical shear stress. However, subsequent

Figure 8–8 Applied maximum applied shear stress, τ_{bs} and τ_{sc} on bed and sides of trapezoidal channels in a curved reach

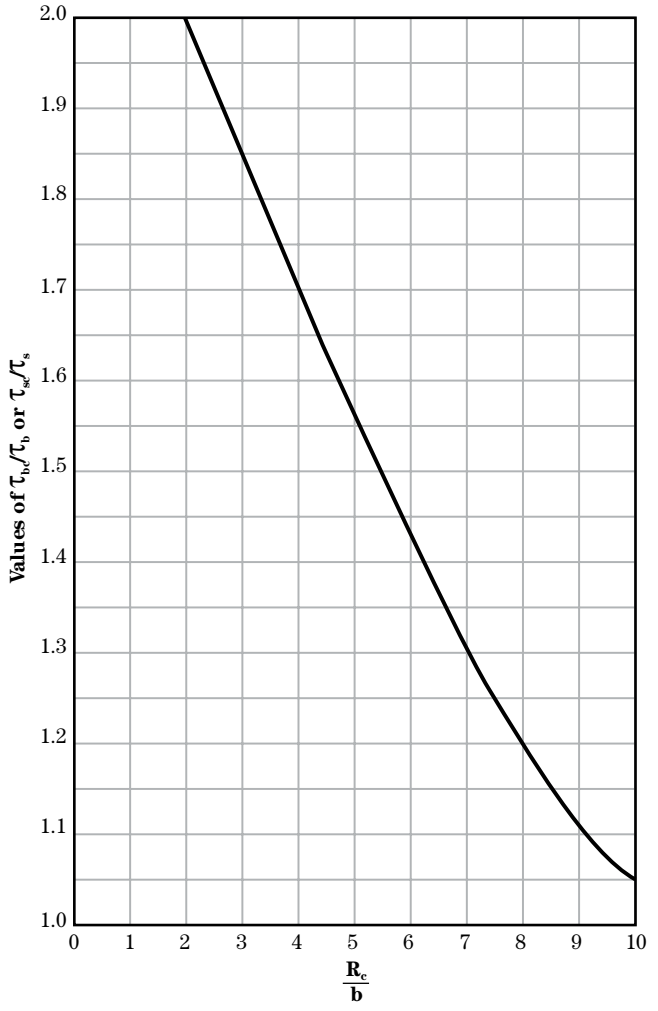
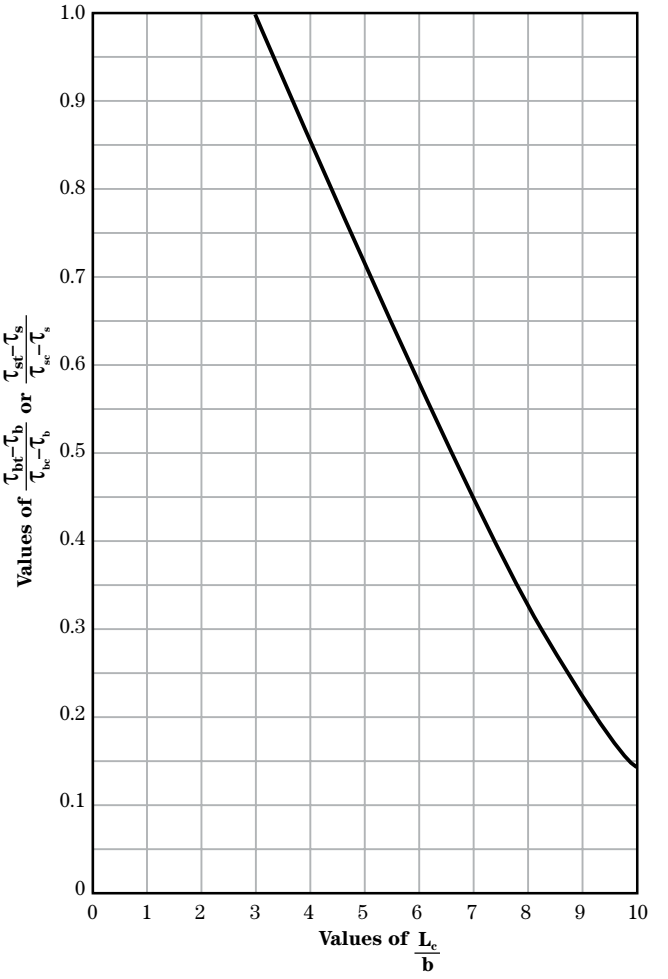


Figure 8–9 Applied maximum applied shear stress, τ_{bt} and τ_{st} on bed and sides of trapezoidal channels in straight reaches immediately downstream from curved reaches



Note:
R_c = radius of curvature
b = bottom width
d = channel depth
L_c = length of curve
τ_b = applied shear stress on a channel bed
τ_s = applied shear stress on the side of a channel
τ_{bc} = applied shear stress on channel bed in a curve
τ_{sc} = applied shear stress on channel side in a curve
τ_{bt} = applied shear stress on channel bed immediately downstream of a curve
τ_{st} = applied shear stress on channel side immediately downstream of a curve

work identified three significant problems associated with the curve itself. First, the procedure did not account for the bedforms that developed with sediment transport. Second, the critical dimensionless shear stress is based on the average sediment transport of numerous particles and does not account for the sporadic entrainment of individual particles at very low shear stresses. Thirdly, critical dimensionless shear stress for particles in a sediment mixture may be different from that for the same size particle in a uniform bed material. In general, for purposes of design of threshold channels, in which no bed movement is a requirement, the Shields curve will underestimate the critical dimensionless shear stress and is not recommended unless a factor of safety is added.

Adjustment for bedforms

Gessler (1971) determined that Shields did not separate grain shear stress from bedform shear stress in his experimental flume data analysis. Bedforms developed with sediment transport for the fine-grained bed material in some of Shields flume data. Since a portion of the total applied shear stress is required to overcome the bedform roughness, the calculated dimensionless shear stress would be too high for a natural bed with no bedforms. Gessler reanalyzed Shields' data so that the critical Shields parameter represented only the grain shear stress (fig. 8–11). This curve is more appropriate for determining critical shear stress in plane bed streams with relatively uniform bed gradations. With fully turbulent flow (R* >400), typical of gravel-bed streams, τ* is commonly taken to be 0.047 using Gessler's curve.

Figure 8–11 Gessler's reformulation of Shields diagram. τ is critical grain shear stress and k is grain diameter.

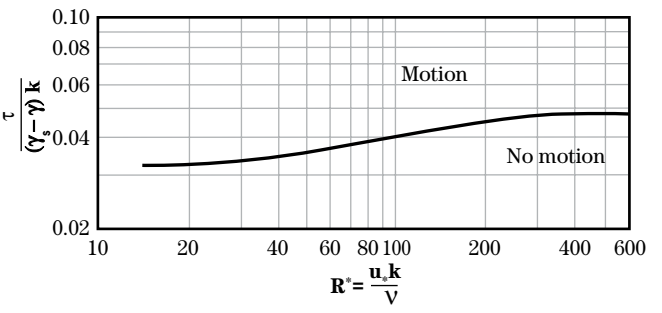
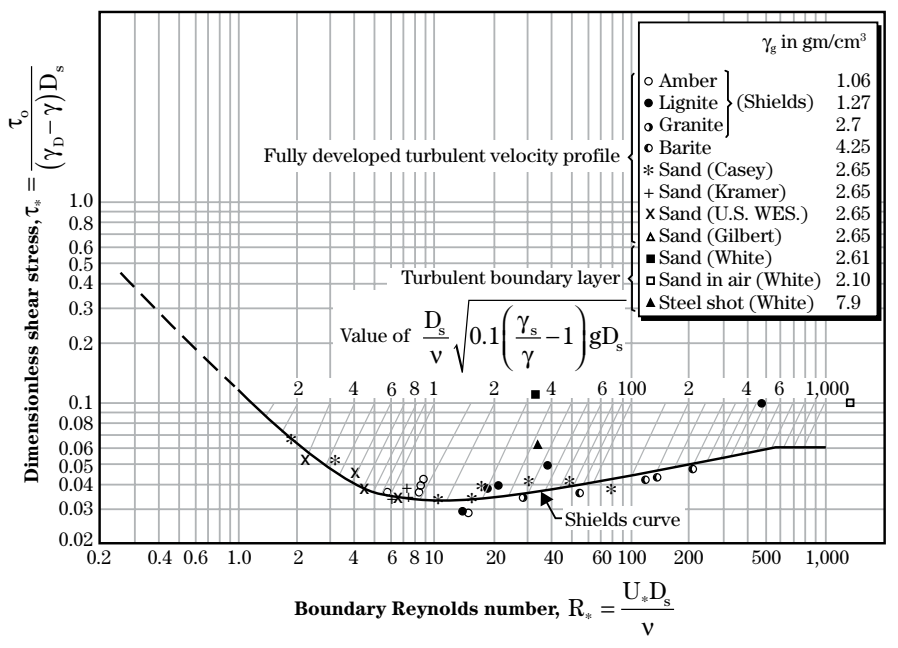


Figure 8–10 Shields curve



Adjustment for mixtures

Natural streambeds seldom have uniform bed gradations. The critical bed shear stress equation must be modified for mixtures. There are two approaches: one is to select a τ^* that is characteristic of mixtures; the other is to select a percent finer grain size that is characteristic of initiation of motion. Meyer-Peter and Muller (1948) and Gessler (1971) determined that when $R^* > 400$, the critical Shields parameter for sediment mixtures was about 0.047 when median grain size was used. Neill (1968) determined from his data that in gravel mixtures, most particles became mobile when τ^* was 0.030, when median grain size was used for D. Andrews (1983) found a slight difference in τ^* for different grain sizes in a mixture, and presented the equation 8–22:

$$\tau_i^* = 0.0834\left(\frac{D_i}{D_{50}}\right)^{-0.872} \qquad \text{(eq. 8–22)}$$

where:

subscript, i =Shields parameter and grain size for size class i

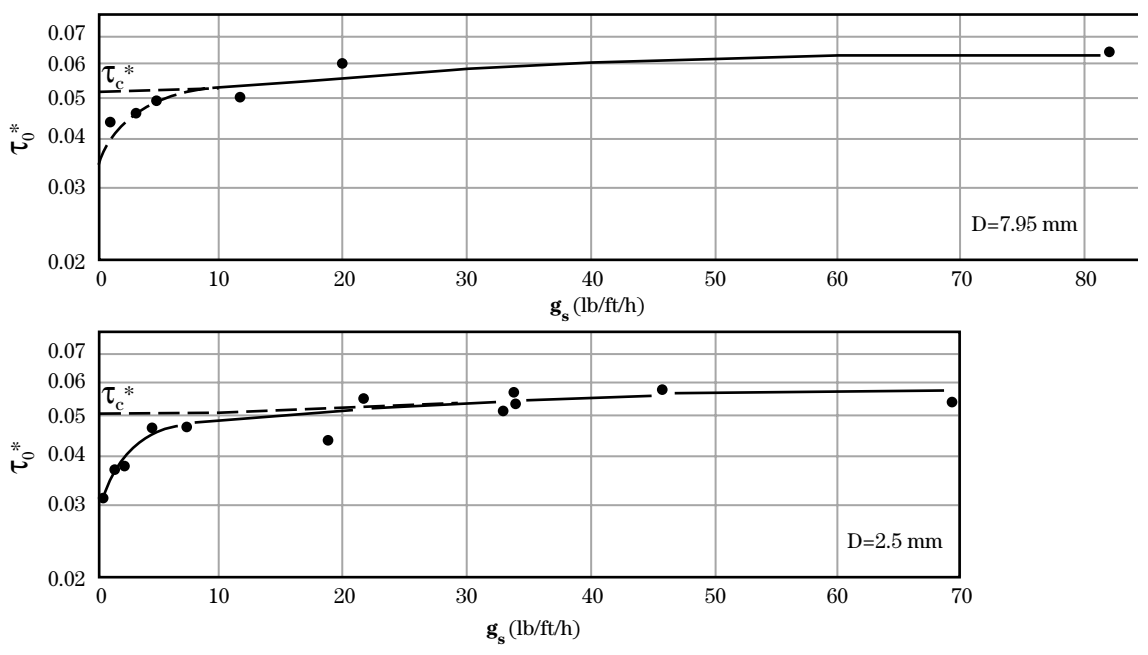
D_{50} =median diameter of the subsurface material

The minimum value for τ^* was found to be 0.020. According to Andrews, the critical shear stress for individual particles has a very small range; therefore, the entire bed becomes mobilized at nearly the same shear stress. However, Wilcock (1998) and Wilcock and McArdell (1993) have demonstrated that this near-equal mobility result applies only to unimodal sediments with a small to modest standard deviation. In coarse beds with a wide range of sizes (especially mixtures of sand and gravel), the fines may begin to move at flows much smaller than the coarse grains.

Gessler's concept for particle stability

Critical shear stress is difficult to define because entrainment is sporadic at low shear stresses caused by bursts of turbulence. Due to the difficulty in defining initiation of motion in a flume, the Shields curve was developed by extrapolating measured sediment transport rates back to zero. Unfortunately, the relationship between the Shields parameter and sediment transport is not linear at low shear stresses. This phenomenon was demonstrated by Paintal (1971) (fig. 8–12). Note that the extrapolated critical dimensionless shear stress was about 0.05, but the actual critical dimensionless shear stress was 0.03.

Figure 8–12 Variation in Shields parameter with decreasing sediment load



Gessler (1971) developed a probability approach to the initiation of motion for sediment mixtures. He reasoned that due to the random orientation of grains and the random strength of turbulence on the bed, for a given set of hydraulic conditions, part of the grains of a given size will move, while others of the same size may remain in place. Gessler assumed that the critical Shields parameter represents an average condition, where about half the grains of a uniform material will remain stable and half will move. It follows then that when the critical shear stress was equal to the bed shear stress, there was a 50 percent chance for a given particle to move. Using experimental flume data, he developed a probability function, p, dependent on τ_c/τ where τ_c varied with bed size class (fig. 8–13). He determined that the probability function had a normal distribution, and that the standard deviation (slope of the probability curve) was a function primarily of turbulence intensity, and equal to 0.057. Gessler found the effect of grain-size orientation to be negligible. The standard deviation also accounts for hiding effects; that is, no attempt was made to separate hiding from the overall process. Gessler's analysis demonstrates that there can be entrainment of particles, even when the applied shear stress is less than the critical shear stress; and that not all particles of a given size class on the bed will necessarily be entrained, until the applied shear stress exceeds the critical shear stress by a factor of 2. The design implications of this work are:

- If near-complete immobility is desired in the project design, the Shields parameter used to determine critical shear stress should be on the order of half the typically assigned value.
- To assure complete mobility of the bed (fully alluvial conditions), the applied grain shear stress should be twice the critical shear stress.

The inherent dangers of using 50 percent or 200 percent of critical shear stress are that the channel could aggrade or incise.

Gessler used the probability approach to determine if the bed surface layer of a channel was stable (immobile). He suggested that the mean value of the probabilities for the bed surface to stay in place should be a good indicator of stability:

$$\bar{P} = \frac{\int_{i_{min}}^{i_{max}} P^2 f_i D_i}{\int_{i_{min}}^{i_{max}} P f_i D_i} \qquad \text{(eq. 8–23)}$$

where:

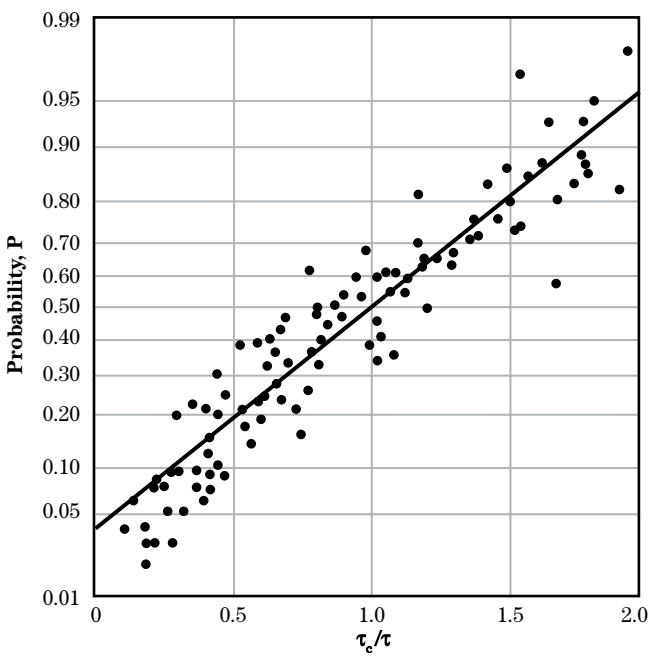
$\bar{P}$ = probability function for the mixture (depends on the frequency of all grain sizes in the underlying material)

f_i = fraction of grain size i

If the gradation of the channel bed is known, τ_c for each size class is determined from figure 8–11, and P for each size class is determined from figure 8–13. $\bar{P}$ can then be calculated from equation 8–23. Gessler suggested that when $\bar{P}$ was less than 0.65, the bed was unstable.

The probability concept was presented in an empirical fashion by Buffington and Montgomery (1997). They analyzed critical shear stress data from many investigators and suggested ranges for the critical Shields parameter. For visually base data, where initiation of motion was determined by investigator observation, Buffington and Montgomery suggested a range for τ^* between 0.073 and 0.030 for fully rough, turbulent flow ($R^* > 400$). They concluded that less emphasis should be placed on choosing a universal value for τ^*, while more emphasis should be placed on choosing defendable values for particular applications. Buffington and Montgomery also provided the compiled data

Figure 8–13 Probability of grains to stay on the bed



from many investigators, including data from natural streams.

Lane's method for coarse grained soils

Lane (1955a) concentrated on the force exerted over a given surface area of the channel, rather than the force exerted on a single particle, as in the Shields parameter and Gessler approaches. He also built in a factor of safety to the critical shear stress, so that his equation more appropriately can be called an allowable shear stress equation. This factor of safety accounts for the shear stress fluctuations in turbulent flow.

For boundaries with coarse-grained discrete soil particles, where the D_{75} is between 0.25 and 5.00 inches (6.35 and 127 mm), the allowable shear stress on the channel bottom, τ_{ab}, can be approximated using equation 8–24 proposed by Lane.

$$\tau_{ab} = 0.4\,D_{75} \qquad \text{(eq. 8–24)}$$

where:
D_{75} = particle size for which 75% of the sediment is smaller (in)
τ_{ab} = allowable shear stress on channel bottom (lb/ft^2)

The allowable shear stress for the channel sides, τ_{as}, is less than that of the same material in the bed of the channel because the gravity force aids the applied shear stress in moving the materials. For channel sides composed of soil particles behaving as discrete single grain materials, considering the effect of the side slope, z, and the angle of repose, ϕ, with the horizontal, the allowable shear stress is:

$$\tau_{as} = 0.4K\,D_{75} \qquad \text{(eq. 8–25)}$$

where:

$$K = \sqrt{\frac{z^2 - \cot^2\phi}{1 + z^2}} \qquad \text{(eq. 8–26)}$$

The angle of repose for various degrees of particle angularity can be determined from figure 8–14 (Lane 1952). When the unit weight, γ_s, of the boundary material greater than D_{75} is significantly different from 160 pounds per cubic foot, the allowable shear stresses, τ_{ab} and τ_{as}, should be multiplied by the factor T.

$$T = \frac{\gamma_s - \gamma}{97.8} \qquad \text{(eq. 8–27)}$$

where:
units of γ are in lb/ft^3

Figure 8–15 (from TR–25) provides adjustment values for allowable bank stress in trapezoidal channels, based on angle of repose and side slope steepness. The allowable stress for the channel sides is thought to be less than that of the same material in the bed because the gravity force adds to the stress in moving the materials.

Lane's method for fine-grained soils

Allowable shear stress in fine-grained soils (D_{75} <6.3 mm) can be determined from figure 8–16 (Lane 1955a). The curves relate the median grain size of the soils to the allowable shear stress. The curve labeled as high sediment content is to be used when the stream under consideration carries a load of 20,000 parts per million by weight or more of fine suspended sediment. The curve labeled low sediment content is to be used for streams carrying up to 2,000 parts per million by weight of fine suspended sediment. The curve labeled clear water is for flows with less than 1,000 parts per million.

When 5 millimeters <D_{50} <6.3 millimeters, use the allowable shear stress for 5 millimeters shown on the chart. When D_{50} is less than 0.1 millimeter and is still noncohesive, use the allowable shear stress for values of 0.1 millimeter.

Cohesive materials

The allowable shear stress concept has been applied to semicohesive and cohesive soils, but values do not correlate well with standard geotechnical parameters because resistance to erosion is affected by such factors as water chemistry, history of exposure to flows, and weathering. Analysis of experience with local channels and laboratory testing of local materials are generally recommended. Figure 8–17 gives an example of allowable shear stresses (tractive forces) for a range of cohesive materials, but where possible, values should be compared against the results of field observation or laboratory testing. The curves in figure 8–17 are converted from USSR (1936) permissible velocity data from straight channels with an average depth of 3 feet. The figure is reported in Chow (1959) and USACE (1991b). The basic soil textural class can be determined as a function of the percentages of clay, silt, and sand in the soil using the soil triangle in figure 8–18 (USDA SCS 1994).

Figure 8–14 Angle of repose for noncohesive material

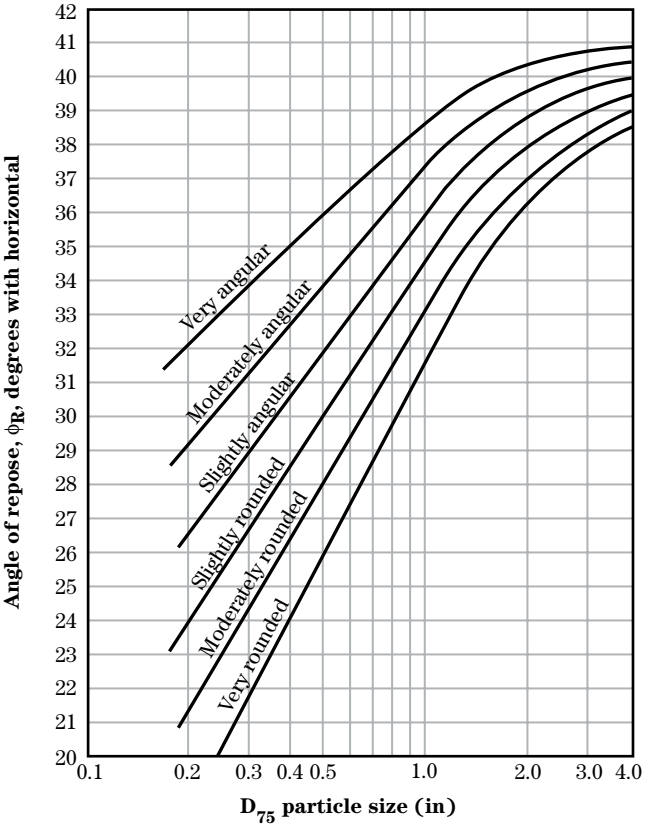


Figure 8–15 K values for allowable stress, sides of trapezoidal channels

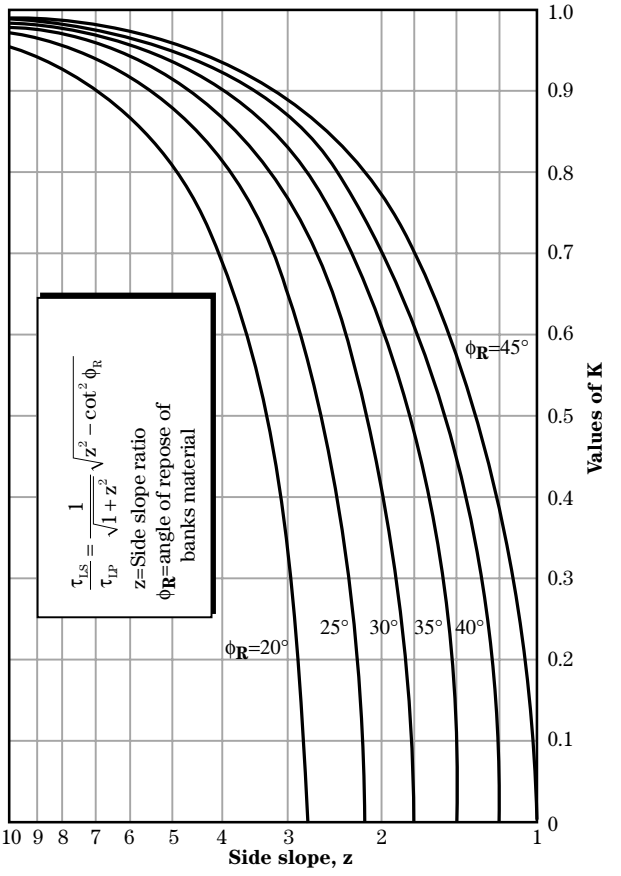


Figure 8–16 Allowable shear stress for granular material in straight trapezoidal channels

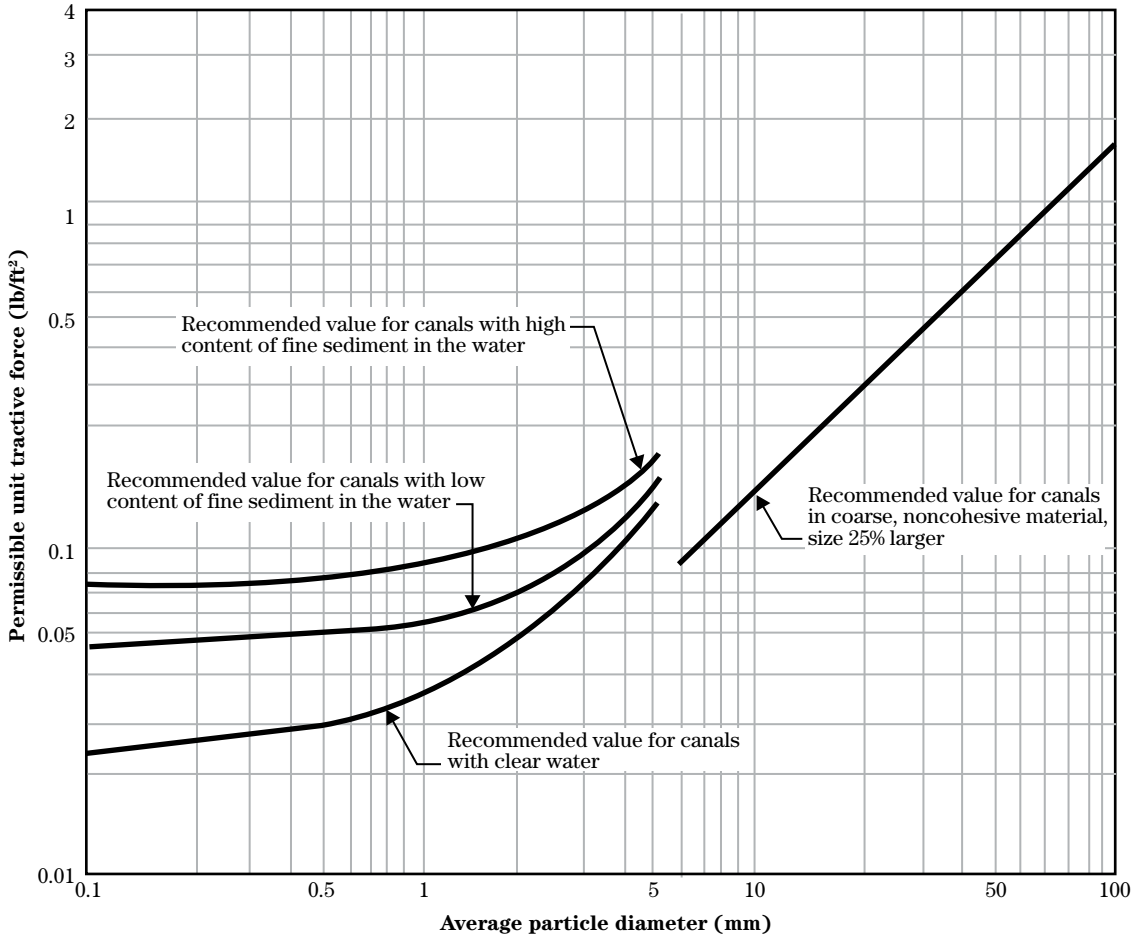


Figure 8–17 Allowable shear stress in cohesive material in straight trapezoidal channels

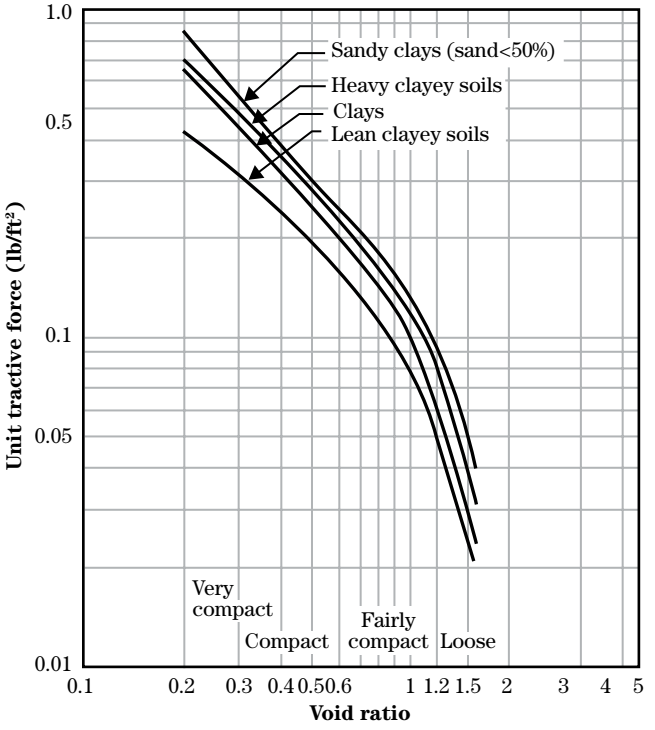
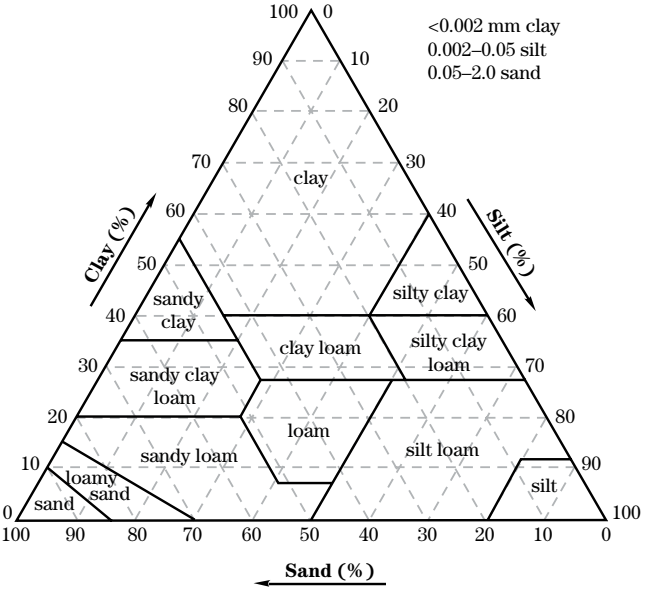


Figure 8–18 USDA textural classification chart



(c) Procedure for application of allowable shear stress method

Application of the allowable shear stress method requires first the determination of shear stress in the design channel and then comparison of the design shear stress to allowable shear stress for the boundary material. The allowable shear stress may be determined by one of three methods: Shields parameter approach, Gessler (1971) probability approach, or Lane tractive force method (Lane 1952). The characteristics of each method are summarized in table 8–5.

The use of the tractive force method to design earth channels involves the following steps modified from those found in TR–25, Design of Open Channels (USDA SCS 1977).

Step 1 Determine the hydraulics of the channel. This includes the hydrologic determinations, as well as the stage-discharge relationships for the channel being considered.

Step 2 Determine the soil properties of the bed and banks of the design reach and of the channel upstream.

Step 3 Determine the concentration of the suspended fine sediment load entering the reach. This is best accomplished by measurements. Channels with suspended fine sediment concentrations less than 1,000 parts per million are considered sediment free. Sediment-free flows are considered to have no effect on channel stability. Channels with suspended fine sediment concentrations greater than 20,000 parts per million are considered to be sediment laden. Sediment-laden flows are considered to enhance sediment stability by filling boundary interstices with cohesive material. If a significant portion of the inflowing sediment load is bed-material load, it is likely that the channel is alluvial, and threshold design methods are not applicable.

Step 4 Check to see if the allowable shear stress approach is applicable. Use table 8–1.

Step 5 Compute the applied shear stress on the boundary of the channel being studied. For noncohesive bed materials, grain shear stress can be calculated using the Limerinos equation or the Lane equation. If the Shields parameter or Gessler probability methods are used, calculate the grain shear stress using the Limerinos equation and the D_{84} of the boundary material. A factor of safety should be added to this calculated grain shear stress if the Shields parameter approach is to be used. If the tractive force method is used, calculate grain shear stress using Lane's equation with the D_{75} of the boundary material. Lane's equation already accounts for the factor of safety, so there is no need to increase the calculated applied shear stress. If the bed material is cohesive, use the total shear stress as the applied shear stress. Use (with caution) figures 8–8 and 8–9 to determine applied shear stress on the outside of bends.

Step 6 Check the ability of the soil materials forming the channel boundary to resist the computed applied shear stress. If the Shields parameter method is used, determine an appropriate Shields parameter and calculate critical shear

Table 8–5 Characteristics of methods to determine allowable shear stress

	Theoretical basis	Bed characteristics	Safety factor	Basis for coefficients
Shields parameter	The force on a single particle that initiates sediment motion	Noncohesive	No	Flumes and channels
Gessler probability	Probability distribution of force on a particle mixture	Noncohesive	Yes	Flume
Lane	Force on surface area	Separate equations and charts for coarse and fine-grained, noncohesive materials and cohesive material	Yes	Channels

stress. If the Gessler probability method is used, calculate critical shear stress for each size class in the mixture using figure 8–11. Then calculate the probability for each size class to stay in place, using figure 8–13. Finally, calculate the probability function for the bed mixture using Gessler's equation. For Lane's coarse-grained soils method, use Lane's equations with D_{75} to calculate allowable shear stress on the channel bottom and the channel slide slope. Lane's K factor for side slope allowable shear stress can also be adapted for use with the Shields parameter and Gessler probability methods. For Lane's method for fine-grained soils use figure 8–16 with the D_{50} and wash load sediment concentration to determine allowable shear stress. For cohesive materials, allowable shear stress should be determined by laboratory testing. Approximate values of allowable shear stress based on soil properties can be determined from figure 8–17.

Step 7 Compare the design shear stress with the allowable shear stress. If the allowable shear stress is greater than the design shear stress, the design is satisfactory. Otherwise, three options are available:

- Redesign the channel to reduce shear stress.
- Provide structural measures (riprap, grade control) to prevent erosion.
- Consider a mobile boundary condition and evaluate the channel using appropriate sediment transport theory and programs.

Step 8 Do a performance check to determine at what discharge the allowable shear stress is exceeded and the bed becomes alluvial.

(d) Limitations and cautions

For channels with substantial bed-material sediment load, aggradation of the design channel could be a problem in a channel designed using allowable velocity or allowable shear stress methods. A minimum velocity or shear stress must be determined that ensures sediment transport through the design reach, in addition to the allowable value. The minimum permissible velocity that prevents deposition is a function of the sediment concentration and the sediment transport capacity of the channel. Generally, for irrigation canal design, a mean velocity of 2 to 3 feet per second may be used safely, when the sediment load in the channel is small (Chow 1959).

In bends and meandering channels, bank erosion and migration may occur even if average velocities and shear stresses are well below allowable values.

An allowable velocity or shear stress analysis will not in itself define completely the channel design because it can be satisfied by a wide range of width, depth, and slope combinations. The design, therefore, must be supplemented by additional guidelines for slope, width, or cross-sectional shape. Usually, the slope will be predetermined within narrow limits, and practicable limits of width-to-depth ratio will be indicated by the existing channel.

The distinction between incipient motion and allowable velocity and shear stress must be remembered. Velocity and shear stress at incipient motion, when the particles on the bed begin to be entrained, are less than allowable velocity and shear stress used in design. Allowable values must include an allowance for the fluctuation of velocity and shear stress caused by turbulence. Channels should be designed using criteria that include some factor of safety beyond incipient motion.

It is important to remember that not all of the shear stress applied on the channel bottom is actually available to erode the channel bed. In sand channels especially, the bed is normally covered with bedforms, which dissipate some of the shear stress. Bedforms and irregularities also occur in many channels with coarser beds. Then it is necessary to use more complex approaches that involve separating the total applied shear stress into two or more parts, where only the shear stress associated with the roughness of the sediment grains must be less than the allowable shear stress.

Example: Allowable shear stress design

Given: A proposed flood channel has a bottom width of 8 feet, side slopes of 2H:1V, and energy slope of 0.00085. The channel will flow at a normal depth of 4 feet, a velocity of 3.2 foot per second, and a discharge of 200 cubic feet per second. The soils are slightly angular sandy gravels, with D_{75} of 0.75 inches. Manning's coefficient for the entire channel is estimated at 0.025. The channel has a curve with radius of curvature of 40 feet.

Problem: Check stability using allowable shear stress approach.

Solution:

Step 1 Calculate actual stresses on bed, sides, and curve.

a. Reference stress, τ_∞

$$\tau_\infty = \gamma d S_e \left(\frac{n_t}{n}\right)^2$$

$$n_t = \frac{(D_{75})^{\frac{1}{6}}}{39} = 0.0244$$

$$\tau_\infty = (62.4)(4)(0.00085)\left(\frac{0.0244}{0.025}\right)^2 = 0.2021 \text{ lb/ft}^2$$

b. Actual stress on channel bed, τ_b

$$\tau_b = \tau_\infty \left(\frac{\tau_b}{\tau_\infty}\right)$$

Using figure 8–5,

$$\frac{b}{d} = \frac{8}{4} = 2 \text{ and z=2}$$

$$\frac{\tau_b}{\tau_\infty} = 0.89$$

$$\tau_b = (0.2021)(0.89) = 0.1799$$

c. Actual stress on channel bed, curved reach, τ_{bc}

$$\tau_{bc} = \tau_b \left(\frac{\tau_{bc}}{\tau_b}\right)$$

Using figure 8–8:

$$\frac{R_c}{b} = \frac{40}{8} = 5$$

$$\frac{\tau_{bc}}{\tau_b} = 1.56$$

$$\tau_{bc} = (0.1799)(1.56) = 0.281 \text{ lb/ft}^2$$

d. Actual stress on channel sides, τ_s

$$\tau_s = \tau_\infty \left(\frac{\tau_s}{\tau_\infty}\right)$$

Using figure 8–6: $\frac{b}{d} = \frac{8}{4} = 2$ and z=2

$$\frac{\tau_s}{\tau_\infty} = 0.76$$

$$\tau_s = (0.2021)(0.76) = 0.154 \text{ lb/ft}^2$$

e. Actual stress on channel sides, curved reach, τ_{sc}

$$\tau_{sc} = \tau_s \left(\frac{\tau_{sc}}{\tau_s}\right)$$

$$\tau_s = (0.2021)(0.76) = 0.154 \text{ lb/ft}^2$$

Step 2 Calculate allowable stresses on beds and sides, τ_{Lb} and τ_{Ls}.

a. Allowable stress on bed, τ_{Lb}

$$\tau_{Lb} = 0.4\ D_{75}$$
$$= (0.4)\ (0.75) = 0.3 \text{ lb/ft}^2$$

b. $\tau_{Ls} = 0.4\ D_{75}K$

For K:

Use figure 8–14 and D_{75} = 0.75 in:

$\Phi_R = 34.3°$ for subangular

Use figure 8–15, z = 2 and $\Phi_R = 34.3°$:

K = 0.6

$\tau_{Ls} = 0.4\ (0.75)\ (0.6) = 0.18 \text{ lb/ft}^2$

Step 3 Compare actual with allowable stress for stability check.

$\tau_b = 0.1799$, $\tau_{bc} = 0.281 < \tau_{Lb} = 0.3 \text{ lb/ft}^2$ (OK)

$\tau_s = 0.154 < \tau_{Ls} = 0.18 \text{ lb/ft}^2 < \tau_{sc} = 0.24$

Therefore, for the channel to be considered to be stable, the curved reach needs a change in the hydraulics, less curvature, and/or some sort of armoring of the banks.

654.0805 Tractive power method

The tractive power method was developed by the NRCS (formally SCS) in the western United States in the 1960s to evaluate the stability of channels in cemented and partially lithified (hardened) soils. In this approach, the aggregate stability of saturated soils is assessed by use of the unconfined compression test. Field observations of several channels were evaluated against the unconfined compression strength of soil samples taken from the same channels. The results are shown in figure 8–19. Soils in channels with unconfined compression strength versus tractive power that plot above and to the left of the S-line have questionable resistance to erosion. Soils in channels with unconfined compression strength versus tractive power that plot below and to the right of the S-line can be expected to effectively resist the erosive efforts of the stream flow.

Tractive power is defined as the product of mean velocity and tractive stress. Tractive stress is calculated using the Lane method for the appropriate soil characteristics.

The use of the tractive power method to design earth channels involves the following steps modified from those found in TR–25, Design of Open Channels (USDA SCS 1977).

Step 1 Determine the hydraulics of the channel. This includes the hydrologic determinations, as well as the stage-discharge relationships for the channel being considered.

Step 2 Determine the soil properties of the bed and banks of the design reach and of the channel upstream. This includes the saturated unconfined compressive strength.

S*tep 3* Determine the concentration of the suspended sediment load entering the reach. This is best accomplished by measurements. Channels with suspended sediment concentrations less than 1,000 parts per million are considered sediment free. Sediment free flows are considered to have no effect on channel stability. Channels with suspended sediment concentrations greater than 20,000 parts per million are considered to be sediment laden. Sediment laden flows are considered to enhance sediment stability by filling boundary interstices with cohesive material. If a significant portion of the inflowing sediment load is bed-material load, it is likely that the channel is alluvial, and threshold design methods are not applicable.

Step 4 Check to see if the tractive power method is applicable. Use table 8–1.

Step 5 Compute the tractive power on the boundary of the channel being studied. Use velocity from step one. Calculate applied tractive force using the appropriate equation based on the boundary characteristics. For noncohesive bed materials, grain shear stress can be calculated using the Limerinos equation or Lane equation. If the bed material is cohesive, use the total shear stress as the applied shear stress. Use (with caution) figures 8–8 and 8–9 to determine applied shear stress on the outside of bends.

Step 6 Check the ability of the soil materials forming the channel boundary to resist the computed applied shear stress, using figure 8–19. If the combination of tractive power and unconfined compressive strength plots below the S-line, the design is satisfactory. Otherwise, three options are available:

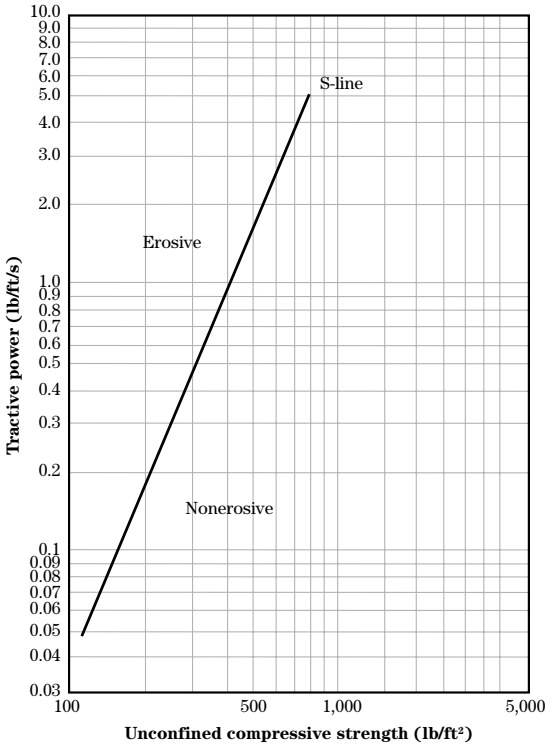
a. Redesign the channel to reduce tractive power.

b. Provide structural measures (riprap, grade control) to prevent erosion.

c. Consider a mobile boundary condition and evaluate the channel using appropriate sediment transport theory and programs.

Step 7 Do a performance check to determine at what discharge the allowable tractive power is exceeded and the bed becomes alluvial.

Figure 8–19 Unconfined strength and tractive power as related to channel stability



654.0806 Grass-lined channels

In channels where climate and soils can support permanent vegetation and baseflow does not exist, grass channel lining may be used to provide protection to erodible soil boundaries. Grass linings have been widely used to protect agricultural waterways, floodways, urban drainageways, and reservoir auxiliary spillways. The material in this section is derived from USDA Agricultural Handbook (AH) 667 (Temple et al. 1987), which has extended the concepts of SCS TP–61 (USDA SCS 1954).

(a) Allowable velocity

The method follows a similar format to the allowable or permissible velocity method described earlier. However, there are some important differences in how the allowable velocity is calculated. The allowable velocity is defined as the velocity that can be sustained for a reasonable length of time. Recommended allowable velocities for different vegetal covers, channel slopes, and soil conditions are shown in table 8–6.

Table 8–6 Allowable velocities for channels lined with grass

Cover	Slope range percent	Allowable velocity (ft/s)	
		Erosion-resistant soils	Easily eroded soils
Bermudagrass	0–5	8	6
	5–10	7	5
	>10	6	4
Buffalograss, Kentucky bluegrass, smooth brome, blue grama	0–5	7	5
	5–10	6	4
	>10	5	3
Grass mixture	0–5	5	4
	5–10	4	3
	Not recommended on slopes greater than 10%		
Lespedeza sericea, weeping lovegrass, ischaemum (yellow bluestem), kudzu, alfalfa, crabgrass	0–5	3.5	2.5
	Not recommended on slopes greater than 5%, except for side slopes in a compound channel		
Annuals—used on mild slopes or as temporary protection until permanent covers are established, common lespedeza, Sudangrass	0–5	3.5	2.5
	Not recommended for slopes greater than 5%		

Climate, soil conditions, and stability are all important factors in the selection of grass type for the channel lining. Grasses that grow in bunches, such as alfalfa, lespedeza, and kudzu, tend to concentrate flow at the bed surface. Although this characteristic may be helpful in discouraging sediment deposition, from a stability standpoint, these grasses are not suitable on steep slopes. For slopes greater than 5 percent, only fine, uniformly distributed sod-forming grasses, such as bermudagrass, Kentucky bluegrass, and smooth brome, are recommended for lining on the channel bottom. Sod-forming grasses tend to spread and may be objectionable in some cases. The upper side slope and channel berms may be planted with grasses, such as weeping lovegrass, that do not spread as easily.

Manning's roughness coefficients were also determined for the grasses tested at the USDA Agricultural Research Service (ARS) Laboratory, Stillwater, Oklahoma. Roughness was determined to be a function of the grass type, product of velocity (V), and hydraulic radius (R). Maximum VR values tested were about 20 square feet per second. These roughness values should be used to calculate the average velocity for the design channel. Average curves for five degrees of flow retardance are shown in figure 8–20 (USDA SCS 1954). Descriptions of the grasses tested and their degree of retardance are given in table 8–7.

(b) Allowable shear stress

Design criteria for grass-lined channels are provided in USDA AH 667 (Temple et al. 1987). This allowable shear stress method is based on a reanalysis of available data, largely SCS TP–61 data, and a better understanding of the interaction of the flow with a vegetated boundary. The method is still semiempirical, but it improves the separation of independent variables in the design relations. Combining this method with appropriate soil erodibility relations results in an improved design procedure that is more flexible than the allowable velocity method. The allowable shear stress design method is also consistent with current nonvegetated channel design practices.

Vegetative linings can fail with increased shear stress, either by particle detachment or failure of individual vegetal elements. For soils most often encountered in practice, particle detachment begins at levels of total

Figure 8–20 Manning's roughness coefficients for grass-lined channels

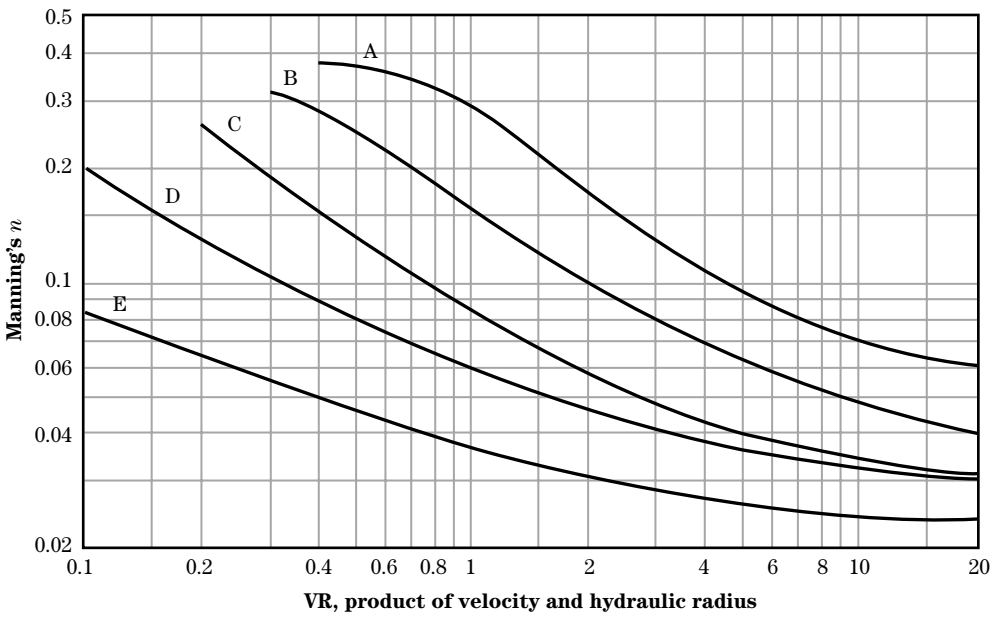


Table 8–7 Classification of degree of retardance for various kinds of grasses

Retardance	Cover	Condition
A—Very high	Weeping lovegrass	Excellent stand, tall (average 30 in)
	Yellow bluestem	Excellent stand, tall (average 36 in)
B—High	Kudzu	Very dense growth, uncut
	Bermudagrass	Good stand, tall (average 12 in)
	Native grass mixture (little bluestem, blue grama, and other long and short Midwest grasses)	Good stand, unmowed
	Weeping lovegrass	Good stand, tall (average 24 in)
	Lespedeza sericea	Good stand, not woody, tall (average 19 in)
	Alfalfa	Good stand, uncut, (average 11 in)
	Weeping lovegrass	Good stand, mowed (average 13 in)
	Kudzu	Dense growth, uncut
	Blue grama	Good stand, uncut (average 13 in)
C—Moderate	Crabgrass	Fair stand, uncut (10–48 in)
	Bermudagrass	Good stand, mowed (average 6 in)
	Common lespedeza	Good stand, uncut (average 11 in)
	Grass-legume mixture—summer (orchardgrass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (6–8 in)
	Centipede grass	Very dense cover (average 6 in)
	Kentucky bluegrass	Good stand, headed (6–12 in)
D—Low	Bermudagrass	Good stand, cut to 2.5 in
	Common lespedeza	Excellent stand, uncut, (average 4.5 in)
	Buffalo grass	Good stand, uncut (3–6 in)
	Grass-legume mixture—fall, spring (orchardgrass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (4–5 in)
	Lespedeza sericea	After cutting to 2 in
E—Very low	Bermudagrass	Good stand, cut to 1.5 in
	Bermudagrass	Burned stubble

shear stress low enough to be withstood by the vegetation without significant damage. When this occurs, the vegetation is undercut, and the weakest vegetation is removed. This leads to decreases in the density and uniformity of the remaining vegetative cover, which in turn leads to greater stresses at the boundary and a rapid failure of the protection. Failure progresses in much the same fashion in very resistant soils where the vegetal elements may sustain damage before the effective stress at the boundary becomes large enough to detach soil particles of aggregates. Damage to the vegetal cover in the form of removal of young and weak plants, shredding and tearing of leaves, and fatigue weakening of stems, results in an increase in effective stress on the boundary until conditions critical to erosion are exceeded. The ensuing erosion further weakens the cover and unraveling occurs. This characteristic of rapid unraveling of the channel lining once a weak point has developed, combined with the variability of vegetative covers, forces the design criteria presented in Agricultural Handbook (AH) 667 to be conservative. Therefore, a design factor of safety is built into the procedure.

The AH 667 procedure assumes that the allowable soil stress is the same for vegetated channels as for unlined channels, for which the tractive force is a suitable design parameter. For effective shear stress to be the sole stability parameter, detachment, rather than sediment transport processes must dominate stability considerations. This means that sediment deposition and sediment transport as bed-material load must be negligible.

(c) Species selection, establishment, and maintenance of grass-lined channels

The selection of grass species for use in channels for erosion control is based on site-specific factors:

- soil texture
- depth of the underlying material
- management requirements of vegetation
- climate
- slope
- type of structure or engineering design
- invasiveness of grass species and downstream impacts

Expected flow rate, availability of seed, ease of stand establishment, species or vegetative growth habit, plant cover, and persistence of established species are other factors that must be considered in selecting the appropriate grass to meet conditions critical to channel stability.

Chapter 2 of AH 667 (Temple et al. 1987) addresses the essential agronomic considerations in selecting, establishing, and maintaining grass channel linings.

(d) Determination of channel design parameters

The independent hydraulic variables governing the stability of a grass-lined open channel are the channel geometry and slope, erodibility of the soil boundary, and properties of the grass lining that relate to flow retardance potential and boundary protection.

Stability design of a grass-lined open channel using the effective stress approach requires the determination of two vegetal parameters. The first is the retardance curve index, C_I, which describes the potential of the vegetal cover to develop flow resistance. The second is the vegetal cover index, C_F, which describes the degree to which the vegetal cover prevents high velocities and stresses at the soil-water interface.

The retardance curve index can be determined from the dimensionless equation (eq. 8–28) where any consistent units of measurement can be used.

$$C_I = 2.5\left(h\sqrt{M}\right)^{\frac{1}{3}} \qquad \text{(eq. 8–28)}$$

where:
h = the representative stem length
M = the stem density in stems per unit area

The stem length will usually need to be estimated directly from knowledge of the vegetal conditions at the time of anticipated maximum flow. Table 8–8 may be used as a guide for the grass species most commonly encountered (Temple et al. 1987). When two or more grasses with widely differing growth characteristics are involved, the representative stem length is determined as the root mean square of the individual stem lengths. The reference stem densities contained in table 8–9 may be used as a guide in estimating M when more direct information is unavailable. The values in

Table 8–8 Characteristics of selected grass species for use in channels and waterways

Grass species	Height at maturity (ft)	Height at maturity (m)
Cool-season grasses		
Creeping foxtail	3–4	0.9–1.2
Crested wheatgrass	2–3	0.6–0.9
Green needlegrass	3–4	0.9–1.2
Russian wild rye	3–4	0.9–1.2
Smooth bromegrass	3–4	0.9–1.2
Tall fescue	3–4	0.9–1.2
Tall wheatgrass		1.2–1.5
Western wheatgrass	2–3	0.6–0.9
Warm-season grasses		
Bermudagrass	3/4–2	0.2–0.6
Big bluestem	4–6	1.2–1.8
Blue grama	1–2	0.3–0.6
Buffalograss	1/3–1	0.1–0.3
Green spangletop	3–4	0.9–1.2
Indiangrass	5–6	1.5–1.8
Klein grass	3–4	0.9–1.2
Little bluestem	3–4	0.9–1.2
Plains bristlegrass	1–2	0.3–0.6
Sand bluestem	5–6	1.5–1.8
Sideoats grama	2–3	0.6–0.9
Switchgrass	4–5	1.2–1.5
Vine mesquitegrass	1–2	0.3–0.6
Weeping lovegrass	3–4	0.9–1.2
Old World bluestems		
Caucasian bluestem	4–5	1.2–1.5
Ganada yellow bluestem	3–4	0.9–1.2

Table 8–9 Retardance curve index by SCS retardance class

SCS retardance class	Retardance curve index
A	10.0
B	7.64
C	5.60
D	4.44
E	2.88

this table were obtained from a review of the available qualitative descriptions and stem counts reported by researchers studying channel resistance and stability.

Since cover conditions vary from year to year and season to season, it is recommended that an upper and lower bound be determined for C_I. The lower bound should be used in stability computations, and the upper bound should be used to determine channel capacity. Some practitioners find that the use of SCS retardance class (table 8–9) is a preferable approach.

The vegetal cover index, C_F, depends primarily on the density and uniformity of density in the immediate vicinity of the soil boundary. Because this parameter is associated with the prevention of local erosion damage which may lead to channel unraveling, the cover factor should represent the weakest area in a reach, rather than the average for the cover species. Recommended values for the cover factor are presented in table 8–10. Values in this table do not account for such considerations as maintenance practices or uniformity of soil fertility or moisture. Therefore, appropriate engineering judgment should be used in its application.

Table 8–10 Properties of grass channel linings values (apply to good uniform stands of each cover)

Cover factor (C_F)	Covers tested	Reference stem density (stems/ft^2)	Reference stem density (stems/m^2)
0.90	Bermudagrass	500	5,380
	Centipede grass	500	5,380
0.87	Buffalograss	400	4,300
	Kentucky bluegrass	350	3,770
	Blue grama	350	3,770
0.75	Grass mixture	200	2,150
0.50	Weeping lovegrass	350	3,770
	Yellow bluestem	250	2,690
0.50	Alfalfa	500	5,380
	Lespedeza sericea	300	3,280
0.50	Common lespedeza	150	1,610
	Sudangrass	50	538

Multiply the stem densities given by 1/3, 2/3, 1, 4/3, and 5/3 for poor, fair, good, very good, and excellent covers, respectively. Reduce the C_F by 20% for fair stands and 50% for poor stands.

Two soil parameters are required for application of effective stress concepts to the stability design of lined or unlined channels having an erodible soil boundary: soil grain roughness, n_s, and allowable effective stress, τ_a. When the effective stress approach is used, the soil parameters are the same for both lined and unlined channels with negligible bed-material sediment transport.

Soil grain roughness is defined as the roughness associated with particles or aggregates of a size that can be independently moved by the flow at incipient channel failure. For noncohesive soils, the soil grain roughness and effective shear stress are both a function of the D_{75} grain size. When D_{75} is greater than 1.3 millimeter, the soil is considered coarse grained. When D_{75} is less than 1.3 millimeter, the soil is considered fine grained. Fine-grained roughness is considered to have a constant value of 0.0156. Fine-grained effective shear stress is taken to have a constant value of 0.02 pound per square foot. Coarse-grained shear stress and roughness are given in figures 8–21 and 8–22.

A soil grain roughness of 0.0156 is assigned to all cohesive soils. The allowable effective stresses are a function of the unified soil classification system soil type, the plasticity index, and the void ratio. The basic allowable shear stress, τ_{ab}, is determined from the plasticity index and soil classification, and then adjusted by the void ratio correction factor, C_e, using the following equation:

$$\tau_a = \tau_{ab} C_e^2 \qquad \text{(eq. 8–29)}$$

The basic allowable effective stress can be determined from figure 8–23 and the void ratio correction factor from figure 8–24. These two figures were developed directly from the allowable velocity curves in AH 667. Stress partitioning (slope partitioning) is essential to application of figures 8–21 to 8–24, with or without vegetation (Temple et al. 1987).

(e) General design procedure

Use the basic shear stress equation to determine effective shear stress on the soil beneath the vegetation. Use any consistent units of measurement.

$$\tau_e = \gamma d S \left(1 - C_F\right)\left(\frac{n_s}{n}\right)^2 \qquad \text{(eq. 8–30)}$$

where:

τ_e = effective shear stress exerted on the soil beneath vegetation (lb/ft^2 or N/m^2)
γ = specific weight of water (lb/ft^3 or N/m^3)
d = maximum depth of flow in the cross section (ft or m)
S = energy slope, dimensionless
C_F = vegetation cover factor (0 for unlined channel), dimensionless
n_s = grain roughness of underlying soil, typically taken as dimensionless
n = roughness coefficient of vegetation, typically taken as dimensionless

The flow depth is used instead of the hydraulic radius because this will result in the maximum local shear stress, rather than the average shear stress. The cover factor is a function of the grass and stem density. Roughness coefficients are standard Manning's roughness values; n_s can be determined from figure 8–22, n can be determined from the old SCS curves (fig. 8–20) or from the following equation.

$$n_R = \exp\left\{C_I\left[0.0133\left(\ln R_v\right)^2 - 0.0954 \ln R_v + 0.297\right] - 4.16\right\} \qquad \text{(eq. 8–31)}$$

where:

R_v = (VR/ν) x 10^{-5} (this dimensionless term reduces to VR for practical application in English units)
V = channel velocity (ft/s or m/s)
R = hydraulic radius (ft or m)

Limited to $0.0025 C_I^{2.5} < R_v < 36$

A reference value of Manning's resistance coefficient, n_R, is applicable to vegetation established on relatively smoothly graded fine-grained soil.

If vegetated channel liner mats are used, manufacturer-supplied roughness coefficients for particular mats may be used in the equation.

Maximum allowable shear stress, τ_{va}, in pound per square foot is determined as a function of the retardance curve index, C_I. Very little information is available for vegetal performance under very high stresses and this relation is believed to be conservative.

$$\tau_{va} = 0.75 C_I \qquad \text{(eq. 8–32)}$$

Figure 8–21 Allowable shear stress for noncohesive soils

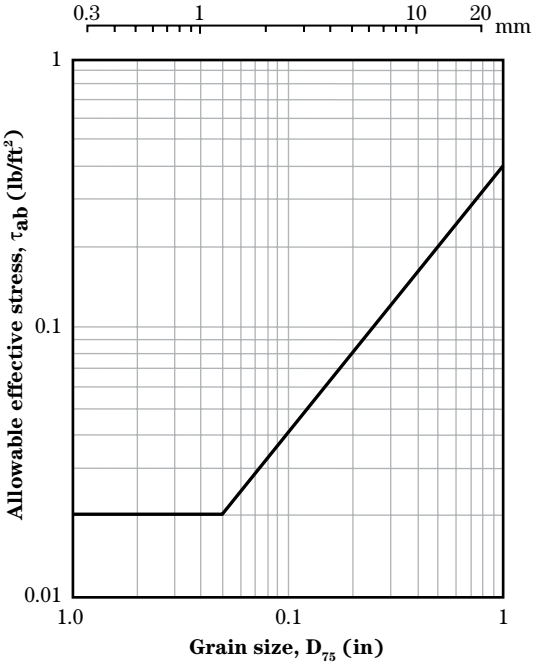


Figure 8–22 Soil grain roughness for noncohesive soils

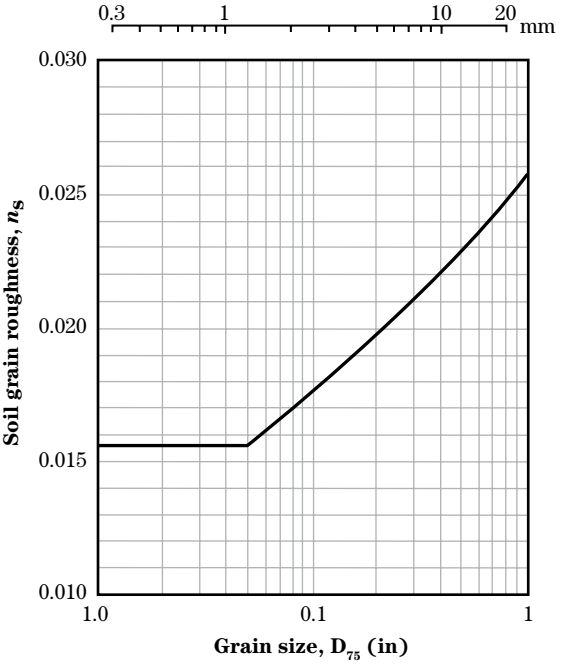


Figure 8–23 Basic allowable shear stress for cohesive soils

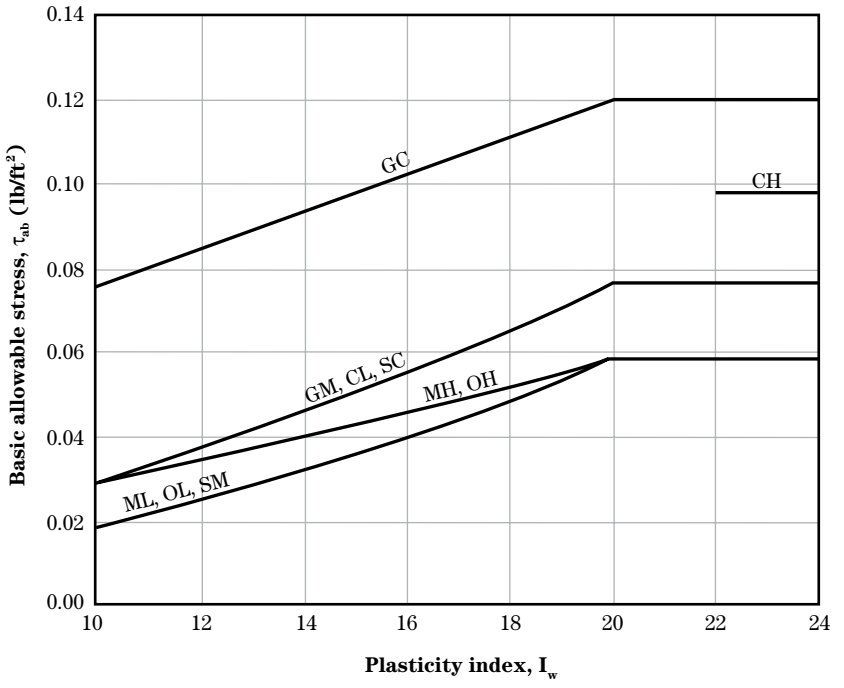
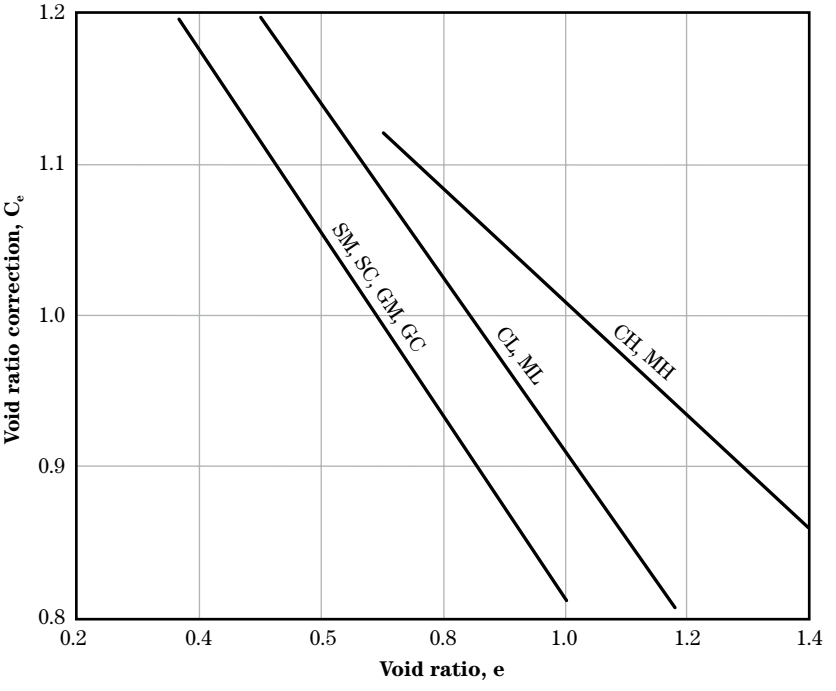


Figure 8–24 Void ratio correction factor for cohesive soils



Example problem: Threshold channel design of a grass-lined channel

Given: A vegetated floodway is to be constructed to bypass flood flows around an urban area. HEC–RAS computer program has been used to analyze the hydraulics of a preliminary design. The proposed floodway has a trapezoidal shape with bottom width of 50 feet and side slopes of 3H:1V. The floodway n value is 0.03. The floodway will have straight and curved reaches with radii of curvature equal to 300 feet. Energy slopes range from 0.00026 to 0.00060, with respective maximum flow depths of 11.0 feet and 10.5 feet.

Soils laboratory test data indicate that the floodway will be excavated into a CL soil, with plasticity index greater than 20, and void ratio of 1.2. Planned vegetation is a grass mixture of brome and Kentucky bluegrasses. Vegetation is expected to be maintained at a fair stand, equivalent to a retardance class of D.

Determine: Allowable stresses and actual stresses and compare.

Solution:

Step 1 Determine allowable stresses. Note that different references subscript the symbol for stress, τ, differently. TR–25 refers to allowable stress with the symbol, τ_L; the L stands for limiting.

a. Allowable soil stress

Basic allowable soil stress,

τ_{ab} = 0.076 lb/ft^2 (fig. 8–23; CL soil and plasticity index, I_w >20)

Void ratio correction factor, $C_e = 1.48 - 0.57e$

C_e = 0.8 (fig. 8–24; void ratio, e = 1.2)

Allowable soil stress, $\tau_a = \tau_{ab} C_e^2$

$\tau_a = 0.076\ (0.8)^2 = 0.0486$ lb/ft^2

b. Allowable vegetal stress

C_I = 4.44 (table 8–9), retardance class of D)

$\tau_{va} = 0.75\ C_I$

$\tau_{va} = 0.75\ (4.44) = 3.33$ lb/ft^2

Step 2 Determine actual stresses (straight reaches). Note that TR–25 refers to actual stress with the symbol, τ, subscripted for bed, sides.

a. Actual soil stress

$$\tau_e = \gamma dS\left(1 - C_F\right)\left(\frac{n_s}{n}\right)^2$$

For minimum slope of 0.00026 and d = 11.0 ft

γ = 62.4 lb/ft^3

n = 0.0156 (note TR–25 uses the symbol n_t)

C_F = 0.75 (0.8) = 0.6 (table 8–10; reduced 20% for fair stand of grass)

n = 0.03 (for the entire channel)

$\tau_e = (62.4)\ (11.0)\ (0.00026)\ (1 - .6)(0.0156/0.03)^2$

τ_e = 0.0193 lb/ft^2 ,which is less than the allowable soil stress of 0.0486 lb/ft^2

For maximum slope of 0.0006 and d = 10.5 ft

$\tau_e = (62.4)\ (10.5)\ (0.0006)\ (1 - 0.6)\ 0.0156/0.03)^2$

τ_e = 0.0425 lb/ft^2, which is less than the allow able soil stress of 0.0486 lb/ft^2

b. Actual vegetal stress

$$\tau_v = \left(\gamma dS_e\right) - \tau_e$$

For minimum slope,

$$\tau_v = \left(62.4\right)\left(11.0\right)\left(0.00026\right) - \left(0.0193\right)$$

$\tau_v = 0.1592$ lb/ft^2

For maximum slope,

$$\tau_v = \left(62.4\right)\left(10.5\right)\left(0.0006\right) - \left(0.0425\right)$$

$\tau_v = 0.3506$ lb/ft^2

which is less than the allowable vegetal stress of 3.33 pounds per square foot.

Example problem: Threshold channel design of a grass-lined channel—Continued

Step 3 Determine actual soil stress (curved reaches), τ_{ec}.

For minimum slope of 0.00026,

$$\frac{w}{r_c} = \frac{\text{water width}}{\text{radius of curvature}}$$

$$\frac{w}{r_c} = \frac{\left[(11.0)(5)(2)+50\right]}{300} = 0.533$$

$$\frac{\tau_{ec}}{\tau_e} = 2.1$$

$$\tau_{ec} = 2.1(0.0193) = 0.0405 \text{ lb/ft}^2$$

which is less than the allowable soil stress of 0.0486 lb/ft^2

For maximum slope of 0.0006,

$$\frac{w}{r_c} = \frac{\left[(10.5)(5)(2)+50\right]}{300} = 0.517$$

$$\frac{\tau_{ec}}{\tau_e} = 2.05$$

$$\tau_{ec} = 2.05(0.0425) = 0.0870 \text{ lb/ft}^2$$

which is greater than the allowable soil stress of 0.0486 pounds per square foot. The curve sections with energy slope of 0.0006 should be considered for change of planform (less curvature or flatter energy slope) or armoring.

654.0807 Allowable velocity and shear stress for channel lining materials

Allowable velocity and allowable shear stress values for a number of different channel lining materials are presented in table 8–11. Data in the table were compiled from many sources by Fischenich (2001b). Information for specific soil bioengineering practices is provided in NEH654 TS14I. Ranges of allowable velocity and shear stress, therefore, are presented in the table. For manufactured products, the designer should consult the manufacturer's guidelines to determine thresholds for a specific product.

The values in table 8–11 relate to cross-sectional averaged values. The data typically come from flumes where the flow is uniform and does not exhibit the same level of turbulence as natural channels. The recommended values are empirically derived. The designer should consider modifying tabular values based on site-specific conditions such as duration of flow, soils, temperature, debris, ice load in the stream, and plant species, as well as channel shape and planform (Hoag and Fripp 2002). To account for some of these differences, Fischenich recommends that a factor of safety of between 1.2 and 1.3 be applied to the tabular values.

The allowable limits of velocity and shear stress published by manufacturers for various products are typically developed from studies using short durations. Studies have shown that extended flow duration reduces the erosion resistance of many types of erosion control products as shown in figure 8–25. Fischenich (2001b) recommends a factor of safety be applied when flow duration exceeds a couple of hours.

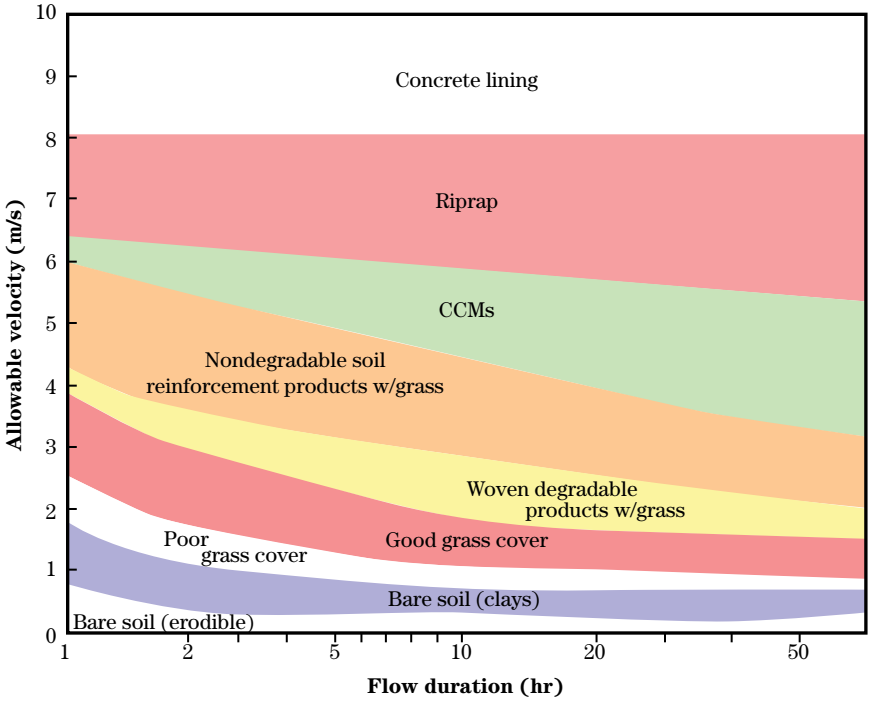
Table 8–11 Allowable velocity and shear stress for selected lining materials[1/]

Boundary category	Boundary type	Allowable velocity (ft/s)	Allowable shear stress (lb/ft^2)	Citation(s)
Temporary degradable reinforced erosion control products (RECP)	Jute net	1–2.5	0.45	B, E, F
	Straw with net	1–3	1.5–1.65	B, E, F
	Coconut fiber with net	3–4	2.25	B, F
	Fiberglass roving	2.5–7	2	B, E, F
Nondegradable RECP	Unvegetated	5–7	3	B, D, F
	Partially established	7.5–15	4–6	B, D, F
	Fully vegetated	8–21	8	C, F
Hard surface	Gabions	1–19	10	A
	Concrete	>18	12.5	E

1/ Ranges of values generally reflect multiple sources of data or different testing conditions

(Goff 1999)
(Gray and Sotir 1996)
(Julien 1995)
(Kouwen, Li, and Simons 1980)
(Norman 1975)
(TXDOT 1999)

Figure 8–25 Effect of flow duration on allowable velocities for various channel linings



*For slopes <5%

654.0808 Basic steps for threshold channel design in stream restoration projects

The following step-by-step procedure for design of natural threshold channels is from American Society of Civil Engineers (ASCE) Manual 54 (ASCE 2006). This method is applicable when width, depth, and slope are design variables; for example, slope can be varied and is not dictated by geology or other constraints. Although the procedure is presented as a series of linear steps, the actual design process is iterative, and design variables should be refined as the process proceeds from preliminary to final results. This method provides only the average channel cross-sectional dimensions. Channel variability in width and depth, and riffles and pools may be added later. Threshold methods should be used to determine the stability of the channel in areas where velocity and shear stress are increased, such as constrictions and riffles.

Step 1 Determine design bed-material gradation/channel boundary.

Determine the design bed-material gradation and the design discharge. The design discharge is the maximum flow at which channel stability is required. Channel-forming discharge theory is not generally used as the design flow for threshold channel design because the boundary of the channel will be immobile, and natural fluvial process will not be able to adjust channel dimensions.

Step 2 Determine preliminary width.

Use hydraulic geometry or regime formula (described in the NEH654.09 on alluvial channel design) with the design discharge to compute a preliminary average flow width. It is appropriate to use hydraulic geometry theory in threshold channels, even though the boundary is immobile. This is because natural flow processes will tend to form helical cells of specific widths; if the channel is too wide, ineffective flow areas will develop in the channel. If wash load is available in the stream, it may become trapped in these ineffective flow areas, and the channel will eventually narrow, even though the boundaries are immobile, and the calculated average velocity is sufficient to move the wash load.

Step 3 Estimate critical shear stress/velocity.

Using the design bed-material size gradation, estimate the critical bed stress. This may be determined using a Shields parameter approach with a factor of safety, the Gessler probability approach, or the Lane tractive force approach. If the allowable velocity approach is used, determine the allowable velocity from published tables.

Step 4 Determine flow resistance (Manning's n).

Use the bed-material size, estimated channel sinuosity, bank vegetation, and flow depth to estimate a flow resistance coefficient. The Cowan (1956) method is applicable for channels with multiple sources of roughness. If resistance due to bars and bedforms are not important, formulas such as those proposed by Limerinos (1970) or Hey (1979) may be used to compute resistance coefficients. Bathurst (1997) provides a review of flow resistance equations and their proper application.

Step 5 Calculate depth and slope.

Using the continuity equation and a uniform flow equation, compute the average depth and bed slope needed to pass the design discharge. Sinuosity may be computed by dividing the valley slope by the bed slope. Adjustment of the flow resistance coefficient for sinuosity and reiteration may be required.

Step 6 Determine planform.

Planform is a function of the sinuosity and meander wavelength. Although threshold channels are not self forming, it is appropriate to use the same techniques outlined in NEH654.09 on alluvial channels to determine planform in threshold channels.

Step 7 Assess for failure and sediment impact.

After the threshold channel design is complete, an assessment of failure should be made. This involves determination of the discharge at which the allowable velocity or shear stress would be exceeded. Confirmation should be made that the channel boundary will not become active, in which case alluvial design techniques should be examined. In addition, the possible impacts of sediment deposition should be assessed. More information on sediment impact assessments is provided in NEH654.13.

Example problem: Threshold channel design

Given:

Valley slope = 0.007 (this is the maximum possible slope)

Bed material D_{50} = 45 mm = 0.148 ft

Bed material D_{75} = 55 mm = 2.17 in

Bed material D_{84} = 60 mm = 0.197 ft

Channel side slope = 3H:1V

Specific weight of sediment = 165 lb/ft^3

Water temperature = 68 °F

Design discharge is 25-year storm = 400 ft^3/s

Problem:

Design a threshold channel to convey the design discharge.

Note: There is no unique solution with the given design constraints.

Step 1 Estimate channel width using hydraulic geometry equation (fig. 9–9, NEH654.09):

$$W = 2.03Q^{0.5}$$
$$W = 2.03(400)^{0.5}$$
$$W = 41 \text{ ft}$$

Note from figure 9–9 in NEH654.09 that widths between 22 and 74 feet are within the 90 percent single response confidence bands. If there are width constraints on the project design they may be applied here. If there are minimum depth requirements, a narrower width may be necessary. It should also be noted that the figure refers to measurements of top width. However, the difference between the top and bottom width is within the error bounds. This example will proceed with the mean width of 41 feet.

Step 2 Determine critical Shields parameter (fig. 8–10):

Initially, assume fully turbulent rough flow where grain Reynolds number >400.

$$\tau^* = 0.047$$

Step 3 Calculate critical shear stress:

$$\tau_c = \tau^*(\gamma_s - \gamma_w)D_{50}$$
$$\tau_c = (0.047)(165 \text{ lb/ft}^3 - 62.4 \text{ lb/ft}^3)(0.148 \text{ ft})$$
$$\tau_c = 0.714 \text{ lb/ft}^2$$

Step 3a. Calculate critical shear stress using the Lane equation:

$$\tau_{ab} = 0.4D_{75}$$
$$\tau_{ab} = 0.4(2.17)$$
$$\tau_{ab} = 0.868 \text{ lb/ft}^2$$

Note that the Lane equation provides a higher critical shear stress. This information will be useful in evaluating the sensitivity of the final design channel.

Step 4 Calculate depth when applied shear stress equal to critical shear stress:

$$d = \frac{\tau_c}{\gamma S}$$
$$d = \frac{(0.714 \text{ lb/ft}^2)}{(62.4 \text{ lb/ft}^2)(0.007)}$$
$$d = 1.63 \text{ ft}$$

Step 5 Calculate area and hydraulic radius for channel:

$$A = d(dz + W)$$
$$A = (1.63)[1.63(3) + 41]$$
$$A = 74.8 \text{ ft}^2$$

$$R = \frac{A}{P} = \frac{A}{\left(W + 2d\sqrt{1+z^2}\right)}$$
$$R = \frac{74.8}{\left[41 + 2(1.63)\sqrt{1+3^2}\right]}$$
$$R = 1.46 \text{ ft}$$

Example problem: Threshold channel design—Continued

Step 6 Check for fully rough flow:

$$R^* = \frac{D_{50}\sqrt{gRS}}{\upsilon}$$

$$R^* = \frac{[0.148]\left[\left(32.2\ \text{ft/s}^2\right)\left(1.46\ \text{ft}\right)\left(.007\right)\right]^{0.5}}{1.082\times10^{-5}\ \text{ft}^2/\text{s}}$$

$$R^* = 7,850$$

R* is greater than 400, therefore, fully rough flow assumption was OK.

Step 7 Calculate Manning's roughness coefficient:

Since this is a gravel-bed stream, assume no form loss and use the Limerinos equation:

$$n = \frac{0.0926R^{\frac{1}{6}}}{1.16 + 2.03\log\frac{R}{D_{84}}}$$

$$n = \frac{\left(0.0926\right)\left(1.46\right)^{\frac{1}{6}}}{\left[1.16 + 2.03\log\left(\frac{1.46}{0.197}\right)\right]}$$

$$n = \frac{\left(0.0926\right)\left(1.46\right)^{\frac{1}{6}}}{\left[1.16 + 2.03\log\left(\frac{1.46\ \text{ft}}{0.197\ \text{ft}}\right)\right]}$$

$$n = .0337$$

Step 8 Calculate velocity and discharge:

$$V = \frac{1.49}{n}R^{\frac{2}{3}}S^{\frac{1}{2}}$$

$$V = \frac{\left(1.49\right)\left(1.46\right)^{\frac{2}{3}}\left(.007\right)^{\frac{1}{2}}}{\left(.0337\right)}$$

$$V = 4.76\ \text{ft/s}$$

$$Q = VA = \left(4.76\ \text{ft/s}\right)\left(74.8\ \text{ft}^2\right)$$

$$Q = 356\ \text{ft}^3/\text{s}$$

Step 9 Modify slope until design discharge is achieved.

This iterative process can be achieved using a spreadsheet similar to the one shown in figure 8–26. The slope is decreased until the design discharge can be conveyed, without exceeding the critical shear stress. The calculated maximum slope is 0.00643. The channel planform would have a sinuosity of 1.09. The spreadsheet can also be used to evaluate the sensitivity of the solution. For example, if Gessler's criterion is applied that a stable bed should have a probability of 0.65 for the grains to stay in place, the critical shear stress is divided by 1.25 (fig. 8–13). This yields a maximum slope of 0.00473 and a sinuosity of 1.48. The solution is very sensitive to the critical shear stress. An alternative to adjusting the channel slope is to adjust the channel width between limits of the 90 percent single response confidence limits.

If movement of the bed material in this channel is a concern, select the solution where the probability of the grains on the bed to stay in place is 0.65.

Base width = 41 ft
Depth = 1.93 ft
Slope = 0.0047
Sinuosity = 1.48

As a final check, the designer should assess if the incoming sediment load can be transported through the design channel without depositing. If there is a significant incoming bed-material load, this is not a threshold channel, and alluvial channel design methods should be used. This sediment assessment is addressed in more detail in NEH654.13.

Example problem: Threshold channel design—Continued

Figure 8–26 Spreadsheet calculations for threshold channel using critical shear stress

Valley slope	**0.007**
Width	**41 ft**
Side slope	**3**
Critical shear	**0.714 lb/ft²**
D_{84}	**0.197 ft**

Iteration	Slope	Sinuosity	Max depth (ft)	Area (ft²)	R (ft)	*n*	Velocity (ft/s)	Discharge (ft³/s)
1	0.007	1.00	1.63	75.0	1.46	0.0337	4.76	357
2	0.0065	1.08	1.76	81.5	1.56	0.0334	4.84	394
3	0.0064	1.09	1.79	82.9	1.58	0.0333	4.86	403
4	0.00642	1.09	1.78	82.6	1.58	0.0334	4.86	401
5	0.00643	1.09	1.78	82.5	1.58	0.0334	4.85	400

Spreadsheet calculations for threshold channel using 0.8 times critical shear stress

Critical shear	**0.571 lb/ft²**

Iteration	Slope	Sinuosity	Max depth (ft)	Area (ft²)	R (ft)	*n*	Velocity (ft/s)	Discharge (ft³/s)
1	0.007	1.00	1.31	58.7	1.19	0.0347	4.04	237
2	0.005	1.40	1.83	85.1	1.62	0.0333	4.37	372
3	0.004	1.75	2.29	109.5	1.97	0.0325	4.56	500
4	0.0046	1.52	1.99	93.4	1.74	0.0330	4.44	415
5	0.0047	1.49	1.95	91.2	1.71	0.0330	4.42	403
6	0.00473	1.48	1.93	90.5	1.70	0.0331	4.42	400

654.0809 Conclusion

Channels cut through bedrock or coarse bed materials, grass-lined channels, and channels with cohesive beds may be designed using threshold methods. Typically, bed-material sediment transport is negligible in a threshold channel, although fine sediments that do not interchange with the bed (wash load) may be transported through the channel. The objective of the threshold channel design procedure is to ensure that the design hydraulic parameters are less than the allowable values for the channel boundary. To provide a factor of safety, allowable design variables are typically less than the critical values for the boundary material used. Average channel velocity and shear stress are the hydraulic parameters typically used for threshold channel design. As with any stream restoration, stabilization or creation, the application of design techniques should be done with caution. In many circumstances, several techniques should be examined. For channels designed using threshold assumptions and procedures, the designer must confirm that deposition or erosion will not change the boundary conditions and result in alluvial channel behavior.

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Chapter 9 Alluvial Channel Design

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Cover photo: In an alluvial channel, there is a continual exchange of the channel boundary material with the flow.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 9 Alluvial Channel Design

654.0900 Purpose

Alluvial channel design techniques are generally used for movable boundary systems and streams with beds and banks made of unconsolidated sediment particles. In an alluvial channel, there is a continual exchange of the channel boundary material with the flow. Therefore, the design of an alluvial channel as part of a restoration project requires an assessment of sediment continuity and channel performance for a range of flows. A wide variety of sources and techniques are available to the designer for designing stable alluvial channels. This chapter provides an overview and discussion of some of the most common alluvial channel design techniques. The use and application of regime, analogy, hydraulic geometry, and analytical methods are presented and described. Examples have been provided to illustrate the methods.

654.0901 Introduction

The channel geometry and flow conditions in an alluvial stream are interrelated. The river's shape and size are determined by the river itself through the processes of erosion, sediment transport, sedimentation, and resuspension. Alluvial rivers are free to adjust section, pattern, and profile in response to hydraulic changes. Alluvial streams flow through channels with bed and banks made of sediments transported by the stream under-current conditions. In alluvial streams, the independent variables that drive the hydraulic design of the channel are discharge, sediment inflow, and bed and bank-material composition. The dependent or design variables are width, depth, slope, and planform.

Alluvial channel design approaches fall into five general categories: regime, analogy, hydraulic geometry, extremal, and analytical methods. Each method has its advantages and disadvantages, depending on the stream reach being restored.

A summary of alluvial channel design methods described in this chapter is presented in table 9–1. The table summarizes the basic theory and assumptions behind each method, input requirements for using the method, and basic limitations associated with each method. Table 9–1 is a general guide, recognizing that exceptions will be encountered. Designs can become complex, especially in wood-dominated systems or when some of the necessary input data is contradictory or missing. When there is uncertainty regarding the appropriate technique, it is recommended that the designer use several of what appear to be the most appropriate techniques and look for agreement on critical design elements.

Table 9–1 Characteristics of alluvial channel hydraulic design methods

Design method	Theory and assumptions	Requirements	Limitations
Regime	Dependent channel dimensions of width, depth, and slope can be determined from regression relationships with independent variables of channel-forming discharge, bed gradation, and sediment-inflow concentration. Based on the assumption that alluvial canals will evolve to the same stable channel dimensions, given the same independent driving variables	Channel-forming discharge and inflowing sediment concentration must be estimated. Bed and bank characteristics must be determined from field evaluations	Applicability is limited to channels similar to those used to develop the regression equations. Most of the data came from irrigation canals. Froude numbers should be less than 0.3, sediment transport low, and discharge relatively uniform, similar to flow in canals
Analogy	Channel dimensions from a reference reach can be transferred to another location. Based on the assumption that alluvial streams will evolve to the same stable channel dimensions, given the same independent driving variables	Reference reach must be stable and alluvial. Reference reach must have same channel-forming discharge, valley slope, and similar bed and bank characteristics. Watershed conditions must be similar	Difficult to find a suitable reference reach, especially in developed watersheds. Dependent design variables from the reference reach must be used as a combined set
Hydraulic geometry	Dependent channel dimensions of width, depth, and slope can be determined from regression relationships with independent variables. Independent variables may include one or more of the following: channel-forming discharge, drainage area, bed gradation, bank conditions, or sediment-inflow concentration. Based on the assumption that alluvial streams will evolve to the same stable channel dimensions, given the same one or two independent driving variables	Regression curves must be developed from stable and alluvial reaches and from physiographically similar watersheds. Channel-forming discharge must be estimated. Bed and bank characteristics must be determined from field evaluations	Applicability is limited to channels similar to those used to develop the regression equations. There is a high degree of uncertainty associated with the assumptions that (1) channel dimensions can be determined by a single independent variable; and (2) and with the determination of the channel-forming discharge. Design is only for the channel-forming discharge. Modifications may be required to convey higher flows. Sediment transport is typically low

Table 9–1 Characteristics of alluvial channel hydraulic design methods—Continued

Design method	Theory and assumptions	Requirements	Limitations
Extremal hypothesis	Alluvial channels will adjust channel dimensions so that energy expenditure is minimized. Depth and sediment transport can be calculated from physically based equations including continuity, hydraulic resistance and sediment transport. Typically, these equations are based on the assumptions of fully turbulent, hydraulically rough and gradually varied flow	Channel-forming discharge and inflowing sediment concentration must be estimated. Estimates of bed-material gradation and resistance coefficients must be obtained. Appropriate hydraulic resistance and sediment transport equations must be solved simultaneously, which requires a computer program or detailed spreadsheet analysis	Support for the extremal hypothesis is divided. Many stable alluvial channels exist at conditions different from the computed extremal condition
Analytical	Depth and sediment transport can be calculated from physically based equations including continuity, hydraulic resistance, and sediment transport. Typically, based on the assumptions of fully turbulent, hydraulically rough, gradually varied flow	Channel-forming discharge and inflowing sediment concentration must be estimated. Bank characteristics must be determined from field evaluations. Estimates of bed-material gradation and resistance coefficients must be obtained. Appropriate hydraulic resistance and sediment transport equations must be solved simultaneously, which requires a computer program or detailed spreadsheet analysis	A family of solutions is obtained from the hydraulic resistance and sediment transport equations. Another method must be used to obtain the third independent variable

654.0902 Alluvial channel design variables

Alluvial channels are different from threshold channels in that the channel boundary is mobile, and sediment transport is significant. The National Engineering Handbook (NEH) 654.08 presents a basic overview of threshold channel design techniques. In an alluvial channel design, stability depends on both the channel geometry and composition of the boundary materials. Alluvial channels are capable of adjustment. Stable natural alluvial channels typically form their geometry by moving boundary material. Channel-forming discharge is typically used to determine preliminary channel dimensions, but the full range of expected discharges should be used to determine final dimensions. The hydraulic design variables of width, depth, slope, and planform are the primary dependent variables in an alluvial channel (table 9–2). Their magnitudes are determined by the independent variables of sediment inflow, water inflow, and bank composition. The downstream water surface elevation is an independent variable that could have a significant effect on the dependent variables in some cases. Boundary resistance along the channel banks and sometimes along the bed can be both dependent and/or independent, depending on local circumstances.

Design of alluvial irrigation canals has traditionally been accomplished using regime methods. Regime methods rely on regression equations that are used to determine the dependent variables. The independent variables of discharge and sediment concentration are single-valued functions and, therefore, are applicable to cases where the discharge is relatively uniform with time. Regime methods are applicable for low-energy systems with low sediment transport.

Table 9–2 Alluvial channel design variables

Dependent variables	Independent variables
Width	Sediment inflow
Depth	Water inflow
Slope	Bank composition
Planform	

The design philosophy for an alluvial channel to be designed as a natural stream, as part of a restoration project, is to employ both geomorphic principles and physically based analytical techniques to determine the design variables. Average magnitudes for width, depth, and slope are determined first. Planform and other features such as riffles, pools, and habitat enhancement structures are added later. The initial or preliminary average channel geometry is determined using a single channel-forming discharge.

Sizing the channel for the channel-forming discharge promotes channel stability. Project constraints may not allow the channel geometry to fit the dimensions suggested by the channel-forming discharge, but an effort should be made to be as close as possible to the stable channel geometry to reduce project maintenance costs. Later in the design process, a full range of discharges is used to evaluate the channel design and emulate the full range of natural discharges. The initial design, however, may need to be adjusted.

Analytical techniques are employed to ensure that the combinations of design variables are compatible. With three unknowns, three equations are required to determine the magnitude of each design variable. A hydraulic resistance equation, such as Manning's equation, can be one design equation. A sediment transport equation, such as Meyer-Peter and Müller's equation can be the second design equation. Resistance and sediment transport equations are well established and can be used with a reasonable level of confidence in the design process. One additional equation is needed. Four alternatives are considered to determine this third equation: analogy methods, hydraulic geometry relationships, constraint of one of the variables, or adopting an extremal hypothesis.

654.0903 Regime methods

Regime methods were introduced by British engineers in the late nineteenth century to design and operate extensive irrigation systems in India. These canals were excavated into fine sand-bed material and carried their design discharge within the channel. Sediment entered the canals through the canal head works. The objective of channel design was to set the channel dimensions so that the inflowing sediment load would be passed without significant scour or deposition. Channels that carried their design flow without significant degradation or aggradation were said to be in regime. Data collected from these regime channels were used to develop relationships between hydraulic and sediment variables deemed to be significant. Most regime formulations include relationships to calculate channel width, depth, and slope as functions of channel-forming (or dominant) discharge and bed-material size.

One of the major deficiencies of the regime approach is that the equations often contain empirical coefficients that must be estimated primarily using judgment and experience. Regime equations are typically regression equations. They should not be used in cases where the discharge, sediment transport, bed gradations, and channel characteristics of the project channel are significantly different from those used in the development of the regime relationships. In general, regime relationships are applicable to flows at low Froude numbers, in the ripple-dune regime, with low sediment transport, and relatively uniform discharges. Since many of these equations were developed using canals where the flows remain in the channel, they do not reflect the effects of flood plain flows in the channel formation and maintenance in a natural channel. In short, since this theory is based on steady, uniform flows in canals, a sole application of it to unsteady, nonuniform rivers is not necessarily an optimum design method, given other alternatives.

(a) Blench regime equations

Stable channel dimensions may be calculated using the Blench regime equations. These regime equations are also addressed in American Society of Civil Engineers (ASCE) Manual 54 (ASCE 2006). The data used to develop the Blench regression equations came from Indian canals with sand beds and slightly cohesive-to-cohesive banks. The sediment inflow was affected by sediment exclusion and/or ejection structures and was generally less than 30 milligrams per liter (Federal Interagency Stream Restoration Working Group (FISRWG) 1998). The equations were intended for design of canals with sand beds. The basic three channel dimensions—width, depth, and slope—are calculated as a function of bed-material grain size, channel-forming discharge, bed-material sediment concentration, and bank composition. The regression equations are not dimensionless and must be used with the units used in their derivation.

$$d = \left(\frac{F_S Q}{F_B^2}\right)^{\frac{1}{3}}$$

$$W = \left(\frac{F_B Q}{F_S}\right)^{0.5}$$

$$F_B = 1.9\sqrt{D_{50}}$$

$$S = \frac{F_B^{0.875}}{\frac{3.63g}{\upsilon^{0.25}} W^{0.25} d^{0.125}\left(1 + \frac{C}{2,330}\right)} \qquad \text{(eq. 9–1)}$$

where:

W = channel width (ft)
F_B = bed factor
F_S = side factor
Q = water discharge (ft^3/s)
D_{50} = median grain size of bed material (mm)
d = depth (ft)
S = slope
C = bed-material sediment concentration (ppm)
g = acceleration of gravity (ft/s^2)
υ = kinematic viscosity (ft^2/s)

The results are true regime values only if Q is the channel-forming discharge. However, a width, depth, and slope may be calculated for any discharge by these equations.

Blench suggested that the following values be used for the side factor:

F_S = 0.10 for friable banks
F_S = 0.20 for silty, clay, loam banks
F_S = 0.30 for tough clay banks

(b) Modified regime method

The modified regime method was introduced by Simons and Albertson (1963) and is based on data from canals in India and the United States. Simons and Albertson expanded the range of conditions used in development of previous regime equations, reducing reliance on empirical coefficients. This method is also addressed in Design of Open Channels, TR–25 (U.S. Department of Agriculture (USDA) Soil Conservation Service (SCS) (1977). Regime canals in California's Imperial Valley, the San Luis Valley in Colorado, and canals in Wyoming, Colorado, and Nebraska, were used to develop the equations. Limits of data sets used to derive the modified regime equations are given in table 9–3 (FISRWG 1998). Three sets of equations were developed for three classes of channels based on the composition of streambed and streambanks. The need for computing bed, bank, or sediment concentration factors is eliminated. Inflowing sediment concentration is not an independent variable. The equations are presented in table 9–4. These are not dimensionless equations and must be used with the units used in their derivation.

The following relationships between channel geometry and slope are applicable to all three channel types.

$$d = 1.23R \qquad (1 < R < 7)$$

$$d = 2.11 + 0.934R \qquad (7 < R < 12)$$

$$W = 0.9P$$

$$W = 0.92TW - 2.0 \qquad \text{(eq. 9–2)}$$

where:
Q = channel-forming discharge (ft^3/s)
P = perimeter (ft)
R = hydraulic radius (ft)
A = channel cross-sectional area (ft^2)
V = mean channel velocity (ft/s)
W = average channel width (ft)
d = average flow depth (ft)
TW = channel top width (ft)

According to Simons and Albertson, the channel Froude number must be less than 0.3 to avoid excessive scour.

Procedure for application of the modified regime method

Step 1 Determine the channel-forming discharge. Use methods outlined in NEH654.05. The channel-forming discharge is the primary independent variable in the modified regime equations.

Step 2 Determine the character of the bed and bank materials. Determine characteristics for both the design reach and the upstream reach. Classify the boundary materials as either sand bed and sand banks, sand bed and cohesive banks, or cohesive bed and cohesive banks. Coefficients for the modified regime equations are determined from the boundary classification.

Step 3 Calculate sediment transport rate. Select an appropriate sediment transport equation, and calculate inflow to the design reach.

Step 4 Check to see if the modified regime approach is applicable. Use table 9–1.

Step 5 Determine the channel geometry and acceptable safe slope using the modified regime equations. Use table 9–4.

Step 6 Check the slope calculated with the modified regime equations. Use Manning's equation with a realistic roughness coefficient and cross-sectional geometry consistent with that determined in step 4.

After channel dimensions have been determined, it is prudent to evaluate the sediment transport capacity of the design reach and compare it to the upstream supply reach. This can be accomplished by calculating sediment transport capacity in the two reaches, using an appropriate sediment transport equation. More detail on how to make this evaluation is given in NEH654.13, which addresses sediment budget analysis.

Table 9–3 Limits of data sets used to derive Simons and Albertson modified regime equations

Data source	Median bed-material size (mm)	Banks	Discharge (ft^3/s)	Sediment concentration (ppm)	Slope (L/L)	Bedforms
United States and Indian canals	0.318 to 0.465	Sand	100 to 400	<500	0.000135 to 0.000388	Ripple to dunes
	0.06 to 0.46	Cohesive	5 to 88,300	<500	0.000059 to 0.00034	Ripples to dunes
	Cohesive 0.029 to 0.36	Cohesive	137 to 510	<500	0.000063 to 0.000114	Ripples to dunes

Table 9–4 Coefficients for modified regime equations

Q (ft^3/s)	Sand bed and sand banks	Sand bed and cohesive banks	Cohesive bed and cohesive banks
$P\ (ft) = C_1 Q^{0.512}$	3.30	2.51	2.12
$R\ (ft) = C_2 Q^{0.361}$	0.37	0.43	0.51
$A\ (ft^2) = C_3 Q^{0.873}$	1.22	1.08	1.08
$V\ (ft/s) = C_4 (R^2 S)^{1/3}$	13.9	16.1	16.0
$W/d = C_5 Q^{0.151}$	6.5	4.3	3.0

Note: * A soil is classed as cohesive if the plasticity index is >7

Example 1: Modified regime method

Given: The channel has a sand bed and cohesive banks. The channel-forming discharge is 600 ft^3/s. Use 2H:1V side slopes and $n = 0.022$.

Problem: Design a stable trapezoidal channel using the modified regime approach.

Solution:

Step 1 Compute the channel perimeter, P:

$$P = 2.51Q^{0.512}$$
$$P = (2.51)(600)^{0.512}$$
$$P = 66.4 \text{ ft} \qquad \text{(eq. 9–3)}$$

Step 2 Compute the hydraulic radius, R:

$$R = 0.43Q^{0.361}$$
$$R = (0.43)(600)^{0.361}$$
$$R = 4.33 \text{ ft} \qquad \text{(eq. 9–4)}$$

Step 3 Compute the flow area, A:

$$A = PR$$
$$A = (66.4)(4.33)$$
$$A = 288 \text{ ft}^2 \qquad \text{(eq. 9–5)}$$

or

$$A = 1.08Q^{0.873}$$
$$A = (108)(600)^{0.873}$$
$$A = 288 \text{ ft}^2 \qquad \text{(eq. 9–6)}$$

Step 4 Compute mean velocity, V:

$$V = \frac{Q}{A}$$
$$V = \frac{(600)}{(288)}$$
$$V = 2.08 \text{ ft/s} \qquad \text{(eq. 9–7)}$$

Step 5 Compute the depth, d:

When R < 7 ft

$$d = 1.23R$$
$$d = (1.23)(4.33)$$
$$d = 5.33 \text{ ft} \qquad \text{(eq. 9–8)}$$

Step 6 Compute the Froude number:

$$F = \frac{V}{(gd)^{0.5}}$$
$$F = \frac{(2.08)}{\left[(32.2)(5.33)\right]^{0.5}}$$
$$F = 0.159$$
$$F < 0.3 \qquad \text{(eq. 9–9)}$$

therefore, design meets this requirement for stability.

Step 7 Compute bottom width, BW:

$$W = 0.9P$$
$$W = 0.9(66.4)$$
$$W = 59.8 \text{ ft}$$
$$W = 0.92TW - 2.0$$
$$59.8 = 0.92TW - 2.0$$
$$TW = 67.2 \text{ ft} \qquad \text{(eq. 9–10)}$$

For 2H: 1V side slopes

$$BW = 67.2 - (2)(2)(5.33)$$
$$BW = 45.9 \text{ ft} \qquad \text{(eq. 9–11)}$$

Step 8 Calculate the width-to-depth ratio, W/d:

$$\frac{W}{d} = 4.3Q^{0.151}$$
$$\frac{W}{d} = 4.3(600)^{0.151}$$
$$\frac{W}{d} = 11.3 \qquad \text{(eq. 9–12)}$$

Example 1: Modified regime method—Continued

Step 9 Calculate regime slope and regime hydraulic roughness coefficient:

$$V = C_4 \left(R^2 S\right)^{\frac{1}{3}} \qquad \text{(table 9-4)}$$

$$2.08 = 16.1\left[\left(4.33\right)^2 S\right]^{\frac{1}{3}}$$

$$S = 0.000115$$

$$n = 1.486 A R^{\frac{2}{3}} \frac{S^{\frac{1}{2}}}{Q} \qquad \text{(Manning's equati}$$

$$n = \left(1.486\right)\left(288\right)\left(4.33\right)^{\frac{2}{3}} \frac{\left(0.000115\right)^{\frac{1}{2}}}{\left(600\right)}$$

$$n = 0.020$$

(eq. 9–13)

Step 10 Calculate the channel slope assuming uniform flow:

$$S = \frac{\left(Qn\right)^2}{\left(2.208 A^2 R^{\frac{4}{3}}\right)}$$

$$S = \frac{\left[\left(600\right)\left(0.022\right)\right]^2}{\left[2.208\left(288\right)^2\left(4.33\right)^{\frac{4}{3}}\right]}$$

$$S = 0.000135$$

(eq. 9–14)

Step 11 Select the channel design slope:

At this point in the design process, the designer must decide if there is more confidence in the regime hydraulic roughness coefficient, 0.020, or the assigned hydraulic roughness coefficient, 0.022. If it can be demonstrated that the regime relationship for slope fits existing data in channels physiographically similar to the design channel, the engineer might choose the regime slope. Without such calibration data, there is less uncertainty related to assigning a roughness coefficient and using the slope from the uniform flow equation.

For example:

S = 0.000135 (from step 10)

d = 5.3 ft (from step 5)

BW = 45 ft (from step 7)

Step 12 After channel dimensions have been determined, calculate the sediment transport capacity of the design reach, and compare it to the upstream supply reach. If the sediment transport capacity of the supply reach is greater than the sediment transport capacity of the design channel, either sediment removal must be provided for, or the design must be modified. This step is described in more detail in NEH654.13.

654.0904 Analogy method and reference reaches

Estimates for stable channel design width, depth, and slope in an alluvial channel can be made using channel dimensions from a similar stable channel. The channel reach from which the design dimensions are taken is frequently called a reference reach. The concept is that alluvial streams will evolve to the same stable channel dimensions, given the same independent driving hydraulic variables. To apply the analogy method, the bed and bank materials, sediment inflow, slope, valley type, and annual discharge hydrograph should be close to the same in both the design and reference reaches. When these conditions exist, the reference reach is said to be physiographically similar to the design reach. All three dependent hydraulic design dimensions from the reference reach must be used in the design reach to maintain physiographic similarity. Given these constraints, it can be difficult to find a suitable reference reach, especially in urban or developed watersheds. However, while locating a suitable reference reach can be problematic, many stream restorations have been planned, measured, and designed using this approach.

A reference reach is a site that is able to transport sediments and detritus from its contributing watershed drainage area, while maintaining a consistent profile, dimension, and plan view, over time. The reference reach with the highest level of confidence would be the existing channel in the project reach or just upstream or downstream from the project reach. If the existing channel is used as a reference reach, the channel must be stable, and there should be no significant recent or future changes in the watershed. Urbanization in the watershed can significantly change both the inflow hydrograph and the sediment inflow. The analogy method is inappropriate for streams where the entire fluvial system, or a significant part of it, is in disequilibrium.

A stable historic channel can sometimes be used as a reference reach to obtain estimates for the dependent design variables of channel width and planform. This is feasible if historical width and planform information can be determined from mapping, aerial photos, and/or soil borings. However, this technique is not applicable if the watershed sediment yield and runoff characteristics have changed over time. It cannot be assumed that the historically stable channel dimensions will continue to be stable with different water and sediment inflow.

An existing pristine or pre-settlement reach may also be used as an analog or reference reach. These reaches are rare, but can sometimes be found on USDA Forest Service or National Park Service land, as well as undeveloped portions of less developed countries. However, the use of these analogs is hindered by the same issues as for the historic channels. As a result, their use is generally not feasible to use in the vast majority of stream restoration projects, unless restoration to the pristine pre-settlement condition is the project goal.

In practice, several reference reaches with relatively similar channel-forming discharges may be used to develop a range of solutions for a single dependent design variable, typically, width. Analytical methods can then be employed to determine the other dependent design variables. Other design features such as planform, riffle and pool spacing, riffle widths, and pool depths can be determined using the reference reaches. The reference reaches must be stable and alluvial. The bed and banks in both the project and reference reaches must be composed of similar sediments. There should be no significant differences in watershed hydrology, channel flows, sediment inflow, or bed-material load between the project and reference reaches.

(a) Limitations of analogy method

It can be very difficult to find a stable alluvial reference reach with characteristics physiographically similar to the reach to be restored. The independent driving variables of sediment inflow, bed and bank material, and channel-forming discharge must be similar. The dependent design variables of slope, depth, and width must be taken together as a set.

654.0905 Hydraulic geometry method

Hydraulic geometry theory is an extension of regime theory. Regime theory was developed to design canals. Hydraulic geometry was developed for analysis of natural streams and rivers. Hydraulic geometry theory is based on the concept that a river system tends to develop in a predictable way, producing an approximate equilibrium between the channel and the inflowing water and sediment (Leopold and Maddock 1953). The theory typically relates a dependent variable, such as width, to an independent or driving variable, such as discharge or drainage area. Herein lies the primary weaknesses of hydraulic geometry theory—dependent hydraulic design variables are assumed to be related only to a single independent design variable and not to any other design variables.

To help overcome this deficiency, hydraulic geometry relationships are sometimes stratified according to bed-material size, bank vegetation, or bank material type. Rosgen (1998) suggests stream classification as an appropriate tool for differentiating hydraulic geometry relationships. Hydraulic geometry relationships are developed from field observations at stable and alluvial channel cross sections and were originally used as descriptors of geomorphically adjusted channel forms. As design tools, hydraulic geometry relationships may be useful for preliminary or trial selection of the stable channel width. Hydraulic geometry relationships for depth and slope are, however, less reliable and not recommended for final channel design.

A hydraulic geometry relationship for width can be developed for a specific river, watershed, or for streams with similar physiographic characteristics. Data scatter is expected about the developed curve, even in the same river reach. An example of a hydraulic geometry relationship between bankfull discharge and bankfull water surface width developed for a mountainous watershed can be found in Emmett (1975). Emmett collected data at 39 gaging stations in the Salmon River Drainage Basin, Idaho. The relationship between bankfull discharge and bankfull width is shown in figure 9–1. Emmett's mean regression line had a regression coefficient (r^2) of 0.92. Nevertheless, a wide range of bankfull widths were found for any specific bankfull discharge. The data scatter indicates that for a bankfull discharge of 200 cubic feet per second, the bankfull width could reasonably range between 15 and 45 feet. This range does not necessarily indicate instability or different physiographic conditions, but rather the wide range of possible stable widths for a given channel-forming discharge. Some other examples of regional hydraulic geometry studies are Leopold and Maddock (1953); Dunne and Leopold (1978); Charlton, Brown, and Benson (1978); Bray (1982); and Hey and Thorne (1986). Additional guidance for application of hydraulic geometry methods is provided in FISRWG (1998). Table 9–5 provides the range of data used to derive the hydraulic geometry width predictors for gravel-bed rivers shown in table 9–6 (FISRWG 1998; Soar and Thorne 2001).

The more dissimilar the stream and watershed characteristics are in stream reaches used to develop a hydraulic geometry relationship, the greater the expected data scatter around the regression line, and the less reliable the results. It is important to recognize that this scatter represents a valid range of stable channel configurations due to variables such as geology, vegetation, land use, sediment load and gradation, runoff characteristics, and in some geographic areas, woody debris. The composition of the bank is very important in the determination of a stable channel width. The presence and percentage of cohesive sediment in the bank and/or the amount of vegetation on the bank significantly affect the stable alluvial channel width (Schumm 1977; Hey and Thorne 1986).

Figure 9–1 Hydraulic geometry relationship for width for the Upper Salmon River Basin, ID

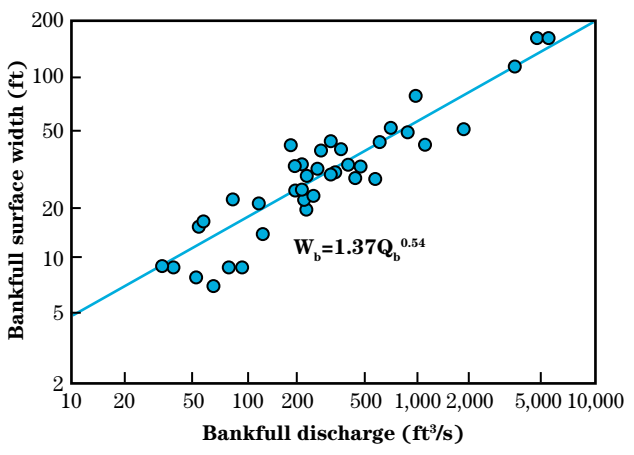


Table 9–5 Limits of data sets used to derive hydraulic geometry equation

Reference	Data source	Median bed material size (mm)	Banks	Discharge (ft^3/s)	Sediment concentration (ppm)	Slope (L/L)	Bed forms
Nixon (1959)	U.K. rivers	Gravel		700–18,000	Not measured		
Kellerhalls (1967)	U.S., Canadian, and Swiss rivers of low sinuosity, and laboratory	7–265 bed armored	Noncohesive	1.1–70,600	Negligible	0.00017–0.0131	Plane
Emmett (1975)	Salmon River, ID	11–58	Cohesive, sand and gravel	40–5,100		0.0009–0.0006	
Charlton, Brown, and Benson (1978)	Meandering U.K. rivers	33–113	Sand or gravel	95–5,500	Negligible	0.0009–0.0137	
Bray (1982)	Sinuous Canadian rivers	1.9–145		194–138,400	Mobile bed	0.00022–0.015	
Parker (1982)	British rivers Alberta–single Alberta–braided		Cohesive Little cohesion No cohesion	100–21,200 400–200,000	Active bed	0.0007–0.015 0.0002–0.015 0.0025–0.015	
Hey and Thorne (1986)	Meandering U.K. rivers	14–176	Cohesive and composite	138–15,000	Computed 0–535	0.0012–0.021	

Table 9–6 Hydraulic geometry width equations for gravel-bed rivers

$$W = aQ^b$$

Reference	Data source	Coefficient a W = m Q = m^3/s	Coefficient a W = ft Q = ft^3/s	Exponent b
Nixon (1959)	U.K.	2.99	1.65	0.5
Kellerhalls (1967)	U.S., Canada, Switzerland	3.26	1.80	0.5
Bray (1973, 1982)*	Canada	3.83	1.90	0.53
Emmett (1975)	Salmon River, ID	2.86	1.37	0.54
Charlton, Brown, and Benson (1978)	U.K.			
	Type A	3.74	2.47	0.45
	Type A_G	3.37–4.86	2.22–3.21	0.45
	Type A_T	2.62–4.11	1.73–2.71	0.45
	Type B	2.43	1.85	0.41
Parker (1982)	U.K. single channel; cohesive or vegetated banks	3.73	2.50	0.446
	Alberta single channel; little cohesion in banks	5.86	3.99	0.441
	Alberta braided; no cohesion in banks	7.08	5.25	0.417
Hey and Thorne (1986)	U.K. rivers			
	Type I	4.33	2.39	0.50
	Type II	3.33	1.84	0.50
	Type III	2.73	1.51	0.50
	Type IV	2.34	1.29	0.50

* Bankfull discharge equated to 2-year recurrence interval discharge

Note:
Type A = low sediment load
Type A_G = low sediment load and grass-lined banks
Type A_T = low sediment load and tree-lined banks
Type B = appreciable sediment load
Type I = grassy banks with no trees or shrubs
Type II = 1 to 5 percent tree/shrub cover
Type III = 5 to 50 percent tree/shrub cover
Type IV = greater than 50 percent tree-shrub cover or incised into flood plain

When a hydraulic geometry relationship is to be used for a channel design, the first choice is to use one developed from stable alluvial reaches of the project stream. It is required that the stable reaches used to develop the relationship have similar physiographic conditions to each other and the project reach. If there are no stable reaches, or if the range of discharges is insufficient, a second choice is to use other streams or tributaries in the same watershed to develop the hydraulic geometry relationship.

The third choice is to use regional relationships developed for other watersheds in the same physiographic region. The transfer of hydraulic geometry relationships developed for one watershed to another watershed should be performed with care. The two watersheds should be similar in historical land use, physiography, hydrologic regime, precipitation, and vegetation. For example, relationships developed for pristine watersheds should not be transferred to urban watersheds. Relationships developed for areas with snowmelt hydrology should not be transferred to areas dominated by convective storms. Since discharge is the variable that shapes the channel, relationships based on discharge can be transferred with more confidence than those based on drainage area, which is basically a surrogate for discharge.

Urbanized streams present particular problems in both the development and the application of hydraulic geometry relationships. Land use and runoff characteristics usually vary greatly, even within a single watershed. The multiplicity of humanmade structures, such as storm sewers, bridge openings, culverts and stormwater management facilities, and amount of impervious pavement changes the amount, duration, and timing of flows. This would be expected to greatly increase data variability. These factors make discharge more poorly correlated with drainage area and, therefore, would make discharge the better choice than drainage area as an independent variable. Locating stable, alluvial reaches in urban or developed watersheds may be difficult.

In all cases, it must be remembered that data used to develop hydraulic geometry relationships should come from stable reaches and that the watersheds and channel boundary conditions should be similar to the project channel.

(a) Procedure for developing hydraulic geometry relationships

Step 1 Locate gaging stations with long-term records. Making sure that the record is homogeneous (temporally consistent watershed conditions) as described in NEH654.05, calculate an annual peak frequency curve and a flow-duration curve. Preferably, these gaging stations will be on physiographically similar reaches of the same river as the project. A second choice is physiographically similar reaches of streams in physiographically similar watersheds. Make sure that discharge ranges are significantly greater and less than the design reach. Do not rely solely on regional relationships or drainage area versus discharge plots. These are already empirical and may not be appropriate for deriving new relationships.

Step 2 Locate stable alluvial channel reaches that can be associated with the gaging stations. Survey a typical channel cross section or several cross sections in the reach. Determine average channel top width at bankfull flow and average channel depth. Rosgen (1998) suggests gathering data from a reach length associated with two meander wavelengths or 20 top widths. Estimate channel hydraulic roughness. Using surveys or contour maps determine average channel bed slope. If the channel slope is discontinuous, use the cross sections to develop a backwater model, and calculate average energy slope. Determine bankfull discharge by using a normal depth equation or a backwater model.

Step 3 Note channel characteristics. Characteristics such as bank material composition, bed-material gradation, and bank vegetation are of interest. These characteristics may be used to ensure the physiographic similarity of the stream or to develop more refined hydraulic geometry relationships.

Step 4 Determine the channel-forming discharge. Determine the 2-year peak discharge from the annual peak frequency curve. Calculate the effective discharge using the flow-duration curve and a sediment transport curve. Determine the bankfull discharge from field measurements and backwater calculations. From these three discharges, estimate the channel-forming discharge as described in NEH654.05.

Step 5 Develop regression curve. Plot the measured channel top width versus the channel-forming discharge, and develop a power regression curve through the data. Plot confidence limits. The final plot should include the data so that the natural range of data can be observed.

(b) Generalized width predictors

Lacking data to develop more reliable hydraulic geometry relationships, generalized width predictors for various river types with different bank characteristics have been developed (Copeland et al. 2001) and are presented in figures 9–2 through 9–11. The range of data used in the development of these equations is shown in table 9–7 (Soar and Thorne 2001).

These predictors include confidence limits and may be used for general guidance when stream or watershed specific data cannot be obtained.

(c) Hydraulic geometry for meandering sand-bed rivers

Hydraulic geometry width predictors (fig. 9–2) were developed from data collected from 58 meandering sand-bed rivers in the United States (Copeland et al. 2001). These rivers were located mostly in Indiana, Illinois, Iowa, Kansas, Oklahoma, Texas, Arkansas, Louisiana, Mississippi, Kentucky, Virginia, North Carolina, and South Carolina (fig. 9–3).

Sufficient data were collected to determine both bankfull discharge and effective discharge. Data were collected from stable reaches, so bankfull discharge should be the most reliable approximator for the channel-forming discharge. In many of these meandering sand-bed rivers, the effective discharge was significantly less than the bankfull discharge. For design purposes, the bankfull discharge was used to define the width predictor. The data were divided into two sets: type T1, where there was less than 50 percent tree cover on the banks (fig. 9–4) and type T2, where there was greater than 50 percent tree cover on the banks (fig. 9–5).

Figures 9–6 and 9–7 are examples of rivers used in the development of the sand-bed hydraulic geometry relations. All sites were tree-lined to some degree; therefore, the predictors should not be used for grass-lined or thinly vegetated banks. The percentage of silt and clay in the banks was not found to be statistically significant in affecting width for these rivers, possibly because the root-binding properties of the trees were more significant in stabilizing the bank than cohesive forces.

Table 9–7 Limits of data sets used to derive generalized hydraulic geometry equations

River type	Median bed material mm	Banks	Discharge ft^3/s	Sediment concentration ppm	Slope
United States meandering sand-bed rivers	0.12–1.63	Cohesive and noncohesive	630–48,300	Significant	0.00007–0.00088
United States gravel-bed rivers	3–122	Variable	39–18,000	Negligible	0.00062–0.024
United Kingdom gravel-bed rivers	14–176	Variable	95–23,000	Negligible	0.00036–0.0021

Figure 9–2 Best-fit hydraulic geometry relationships for width for U.S. sand-bed rivers with banks typed according to density of tree cover

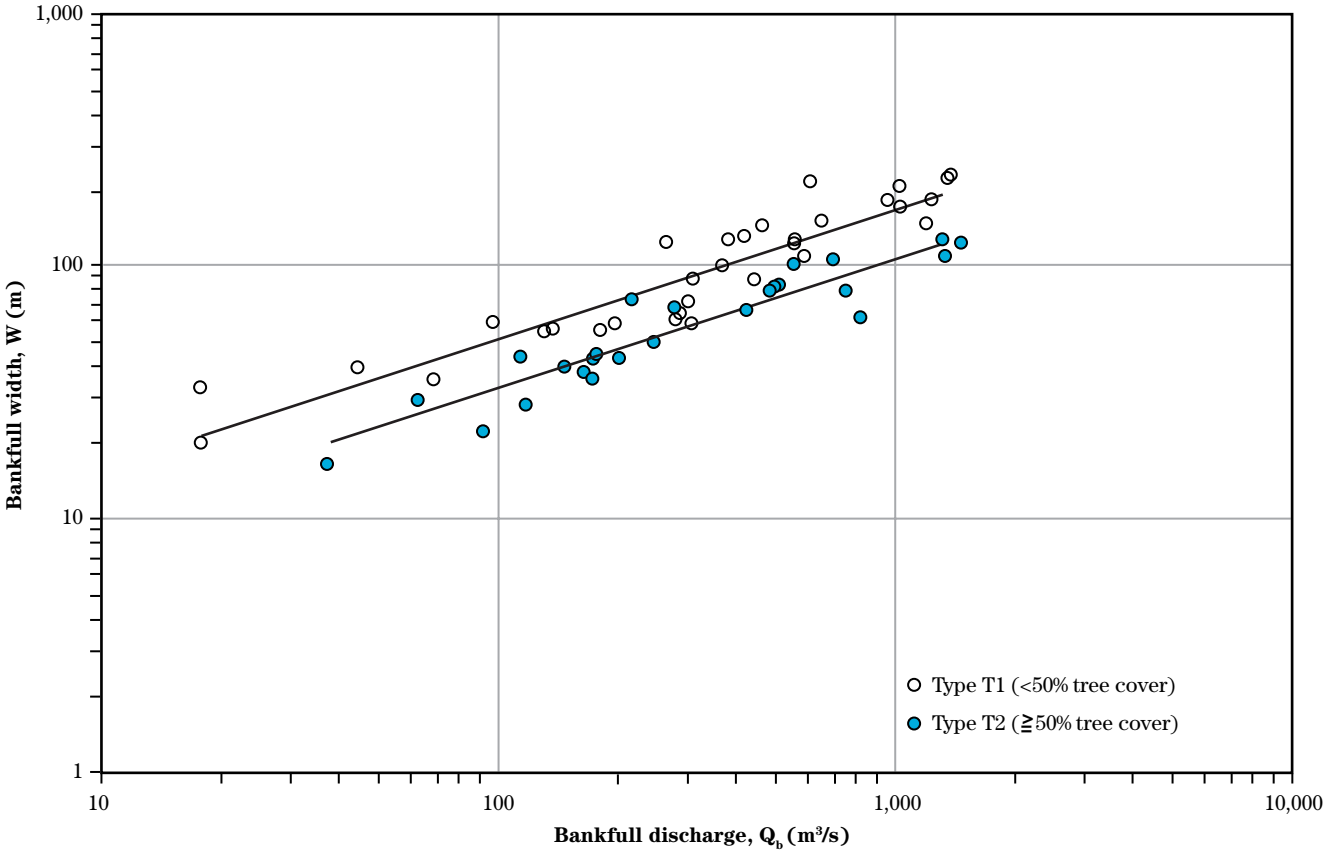


Figure 9–3 Sites used to develop U.S. sand-bed river hydraulic geometry relationships

Figure 9–4 Confidence intervals applied to the hydraulic geometry equation for width based on 32 sand-bed rivers with less than 50% tree cover on the banks (T1). SI units – m and m^3/s (English units – ft and ft^3/s)

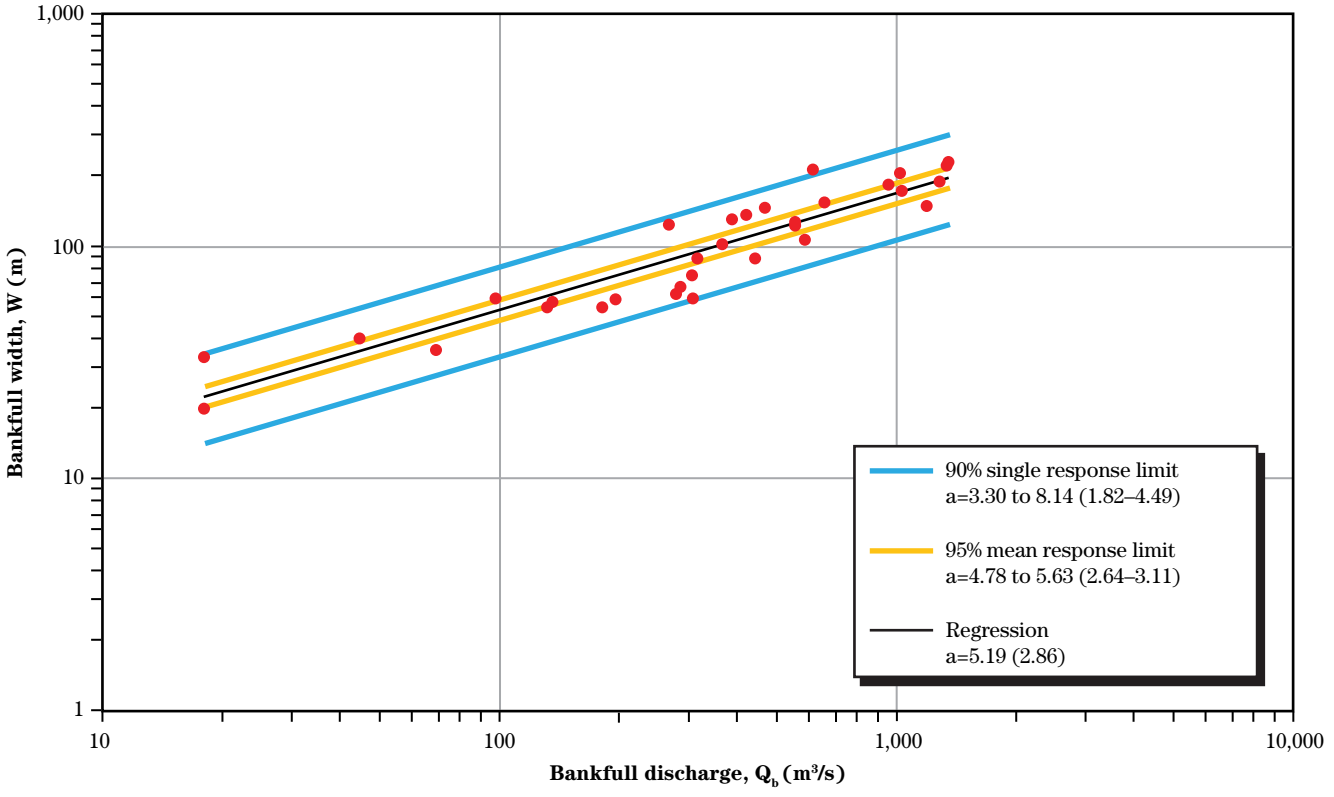


Figure 9–5 Confidence intervals applied to the width hydraulic geometry equation based on 26 sand-bed rivers with at least 50% tree cover on the banks (T2). SI units – m and m^3/s (English units ft and ft^3/s)

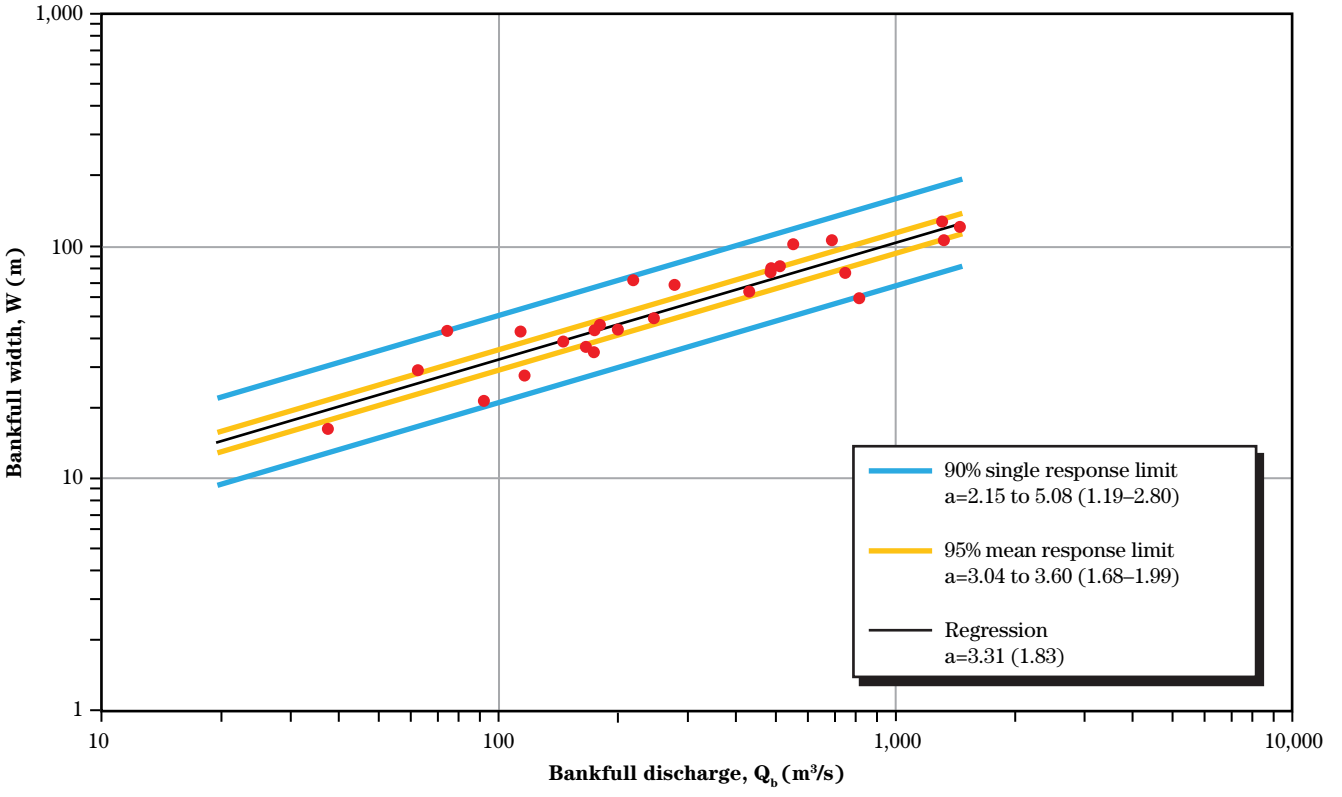


Figure 9–6 Type 1 bankline (T1), less than 50% tree cover

Figure 9–7 Type 2 bankline (T2), greater than 50% tree cover

The hydraulic geometry width predictor is expressed by the general equation:

$$W = aQ^b \qquad \text{(eq. 9–15)}$$

where:
W = channel top width
Q = channel-forming discharge
a = see table 9–8
b = see table 9–8

The hydraulic geometry width predictors each include two sets of confidence bands. The 95 percent mean response limit provides the band in which one can be 95 percent confident that the mean value of the width will occur. This is the confidence interval for the regression line and provides the range of average values of width that can be expected for a given discharge. The 90 percent single response limit provides the envelope curves that contain 90 percent of the data points. This is the confidence interval for an individual predicted value and provides the engineer with the range of possible widths that have been observed to correspond to a given discharge. The confidence interval on an individual predicted value is wider than the confidence interval of the regression line because it includes both the variance of the regression line plus the squared standard deviation of the data set.

While the equations given in table 9–8 may be used for preliminary design purposes, they are subject to several limitations. In the absence of stage-discharge relationships at each site, the bankfull discharge was calculated using Manning's equation and is subject to assumptions related to choice of a resistance coefficient. As cross-sectional geometry was used to calculate discharge, discharge is not truly independent of width in this analysis. Furthermore, only one cross section was measured at each site. Identification of the bankfull reference level, although based on field experience and geomorphic criteria, is always subject to a degree of uncertainty. These factors contribute to the observed variability in the width relationships. Finally, small rivers are not well represented in the data set; therefore, the generalized width predictors should not be applied when channel-forming discharge is less than 600 cubic feet per second in type T1 channels and less than 1,300 cubic feet per second in type T2 channels.

Table 9–8 Hydraulic geometry width predictors for meandering sand-bed rivers

$W = aQ^b$

SI units m and m^3/s (English units ft and ft^3/s)

Data source	Sample size	a	90% single response limit for a	95% mean response limit for a	b	r^2
All sand-bed rivers	58	4.24 (2.34)	2.34–7.68 (1.29–4.24)	3.90–4.60 (2.15–2.54)	0.5	0.76
Type T1: <50% tree cover	32	5.19 (2.86)	3.30–8.14 (1.82–4.49)	4.78–5.63 (2.64–3.11)	0.5	0.87
Type T2: >50% tree cover	26	3.31 (1.83)	2.15–5.08 (1.19–2.80)	3.04–3.60 (1.68–1.99)	0.5	0.85

(d) Hydraulic geometry for gravel-bed rivers

A review of the published gravel-bed river data and hydraulic geometry width predictors for North American and British rivers (Copeland et al. 2001) revealed that North American gravel-bed rivers are generally wider than those found in the United Kingdom, assuming discharge and other conditions are equal. North American data used to develop the hydraulic geometry relationship included data from Brandywine Creek in Pennsylvania (Wolman 1955), Alaskan streams (Emmett 1972), Upper Salmon River in Idaho (Emmett 1975), Colorado, New Mexico, Oregon, Pennsylvania, Tennessee, Utah, West Virginia, and Wyoming (Williams 1978), Alberta, Canada (Annable 1996), and the Rocky Mountain region of Colorado (Andrews 1984). United Kingdom data included data from Nixon (1959), Charlton, Brown, and Benson (1978) and Hey and Thorne (1986). The gravel-bed river data excluded data from braided, anastomosed, and split channel rivers. The hydraulic geometry relationships are shown in fig. 9–8. The difference in these regression curves cannot satisfactorily be explained using the site descriptions given in original publications. A possible explanation is that the United Kingdom sites have on the average more resistant banks than the North American sites. Another plausible explanation is that the North American sites on the average may be flashier. Still, another possibility is that the North American sites may be more active; that is, they may have a higher sediment load. Further research is required to validate these hypotheses.

The hydraulic geometry width predictors for North American and United Kingdom gravel-bed rivers are presented with confidence bands in figures 9–9 and 9–10, respectively. Exponents and coefficients for the hydraulic geometry equation are given in table 9–9. The gravel-bed river data comprise a wide range of bank material types (cohesive, sand, gravel, and composite banks of various strata). However, different width-discharge relationships based on different types of bank material could not be derived for the North American river data from the limited information available.

There were sufficient data available from the United Kingdom gravel-bed rivers to develop distinct width predictors based on erodible banks, low density of trees, and resistant banks, high density of trees (figs. 9–11 and 9–12). These hydraulic geometry relations may be used for preliminary design purposes, recognizing that considerable variability may occur for areas different from the streams used in the development of the equations.

(e) Uncertainty in hydraulic geometry relations

A sufficient number of data points must be measured to ensure that the results from hydraulic geometry analysis are statistically valid. For example, if any three or four random data points were used, a different relation could easily be derived. The fewer and more widely scattered the data points, the less confidence one has in any derived trend. Even with quite a few data points in a relatively homogeneous watershed, there is a great deal of scatter in the data due to natural variability.

Stable natural rivers have morphologies that broadly conform to regime or hydraulic geometry relationships. Therefore, dependent parameters of channel form can be linked to independent controls of flow regime, boundary materials, and riparian vegetation. However, rivers do not follow regime laws precisely. Every river displays local departures from the expected channel form described by morphological equations and possesses inherent variability in space and time. While it is true that natural channel forms are in general predictable, it is also true that each river is in detail unique. Regime dimensions in the natural domain should be interpreted only as representative reach-average, ideal, or target conditions, about which channel morphology fluctuates in time and space.

The coefficient of determination, r^2, in hydraulic geometry analysis numerically represents the amount of variation that can be explained by the selected independent variable. If r^2 is 1.0, there is no variation. The closer the r^2 value is to zero, the less useful the relation, and the wider the scatter in the data. The natural variability of data in a relatively homogeneous watershed such as the upper Salmon River watershed (Emmett 1975) underlines the importance of viewing the data used to develop the curve, not just the curve itself, along with statistical parameters such as r^2 values and confidence limits. If the r^2 value exceeds 95 percent for data collected in natural stream systems, it may indicate autocorrelation or too few data points. Equations given without plotted data points or statistical parameters should be verified for applicability.

Figure 9–8 Downstream width hydraulic geometry for North American gravel-bed rivers, $W = 3.68Q_b^{0.5}$ and U.K. gravel-bed rivers, $W = 2.99Q_b^{0.5}$

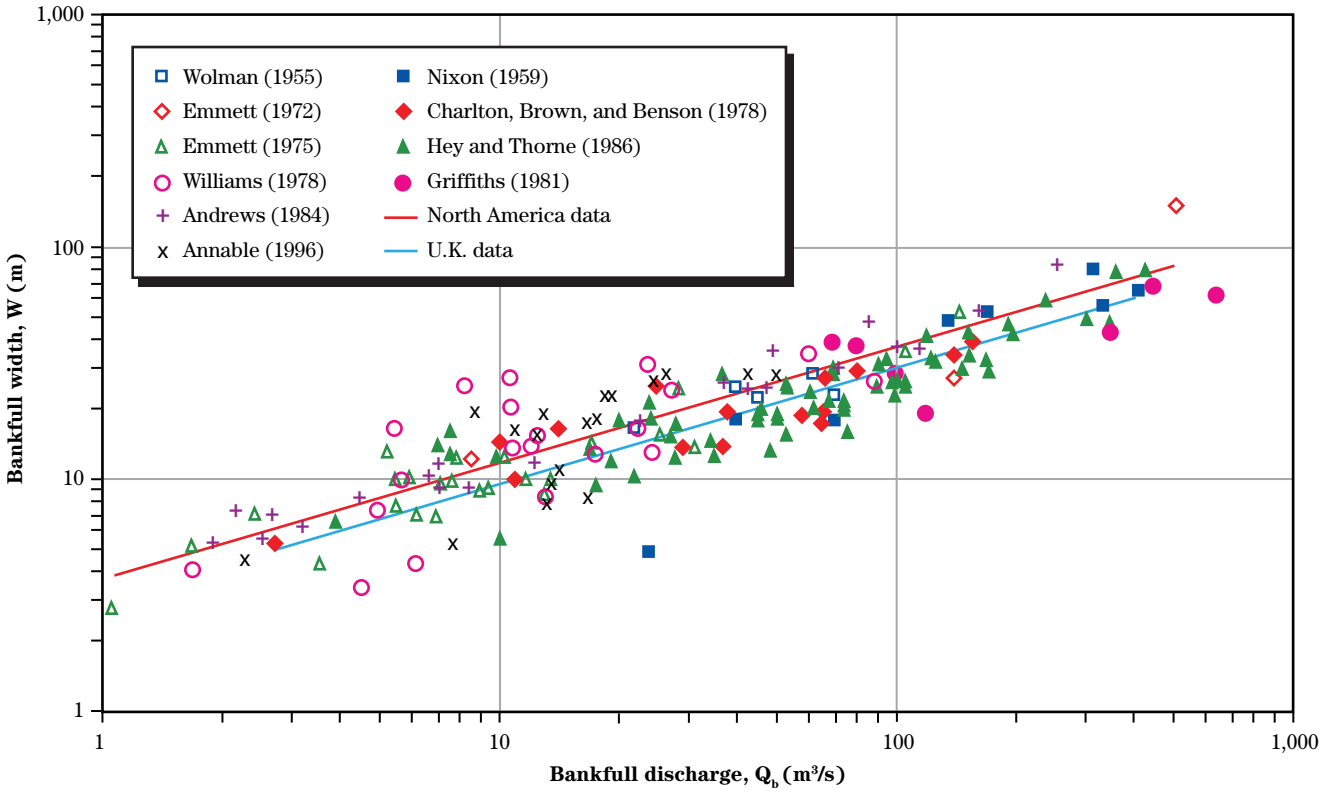


Table 9–9 Hydraulic geometry width predictors for gravel-bed rivers

$$W = aQ^b$$

SI units m and m^3/s (English units ft and ft^3/s)

Data source	Sample size	a	90% single response limit for a	95% mean response limit for a	b	r^2
All North American gravel-bed rivers	94	3.68 (2.03)	2.03–6.68 (1.12–3.69)	3.45–3.94 (1.90–2.18)	0.5	0.80
All U.K. gravel-bed rivers	86	2.99 (1.65)	1.86–4.79 (1.02–2.64)	2.83–3.16 (1.56–1.74)	0.5	0.80
<5% tree or shrub cover, or grass-lined banks (U.K. rivers)	36	3.70 (2.04)	2.64–5.20 (1.46–2.87)	3.49–3.92 (1.93–2.16)	0.5	0.92
≥5% tree or shrub cover (UK rivers)	43	2.46 (1.36)	1.87–3.24 (1.03–1.79)	2.36–2.57 (1.30–1.42)	0.5	0.92

Figure 9–9 Downstream width hydraulic geometry for North American gravel-bed rivers, $W = aQ_b^{0.5}$ with confidence bands. Based on 94 sites in North America. SI units – m and m^3/s (English units ft and ft^3/s)

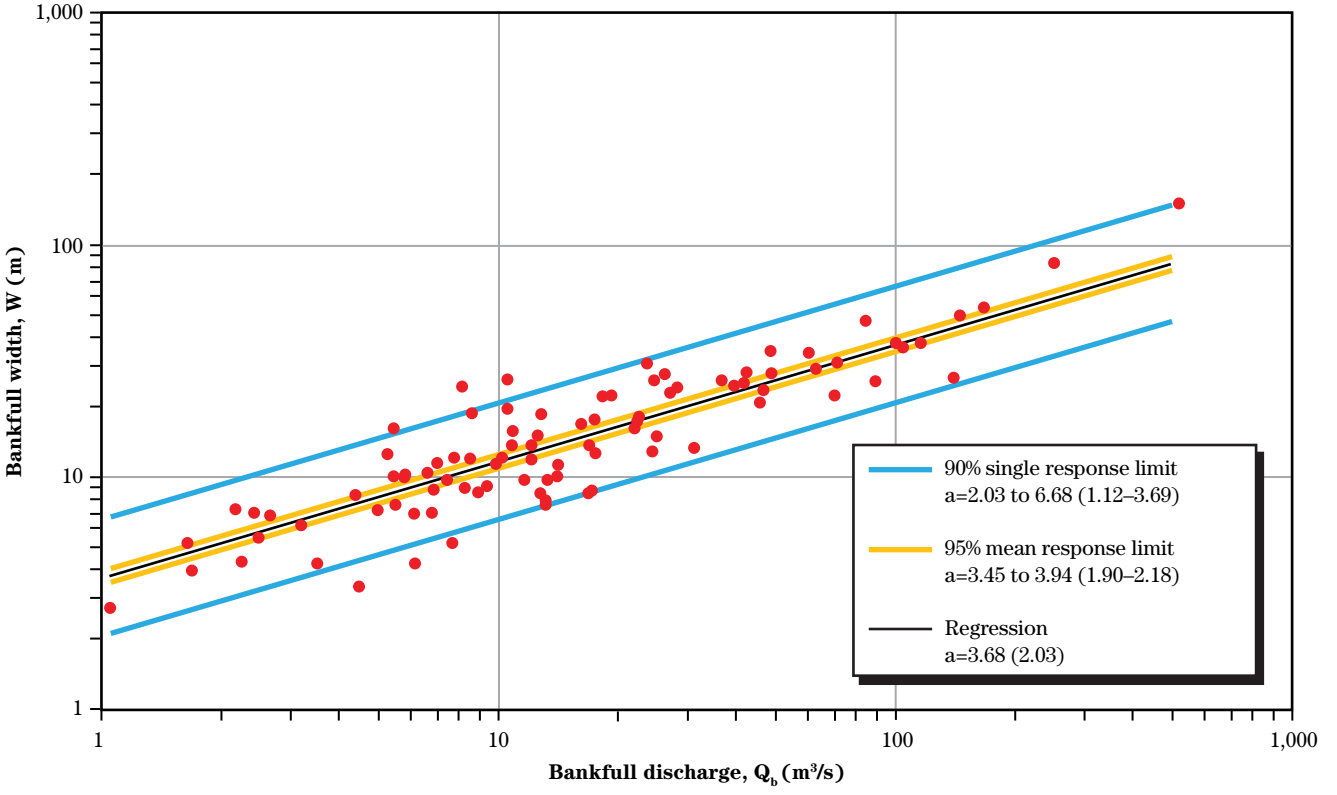


Figure 9–10 Downstream width hydraulic geometry for U.K. gravel-bed rivers, $W = aQ_b^{0.5}$ with confidence bands. Based on 86 sites in the U.K. SI units m and m^3/s (English units ft and ft^3/s)

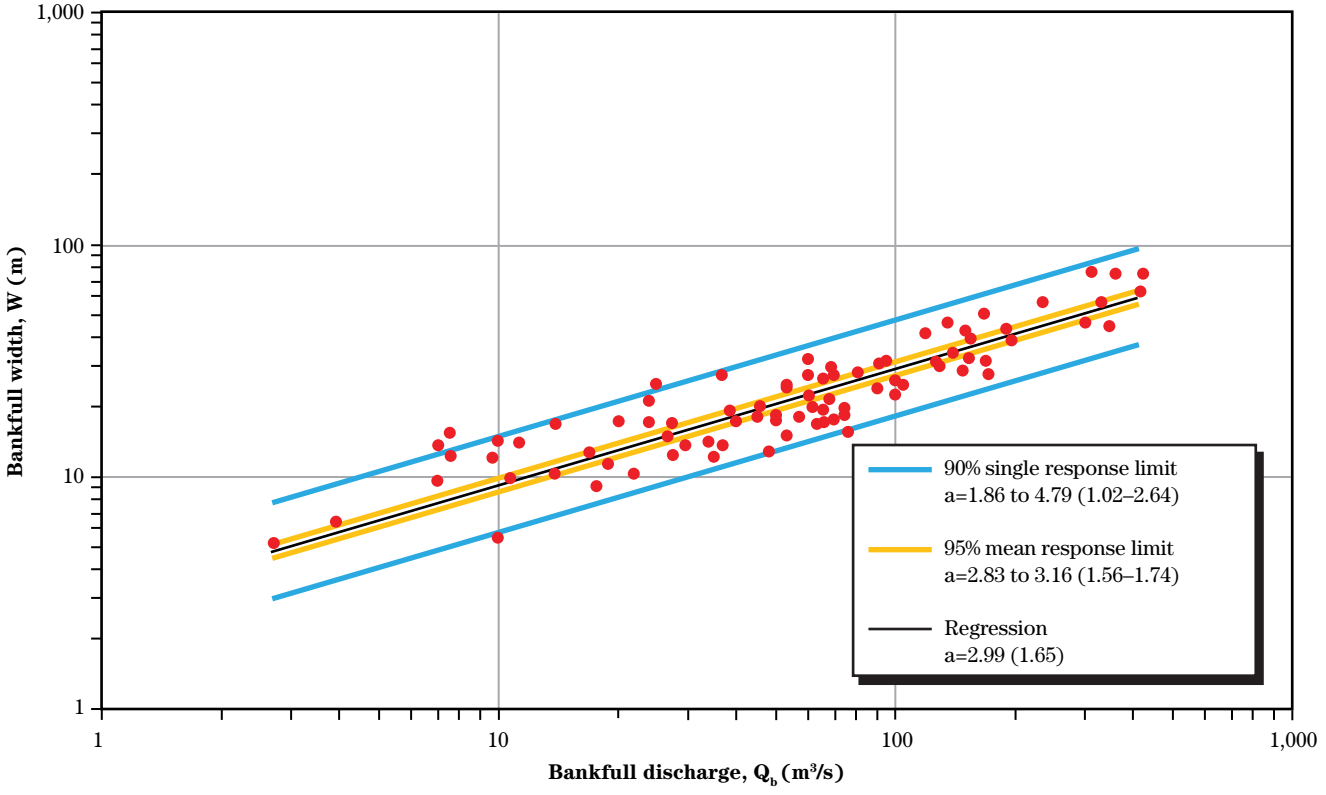


Figure 9–11 Downstream width hydraulic geometry for U.K. gravel-bed rivers, $W = aQ_b^{0.5}$ with confidence bands. Based on 36 sites in the U.K. with erodible banks. SI units m and m^3/s (English units ft and ft^3/s)

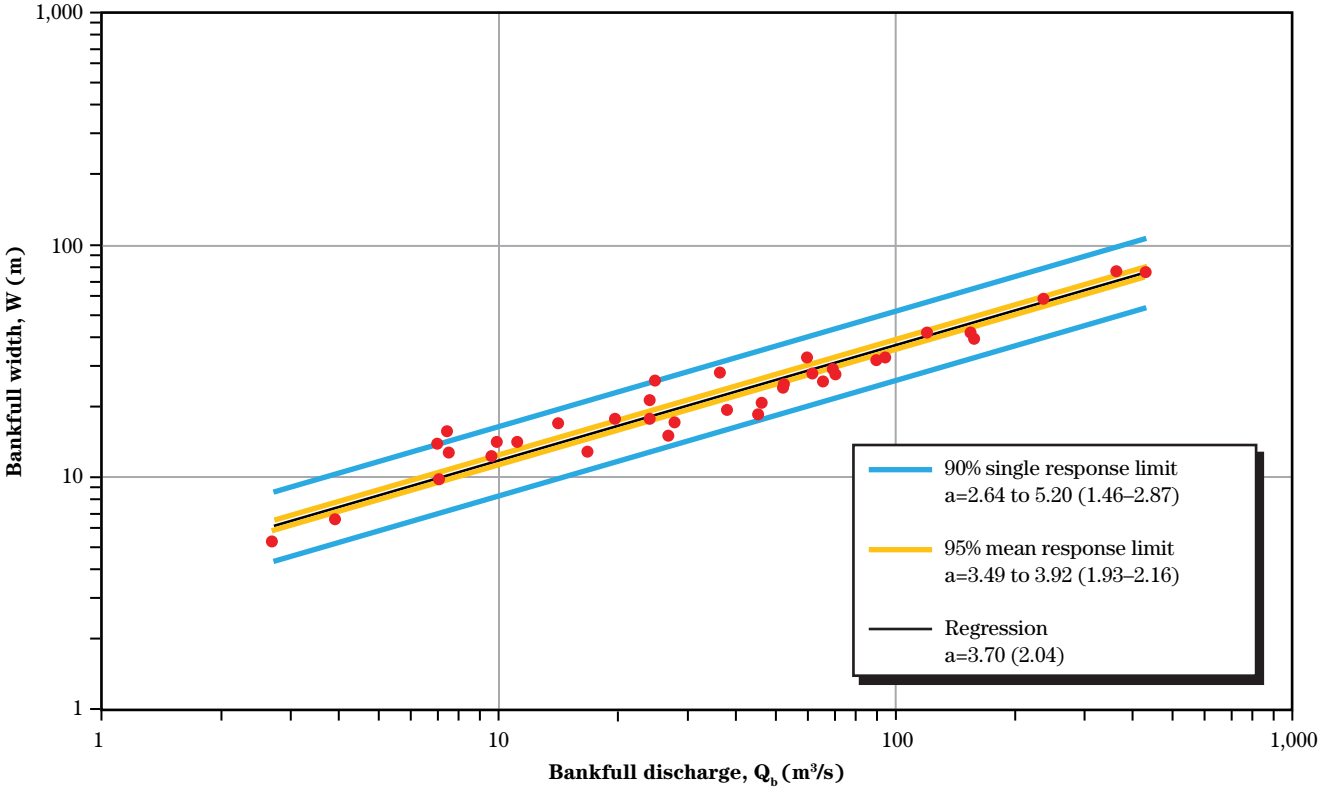
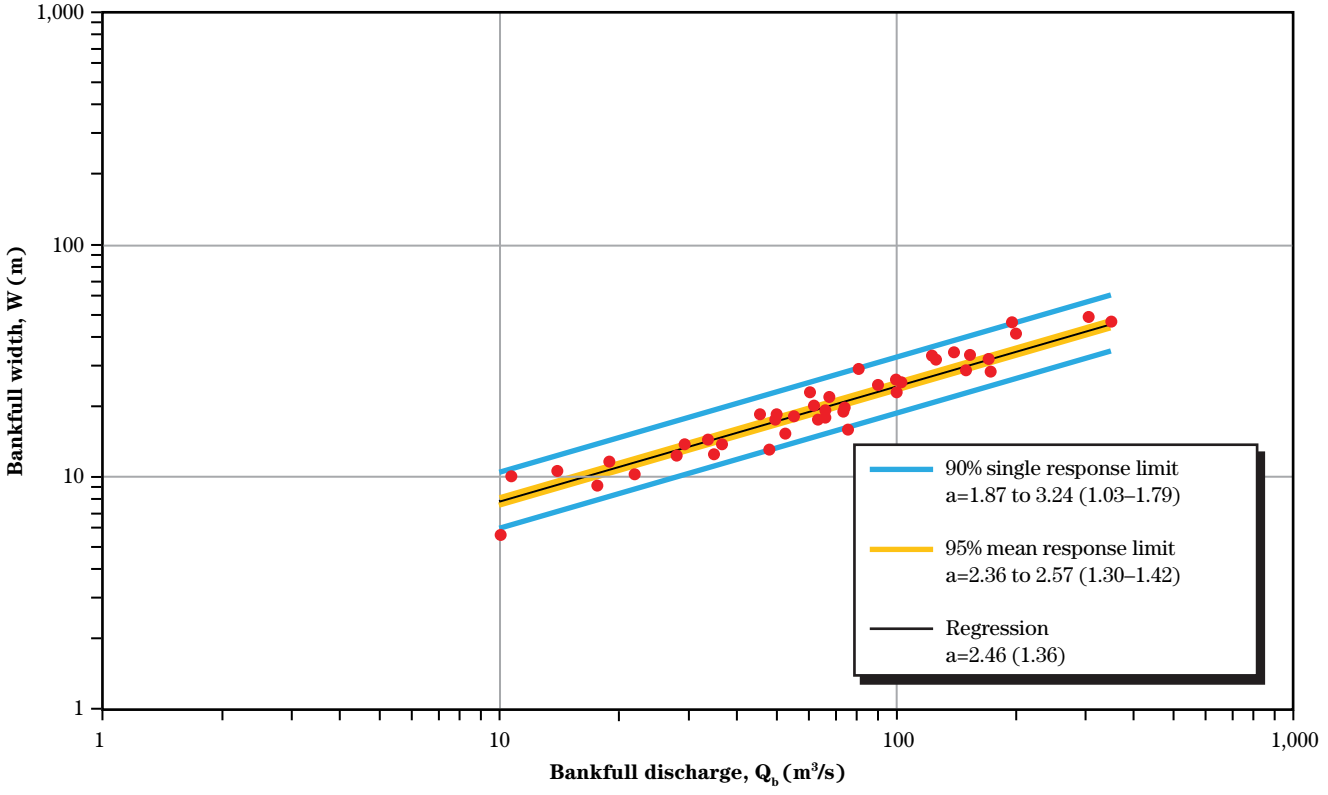


Figure 9–12 Downstream width hydraulic geometry for U.K. gravel-bed rivers, $W = aQ_b^{0.5}$ with confidence bands. Based on 43 sites in the U.K. with resistant banks. SI units m and m^3/s (English units ft and ft^3/s)



(f) Limitations of hydraulic geometry methods

- The formulas provide design variables only for the channel-forming discharge. These design variables may provide the most stable channel, but modifications may be necessary in final design to account for larger flood discharges.
- When developing a hydraulic geometry relationship from field measurements, it is difficult to determine the water surface elevation at channel-forming discharge. This is especially true in unstable channels.
- The hydraulic geometry equation must be developed from physiographically similar streams; that is, streams with depths, slopes, bed and bank material, and sediment inflow concentrations similar to the design channel.
- The assumption that channel dimensions are related only to one or two independent variables is simplistic. The data scatter associated with hydraulic geometry plots demonstrates that stability can occur at more than one combination of width and discharge. The channel-forming discharge may be the most significant factor affecting channel geometry, but other factors can also affect channel dimensions. These include the shape of the annual hydrograph, the shape and magnitude of the annual hydrograph from previous years, upstream or downstream channel control points, and localized variability in alluvial stratum.
- Hydraulic geometry relationships are assumed to be power functions. This assumption provides for visually comforting plots on log-log graph paper, but the actual data scatter may be too great for reliable final engineering design.
- Hydraulic geometry relationships are regression equations and should not be extended beyond the range of the data used to develop them, even in physiographically similar watersheds.

In summary, hydraulic geometry methods suffer the same limitation as the analogy methods. They both depend on a comparison to a channel that is adjusted in some sense. This is true whether it is a reference reach or a channel whose dimensions are used in a hydraulic geometry relationship. If the channel that is to be designed is disturbed or is likely to change due to changes in water or sediment supply, there is really no exact template that is appropriate. Hydraulic geometry relationships are useful for preliminary or trial selection of channel width. Hydraulic and sediment transport analyses are recommended for final channel design.

654.0906 Extremal hypotheses

If a reliable hydraulic geometry relationship cannot be determined from field data or when sediment transport is significant, analytical methods may be employed to obtain a range of feasible solutions. Analytical methods employ an extremal hypothesis as a third equation. One extremal hypothesis assumes that a channel will adjust its geometry so that the time rate of energy expenditure is minimized (Chang 1980; Copeland 1994). Another extremal hypothesis assumes that sediment transport is maximized within the constraints on the system (White, Bettess, and Paris 1982; Millar and Quick 1993). These are equivalent assumptions. Computer programs or look-up charts are required to solve the resistance, sediment transport, and extremal equations simultaneously. The U.S. Army Corps of Engineers Hydraulic Design Package, SAM (Thomas, Copeland, and McComas 2003), as well as HEC–RAS, contains a program to solve these equations. The program uses the Brownlie (1981) resistance and sediment transport equations for sand-bed streams, the Limerinos resistance equation, and the Meyer-Peter and Müller sediment transport equation for gravel-bed streams.

The advantage of using an extremal hypothesis is that a unique solution can be obtained for the dependent variables of width, depth, and slope. However, extensive field experience demonstrates that channels can be stable with widths, depths, and slopes different from those found at the extremal condition. Also, the sensitivity of energy minima or sediment transport maxima to changes in driving variables may be low, so that the channel dimensions corresponding to the extremal value are poorly defined.

654.0907 Constrained dependent variables

In many cases, project constraints limit the theoretical variability in channel geometry. These constraints can be anthropogenic or geologic. For example, the channel slope cannot be greater than the valley slope for a long reach. The channel width may be limited by available rights-of-way. Flood risks and damages may limit allowable depth. For these and many other reasons, the selection of one of the dependent design variables may be based on established project constraints.

654.0908 Analytical methods

After selecting one of the dependent design variables using geomorphic principles, the other two design variables can be computed using a resistance equation and a sediment transport equation. Appropriate equations can be chosen from those described in the American Society of Civil Engineers (ASCE 2006), USACE (1995a), USACE (1991b), Thomas, Copeland, and McComas (2003), or one of many sediment transport textbooks. The data ranges used in the development of sediment transport functions used in Thomas, Copeland, and McComas are given in tables 9–10 through 9–21. These summaries are based on the authors' stated ranges, as presented in their original papers. Otherwise, the summaries were determined based on the author's description of their database in combination with the data listings of Brownlie (1981) or Toffaleti (1968). A review of this information may serve as guidance in selecting the appropriate function.

The stable channel analytical method in the USACE SAM (Thomas, Copeland, and McComas 2003), provides a computer program that simultaneously solves resistance and sediment transport equations. The program provides a family of solutions from which the unique solution for depth and slope can be determined using the width determined from geomorphic principles or from project constraints. This method is described in detail in the following paragraphs.

(a) Stable channel dimensions using analytical techniques

Stable channel dimensions can be calculated analytically using computer programs or spreadsheets. The USACE SAM (Thomas, Copeland, and McComas 2003) calculates stable channel dimensions that will pass a prescribed sediment load without deposition or erosion. This routine is also available in HEC–RAS. The analytical approach (Copeland 1994) determines dependent design variables of width, slope, and depth from the independent variables of discharge, sediment inflow, and bed-material composition. It solves flow resistance and sediment transport equations simultaneously, leaving one dependent variable optional.

The extremal hypothesis (minimum stream power) can be used as a third equation for a unique solution. Be aware of the cautions associated with using the extremal hypothesis as described in the previous section of this chapter. This method is based on a typical trapezoidal cross section and assumes steady, uniform flow. The method is especially applicable to small streams because it accounts for transporting the bed-material sediment discharge in the water above the bed, not the banks, and because it separates total hydraulic roughness into bed and bank components.

(b) Basic equations for sand-bed streams

For sand-bed streams, the sediment transport and resistance equations developed by Brownlie are recommended because they account for bed-form roughness. There are separate resistance equations for upper and lower regime flow. Upper regime flow is characterized by relatively high velocities and high sediment transport. The bedforms are plane bed, antidunes or chutes, and pools, which do not provide significant form resistance. Lower regime flow is characterized by relatively low velocity and low sediment transport. The bedforms are dunes or ripples, which provide significant form resistance. The equations are dimensionless and can be used with any consistent set of units.

Upper regime

$$R_b = 0.2836 D_{50} q_*^{0.6248} S^{-0.2877} \sigma^{0.0813} \qquad \text{(eq. 9–16)}$$

Relatively high velocities and high sediment transport

Lower regime

$$R_b = 0.3742 D_{50} q_*^{0.6539} S^{-0.2542} \sigma^{0.1050} \qquad \text{(eq. 9–17)}$$

$$q_* = \frac{Vd}{\sqrt{gD_{50}^3}} \qquad \text{(eq. 9–18)}$$

Relatively low velocities and low sediment transport

where:
R_b = hydraulic radius associated with the bed (ft or m)
D_{50} = median grain size (ft or m)
S = slope

σ = geometric bed-material gradation coefficient
V = average velocity (ft/s or m/s)
d = water depth (ft or m)
g = acceleration of gravity (ft/s^2 or m/s^2)

To determine if upper or lower regime flow exists for a given set of hydraulic conditions, a grain Froude number F_g and a variable F_g' were defined by Brownlie. According to Brownlie, upper regime occurs if S>0.006 or if $F_g > 1.25F_g'$, and lower regime occurs if $F_g < 0.8F_g'$. Between these limits is the transition zone. In the SAM, $F_g = F_g'$ is used to distinguish between upper and lower regime flow. If a spreadsheet analysis is used, the user may choose a different criterion for determining the break between upper and lower regime flow in the transition zone.

$$F_g = \frac{V}{\sqrt{gd_{50}\left(\frac{\gamma_s - \gamma}{\gamma}\right)}} \qquad \text{(eq. 9–19)}$$

$$F_g' = \frac{1.74}{S^{0.3333}} \qquad \text{(eq. 9–20)}$$

where:
γ_s = specific weight of sediment (lb/ft^3 or N/m^3)
γ = specific weight of water (lb/ft^3 or N/m^3)

The hydraulic radius of the side slope is calculated using Manning's equation:

$$R_s = \left(\frac{(V)(n_s)}{(CME)(S^{0.5})}\right)^{1.5} \qquad \text{(eq. 9–21)}$$

where:
R_s = hydraulic radius associated with the side slopes (ft or m)
V = average velocity (ft/s or m/s)
n_s = Manning's roughness coefficient for the bank
CME = 1.486 (English units) = 1.0 (SI units)

If the roughness height k_s of the bank is known, then it can be used instead of Manning's roughness coefficient to define bank roughness. Strickler's equation can be used to calculate the bank roughness coefficient:

$$n_s = 0.039k_s^{\frac{1}{6}} \qquad \text{(eq. 9–22)}$$

where:
k_s = roughness height (ft)

$$n_s = 0.048k_s^{\frac{1}{6}} \qquad \text{(eq. 9–23)}$$

where:
k_s = roughness height (m)

For riprap, k_s should be set equal to the minimum design d_{90}.

Composite hydraulic parameters are partitioned in the manner proposed by Einstein (1950):

$$A = R_bP_b + R_sP_s \qquad \text{(eq. 9–24)}$$

where:
A = total cross-sectional area (ft^2 or m^2)
P_b = perimeter of the bed (ft or m)
P_s = perimeter of the side slopes (ft or m)

This method assumes that the average velocity for the total cross section is representative of the average velocity in each subsection.

Concentration, C, in parts per million, is calculated using the Brownlie sediment transport equation, which is also a regression equation. The equation is based on the same extensive set of flume and field data used to develop the Brownlie resistance equations. This equation is recommended because of its compatibility with the resistance equations, which are coupled with the sediment transport equation in the numerical solution. The equation is dimensionless and can be used with any consistent set of units.

$$C = 9{,}022\left(F_g - F_{go}\right)^{1.978} S^{0.6601}\left(\frac{R_b}{D_{50}}\right)^{-0.3301}$$

$$F_{go} = \frac{4.596\tau_{*0}^{\;0.5293}}{S^{0.1405}\sigma^{0.1606}}$$

$$\tau_{*0} = 0.22Y + 0.06(10)^{-7.7Y}$$

$$Y = \left(\sqrt{\frac{\gamma_s - \gamma}{\gamma}}\right)^{-0.6}$$

$$R_g = \frac{\sqrt{gD_{50}^3}}{\upsilon} \qquad \text{(eq. 9–25)}$$

where:
C = concentration (ppm)
R_b = bed hydraulic radius (ft or m)
D_{50} = median grain size (ft or m)
σ = bed-material gradation coefficient
γ_s = specific weight of sediment (lb/ft^3 or N/m^3)
γ = specific weight of water (lb/ft^3 or N/m^3)
g = acceleration of gravity (ft/s^2 or m/s^2)
ν = kinematic viscosity (ft^2/s or m^2/s)
The other variables are dimensionless.

Table 9–10 Ackers-White transport function

Parameter	Flume data
Particle size range (mm)	0.04–7.0
Specific gravity	1.0–2.7
Multiple size classes	No
Velocity (ft/s)	0.07–7.1
Depth (ft)	0.01–1.4
Slope (ft/ft)	0.00006–0.037
Width (ft)	0.23–4
Water temperature (˚F)	46–89

Table 9–11 Brownlie transport function

Parameter	River data	Flume data
Particle size range (mm)	0.086–1.4	0.088–1.4
Multiple size classes	No	No
Velocity (ft/s) [calculated]	1.2–7.9	0.7–6.6
Depth (ft)	0.35–57	0.11–1.9
Slope (ft/ft)	0.00001–0.0018	0.00027–0.017
Width (ft)	6.6–3640	0.83–8.0
Water temperature (˚F)	32–95	35–102

Table 9–12 Colby transport function

Parameter	Data range
Particle size range (mm)	0.18–0.70
Multiple size classes	No
Velocity (ft/s)	0.70–8.0
Depth (ft)	0.20–57
Slope (ft/ft)	0.000031–0.010
Width (ft)	0.88–3000
Water temperature (˚F)	32–89
Correction for fines (ppm)	Yes

Table 9–13 Einstein transport function

Parameter	Flume data
Particle size range (mm)	0.78–29
Multiple size classes	Yes
Velocity (ft/s)	0.9–9.4
Depth (ft)	0.03–3.6
Slope (ft/ft)	0.00037–0.018
Width (ft)	0.66–6.6
Water temperature (˚F)	Not reported

Table 9–14 Laursen (Copeland 1994) transport function

Parameter	River data	Flume data
Median particle size range (mm)	0.08–0.70	0.011–29
Multiple size classes	Yes	Yes
Velocity (ft/s)	0.068–7.8	0.70–9.4
Depth (ft)	0.67–54	0.03–3.6
Slope (ft/ft)	0.0000021–0.0018	0.00025–0.025
Width (ft)	63–3640	0.25–6.6
Water temperature (˚F)	32–93	46–83

Table 9–15 Laursen (Madden 1985) transport function

Parameter	Data range
Particle size range (mm)	0.04–4.8
Multiple size classes	Yes
Velocity, (ft/s)	0.85–7.7
Depth (ft)	0.25–54
Slope (ft/ft)	0.00001–0.1
Width (ft)	3–3640
Water temperature (˚F)	36–90

Table 9–16 Meyer-Peter and Müller (1948) transport function

Parameter	Data range
Particle size range (mm)	0.4–29
Particle specific gravity	1.25–4
Multiple size classes	Yes
Velocity (ft/s)	1.2–9.4
Depth (ft)	0.03–3.9
Slope (ft/ft)	0.0004–0.02
Width (ft)	0.5–6.6
Water temperature (˚F)	Not published

Table 9–17 Parker transport function

Parameter	River data
Median particle size range (mm)	18–28
Total particle size range (mm)	2–102
Multiple size classes	Yes
Velocity (ft/s)	2.6–3.7
Depth (ft)	1.0–1.5
Slope (ft/ft)	0.0097–0.011
Width (ft)	16–20
Water temperature (˚F)	41–44

Table 9–18 Profitt (Profitt and Sutherland 1983) transport function

Parameter	River data
Particle size range (mm)	2.90–12
Multiple size classes	Yes
Velocity (ft/s)	2.00–3.4
Depth (ft)	0.35–0.84
Slope (ft/ft)	0.003
Width (ft)	2.00
Water temperature (˚F)	59–63

Table 9–19 Schoklitsch transport function

Parameter	Data range
Particle size range (mm)	0.3–4.9
Multiple size classes	No
Velocity (ft/s)	0.8–4.5
Depth (ft)	0.037–0.74
Slope (ft/ft)	0.00012–0.055
Width (ft)	0.23–2.0
Water temperature (˚F)	Not published

Table 9–20 Toffaleti transport function

Parameter	River data	Flume data
Median particle size range (mm)	0.095–0.76	0.91–0.45
Total particle size range (mm)	0.062–4	0.062–4
Multiple size classes	Yes	Yes
Velocity (ft/s)	0.7–7.8	0.7–6.3
Hydraulic radius (ft)	0.7–56.7	0.07–1.1
Slope (ft/ft)	0.000002–0.0011	0.00014–0.019
Width (ft)	63–3640	0.8–8
Water temperature (˚F)	32–93	40–93

Table 9–21 Yang transport function

Parameter	Sand data	Gravel data
Particle size range (mm)	0.15–1.7	2.5–7.0
Multiple size classes	No	No
Velocity, (ft/s)	0.8–6.4	1.4–5.1
Depth (ft)	0.04–50	0.08–0.72
Slope (ft/ft)	0.000043–0.028	0.0012–0.029
Width (ft)	0.44–1750	0.7–1.3
Water temperature (˚F)	32–94	Not reported

(c) Basic equations for gravel-bed streams

For gravel-bed streams, equations more appropriate for coarse bed streams should be used in the analytical solution. The Limerinos equation is recommended to calculate grain roughness on the bed. The Meyer-Peter and Müller equation is recommended to calculate sediment transport. The advantage of the Limerinos equation is that it accounts for the decrease in roughness with increasing water depth in cases where the bed roughness is primarily due to the dimensions of the sediment grains (sand, gravel, or cobbles) on the bed. The Manning equation can be used to calculate the roughness on the channel side slope. The Manning equation is appropriate for this case because bank roughness is best estimated using experience and engineering judgment. Additional roughness may be added in the manner suggested by Cowan (1956).

The Limerinos equation accounts for the grain roughness in a uniform reach of a gravel-bed stream that is relatively free of bedforms. The Limerinos equation may be presented in dimensionless units as:

$$\frac{V}{U_*'} = 5.66 \log\left(3.80 \frac{R_b'}{D_{84}}\right) \qquad \text{(eq. 9–26)}$$

where:
V = average velocity (ft/s or m/s)
U_*' = shear velocity associated with grain roughness (ft/s or m/s)
R_b' = hydraulic radius associated with grain roughness (ft or m)
D_{84} = grain size for which 84 percent of the bed is finer (ft or m)

Manning's roughness coefficient associated with grain roughness can be determined from the Limerinos equation:

$$n_b' = \frac{(CME)\left(R_b'^{\frac{1}{6}}\right)}{g5.66 \log\left(3.80 \frac{R_b'}{D_{84}}\right)} \qquad \text{(eq. 9–27)}$$

where:
n_b' = roughness coefficient associated with bed
CME = 1.486 English units (1.0 SI units)
R_b' = bed hydraulic radius associated with grain roughness (ft or m)
g = acceleration of gravity (ft/s or m/s)
D_{84} = grain size for which 84 percent of the bed is finer (ft or m)

Additional bed roughness may be added to the grain bed roughness using the Cowan (1956) method. Roughness may be added to account for factors such as surface irregularities, variability in channel shape, obstructions, vegetation, and meandering. Meandering can be accounted for with a meandering coefficient, m. In a straight channel, the meandering coefficient is 1.0. Appropriate values for the meandering coefficient and additions to the grain roughness n value can be found in Cowan (1956) and Chow (1959). Using the Cowan equation the total bed roughness coefficient is:

$$n_b = m\left(n_b'' + n_b'\right) \qquad \text{(eq. 9–28)}$$

where:
n_b = total bed roughness coefficient
m = Cowan meander coefficient
n_b'' = bed roughness other than grain roughness

Using the Manning equation the hydraulic radius associated with the bed is calculated

$$R_b = \left(\frac{(V)(n_b)}{\left(S^{\frac{1}{2}}\right)(CME)}\right)^{\frac{3}{2}} \qquad \text{(eq. 9–29)}$$

where:
S = channel slope, dimensionless

The hydraulic radius associated with the bank or side slope is

$$R_s = \left(\frac{(V)(n_s)}{\left(S^{\frac{1}{2}}\right)(CME)}\right)^{\frac{3}{2}} \qquad \text{(eq. 9–30)}$$

where:
n_s = the roughness coefficient associated with the side slope

Values for m, n_b'' and n_s must be selected by the designer. Using the Cowan method provides water surface elevations that account for the total channel roughness and not just the grain roughness.

Sediment transport can be calculated using the Meyer-Peter and Müller equation:

$$\left(\frac{k_r}{k'_r}\right)^{\frac{3}{2}} \gamma_w R_b S = 0.047\left(\gamma_s - \gamma_w\right) D_m + 0.25\left(\frac{\gamma_w}{g}\right)^{\frac{1}{3}}\left(\frac{\gamma_s - \gamma_w}{\gamma_s}\right)^{\frac{2}{3}} g_s^{\frac{2}{3}}$$

(eq. 9–31)

where:

k_r = total bed roughness = $\frac{1}{n_b}$

k'_r = particle roughness = $\frac{1}{n'_b}$

γ_w = specific weight of water (lb/ft^3 or N/m^3)

γ_s = specific weight of sediment (lb/ft^3 or N/m^3)

D_m = median sediment size (ft or m)

g_s = sediment transport (lb/s-ft or N/s-m)

R_b = bed hydraulic radius (ft or m)

and

$$\left(\frac{k_r}{k'_r}\right)^{\frac{3}{2}} R_b = R'_b \qquad \text{(eq. 9–32)}$$

and

$$D_m = \sum_{i=1}^{n} f_i D_i \qquad \text{(eq. 9–33)}$$

where:

f_i = fraction of size class "i" in bed

D_i = geometric mean diameter of size class "i" in bed (ft or m)

(d) Calculating sediment discharge and concentration

A typical cross section, with the critical hydraulic parameters labeled, is shown in figure 9–13. The concentration calculated from the sediment transport equation applies only vertically above the bed. Total sediment transport in weight per unit time is calculated by the following equation:

$$Q_s = \gamma CBDV \qquad \text{(eq. 9–34)}$$

where:

Q_s = sediment transport (weight/time)

B = base width, length

An average concentration for the total discharge is then calculated:

$$\bar{C} = \frac{Q_s}{0.027Q} \qquad \text{(eq. 9–35)}$$

where:

$\bar{C}$ = concentration using the total discharge (ppm)

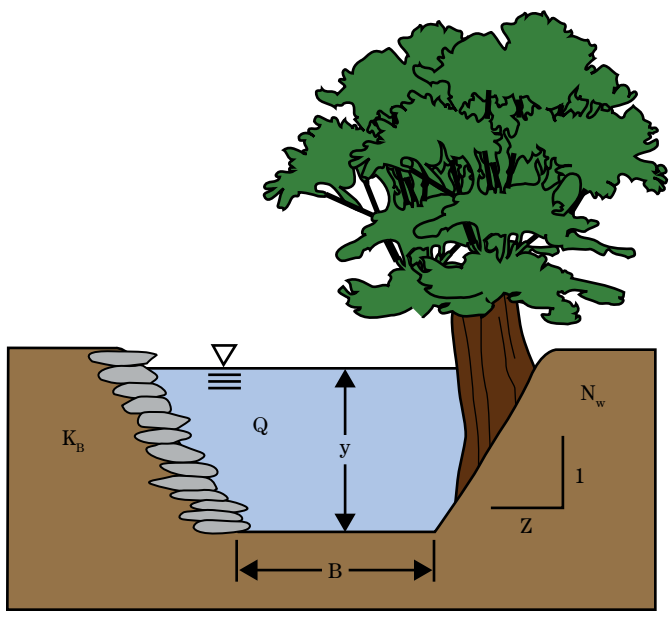
Q_s = sediment transport (tons/d)

Q = discharge (ft^3/s)

(e) Input requirements

Required input data for the analytical method are sediment inflow concentration, side slope, bank roughness coefficient, additional channel roughness and meandering coefficients for the Cowan method, bed material D_{50}, bed-material gradation coefficient, and water discharge. If sediment inflow is to be calculated, which is the recommended procedure, additional data are required for the supply reach. These are base width, side slope, bank roughness coefficient, bed material

Figure 9–13 Typical cross section used in analytical method



median grain size, geometric gradation coefficient, average slope, and discharge. It is important that the base width be representative of the total movable bed width of the channel. Additional channel roughness due to surface irregularities, variability in channel shape, obstructions, instream structures, vegetation, and meandering are added using the Cowan method. If either the USACE SAM or HEC–RAS program is used, adding additional roughness with the Cowan method is only available with the gravel-bed option. If the sand-bed option is used, only grain and form roughness is included in the Brownlie equations, so additional roughness can only be added by increasing the roughness coefficient assigned to the bank. In this case, the bank roughness should serve as a composite of all additional roughness factors. This can be accomplished using one of the hydraulic compositing methods described in NEH654.06. Only flow over the bed is considered capable of transporting the bed-material sediment load.

Water discharge
The design discharge is critical in determining appropriate dimensions for the channel. The channel-forming discharge will provide the most stable channel, but it is also important to evaluate how the design channel will respond during flood events. The channel-forming discharge is typically used to set channel dimensions, and flood discharges are used to evaluate channel performance at design conditions.

Investigators have proposed different methods for estimating the channel-forming discharge. The 2-year frequency peak discharge is sometimes used for perennial streams. Some have suggested that the 10-year frequency peak discharge is more appropriate for ephemeral and intermittent streams. The bankfull discharge is sometimes suggested. Others prefer using the effective discharge, which is the discharge that transports the most bed-material sediment. Currently, there is no generally accepted method for determining the channel-forming discharge. It is recommended that a range of discharges be used in the analysis to test sensitivity of the solution.

Inflowing sediment discharge
This is the concentration of the inflowing bed-material load. The bed-material load should be calculated using the same sediment transport equation and same hydraulic equations that are used in the analysis of the design channel. This is automatically done in USACE SAM or HEC–RAS, if the dimensions and bed-material composition of the upstream supply reach are supplied as input data. Measured data may be used to evaluate the applicability of the Brownlie or Meyer-Peter and Müller equations, but measured data should not be used as input to the analytical method.

Valley slope
Valley slope is the maximum possible slope for the channel invert. The valley slope is determined by the local topography, and a channel with a slope equal to the valley slope would be straight. The valley slope is used to test for sediment deposition. If the minimum slope that will transport the incoming bed-material load is greater than the valley slope, it is not possible to design a stable channel, and deposition is inevitable.

Bank slopes and roughness
The analytical method assumes that all bed-material transport occurs over the bed of the cross section and that none occurs above the side slopes. Therefore, the portion of water conveyed above the side slopes expends energy, but does not transport sediment. This makes the selection of base width in the supply reach important. The base width should reflect the entire alluvial boundary of the channel. In the design reach, the designer must select the channel side slope and side slope roughness. As in the supply reach, sediment transport is calculated only above the base width in the design reach. Therefore, sediment discharge will increase with the selection of steeper design bank angles.

(f) Range of solutions

For each specified combination of water discharge, sediment transport rate, and transport grain size, unique values of slope and depth are calculated. This can be used to evaluate stability in an existing channel or to evaluate stability in a proposed channel. Considering river morphology is important when interpreting these calculated values. Consistent is also important in the selection of channel dimensions; that is, once a width is selected, the depth and slope are fixed. This allows the designer to consider specific project constraints, such as right-of-way, bank height, sinuosity, bend radius, and minimum bed slope. A consistent set of channel dimensions can then be computed.

If the calculations indicate that the slope of the project channel needs to be less than the natural terrain, the calculated slopes can be used to aid in spacing drop structures or in introducing sinuosity into the project alignment.

An example of a family of slope-width solutions that satisfy the resistance and sediment transport equations for the design discharge is illustrated in figure 9–14. Designers typically use these to focus on a range of appropriate width slope combinations, rather than on a single value set. Any combination of slope and base width from this curve will be stable for the prescribed channel design discharge. Combinations of width and slope that plot above the stability curve will result in degradation, and combinations below the curve will result in aggradation. The greater the distance from the curve, the more severe the instability.

Constraints on this wide range of solutions may result from a maximum possible slope or a width constraint due to right-of-way. Maximum allowable depth could also be a constraint. With constraints, the range of solutions is reduced.

Different water and sediment discharges will produce different stability curves. A channel designed for the channel-forming discharge may not be stable at a different discharge. To evaluate the significance of this difference, a stable channel solution is first obtained for the channel-forming discharge. Then, stability curves are calculated for a range of discharges to determine how sensitive the channel dimensions are to variations in water and sediment inflow events. Figure 9–15 shows two stability curves for the same supply reach, but different discharges. The stability curve in this figure is for a channel-forming discharge of 5,000 cubic feet per second. Any width-slope solution along this line will theoretically provide a channel with long-term sediment continuity and stability. If the design channel has a depth constraint for flood control, a width-slope solution is selected from the right end of the stability curve where widths are greater and depths lower.

Conversely, if the design channel has a width constraint due to limited right-of-way, the width-slope solution is selected from the left end of the stability curve. To evaluate channel response for another discharge, a new stability curve is calculated and the design dimensions and compared to the new stability curve. For example, in figure 9–15, a stability curve for a flood discharge of 30,000 cubic feet per second is shown. Width-slope solutions that plot above the flood stability curve indicate that the design channel will degrade during the flood, and width-slope solutions that plot below the flood stability curve will aggrade during the flood. Figure 9–15 shows that degradation will occur during the flood in the channel designed with a depth constraint, and that aggradation will occur during the flood in the channel designed with a width constraint. Note that there is only one combination of width-slope solutions that satisfy sediment continuity for both discharges.

Long-term aggradation and degradation are associated with the channel-forming discharge, but short-term aggradation or degradation can occur during a flood event depending on which channel dimensions are selected from the stable channel stability curve.

Using a spreadsheet or USACE SAM or HEC–RAS, stable channel dimensions can be calculated for a range of widths on either side of a prescribed median value. It is recommended that calculations be made for at least 20 base widths, each with an increment of 0.1 times the median base width. Stability curves can then be plotted from these data. Typically there will be two solutions for each slope.

A solution for minimum stream power can also be calculated. This solution represents the minimum slope that will transport the incoming sediment load. Opinions are divided regarding the use of minimum stream power to uniquely define channel stability.

Figure 9–14 Stability curve from stable channel analytical method

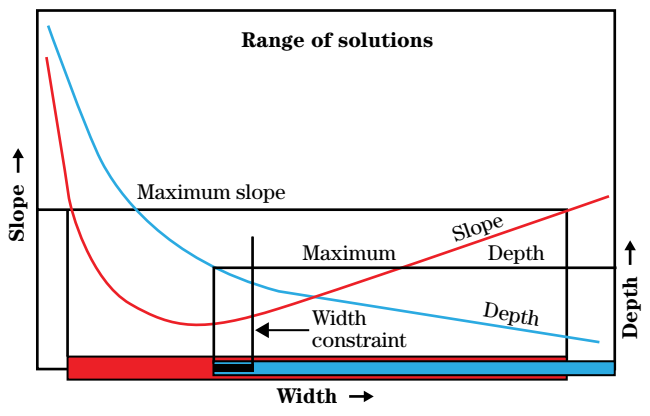
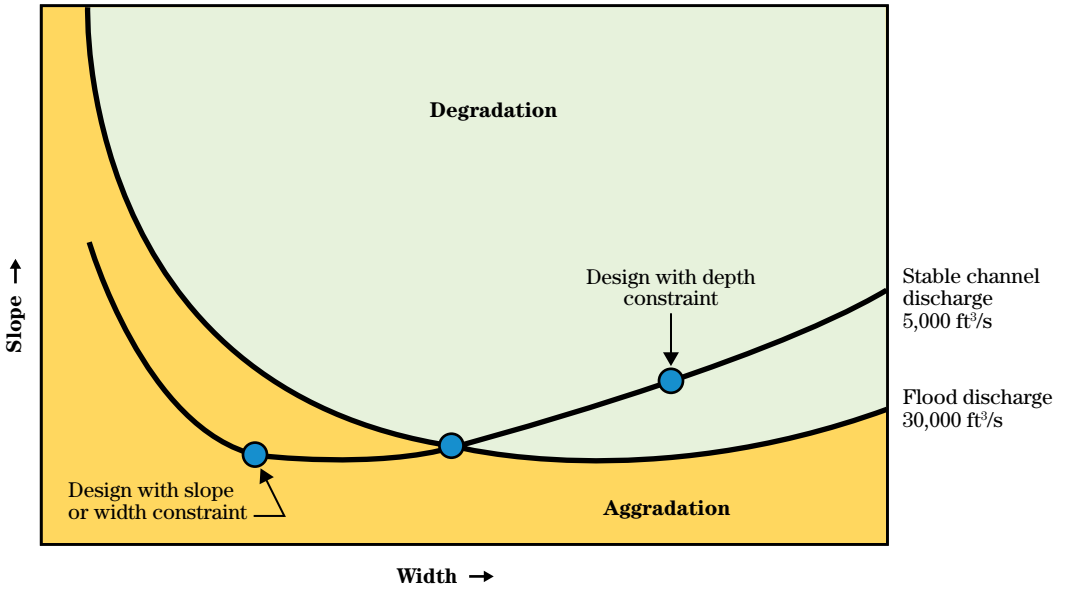


Figure 9–15 Stability curves for channel-forming (stable channel) discharge and a flood discharge



654.0909 Sediment impact analysis

Stream restoration projects should not be designed using only a single flow event and the sediment load transported by that event. This approach does not account for potential instability driven by the range of natural flow events. A determination of the potential for aggradation or degradation in a channel reach requires an assessment of the reach-scale sediment budget. The sediment impact assessment is a closure loop at the end of the design procedure to:

- validate the efficacy of the restored channel geometry
- identify flows which may cause aggradation or degradation over the short term (these changes are inevitable and acceptable in a dynamic channel)
- recommend minor adjustments to the channel design to ensure dynamic stability over the medium to long term

This can be accomplished using a sediment budget approach for relatively simple projects or by using a numerical model that incorporates a solution of the sediment continuity equation for more complex projects. More information on this subject is provided in NEH654.13.

654.0910 Basic steps in alluvial channel design

Step 1 Determine the channel-forming discharge—The initial design step is to determine the stable geometry for a single discharge. Use bankfull discharge, effective discharge, or a specific peak frequency as described in NEH654.05.

Step 2 Determine sediment inflow for the project reach—Calculate a sediment transport rating curve for the upstream supply reach. The sediment discharge may be computed based on a typical upstream cross section using a normal depth equation and an appropriate sediment transport equation.

Step 3 Develop a stability curve—Calculate a family of slope-width-depth solutions that satisfy resistance and sediment transport equations for the channel-forming discharge. This step provides a channel geometry that is capable of transporting the inflowing sediment load through the project reach. The equations are used to calculate the design variables of width, slope, and depth from the independent variables of discharge and sediment inflow.

Step 4 Determine channel width—A channel top width for the channel-forming discharge is selected from the stability curve using geomorphic principles or project constraints. Analogy methods, hydraulic geometry curves, or the extremal hypothesis are geomorphic relations that can be used to select width. Depth and slope for the selected width are determined from the stability curve.

Step 5 Conduct an analytical sediment budget analysis—Using the design channel dimensions, calculate a sediment-transport rating curve in the project reach. Using a flow-duration curve that includes some high flood discharges, calculate sediment yield into and out of the project reach. More information is provided in NEH654.13.

Step 6 Determine channel planform—Sinuosity is determined from the calculated channel slope and valley slope. Remaining planform design parameters include the meander wavelength, an appropriate channel length for one meander wave-

length, and the trace of the channel. These can be determined from analogy methods, hydraulic geometry relations, or analytical techniques that assume minimum expenditure of energy. These techniques are described in NEH654.12.

Step 7 Natural variability in cross-sectional shape—Variability in channel width and depth can either be allowed to develop naturally or can be part of the project design. Sand-bed streams have the ability to create natural variability in channel form rather quickly because they are characterized by significant bed-material sediment transport. Gravel-bed streams typically adjust much more slowly. Streams with very little bed-material movement may not adjust at all. If variability is to be included in the project design, dimensions for cross sections in riffles and pools can be obtained from stable reaches of the existing stream or from reference reaches. Thorne (1988) has provided morphologic relationships for channel width for a meandering sand-bed river. Other researchers have correlated variability to riparian and bank conditions. Analogy methods have also been used in the design of variability. Techniques for design of variability in cross-sectional shape are described in NEH654.12.

Step 8 Instream structures—Successful stream restoration often includes the use of bank protection, grade control, and habitat features. To restore a stream with physical habitat features resembling a natural stream, a combined technology approach is required. Sound physical principles and well established engineering formulas are used in the analysis and design of both soft and hard features. Systems composed of living plant materials are often used in association with inert materials, such as wood or rock, and manufactured products. A significant flood event (normally no smaller than the 10-year frequency discharge) is used to size structures and compute scour depths. In addition, the quantity of water and its related hydroperiod largely determines what type of vegetation will grow in an area. The flexibility of these features depends on the project goals, tolerance for project change, and consequences of failure. Consideration is given to the effects that proposed features could have on flooding. For example, vegetation often increases boundary roughness, decreasing velocities, and increasing flood profiles. Additional design considerations include the level of risk that is acceptable, natural system dynamics, anthropogenic activities in the watershed, the construction time frame, existing infrastructure, and desired speed of improvement, cost, and maintenance. Guidelines for design of instream structures are described in NEH654.14.

Example 2: Stable channel analytical method

Objective: Determine stable channel dimensions for a diversion channel. Upstream natural stream is coming out of a hillside watershed.

Given: Dimensions of the upstream natural channel reach are:

Base width = 22 ft

Side slopes

Left bank = 2.2H:1V

Right bank = 1.1H:1V

Side slope roughness coefficient = 0.07

Channel slope = 0.0025

Bed material — sandy gravel

D_{84} = 22 mm D_{50} = 3.7 mm

D_{16} = 0.43 mm

Channel-forming discharge = 2,500 ft^3/s

Design values for the bypass channel:

Side slopes = 3H:1V

Side slope roughness coefficient = 0.045

Valley slope = 0.0020 (maximum design slope)

Solution: Solve the Brownlie resistance and sediment transport equations using the USACE SAM (Thomas, Copeland, and McComas 2003) or another program or spreadsheet. Example output is shown in figure 9–16. The sand-bed equations are chosen because bedforms may occur at the channel-forming discharge. However, the bed gradation is borderline between sand and gravel, and it would be prudent to make computations using both sand and gravel equations. From this table, stability curves for slope and depth as a function of depth can be platted (figs. 9–17 and 9–18).

The stability curves provide a family of solutions for width, depth, and slope that satisfy the resistance and sediment transport equations. Any combination of solutions on these curves will theoretically be stable in terms of aggradation and degradation. If the extremal hypothesis is adopted, a unique solution is provided. In this case:

Base width = 67 ft
Depth = 6.7 ft
Slope = 0.001879

If a straight channel is desired, then the channel slope would be set equal to the valley slope, 0.0020 (from the stability curves, base width = 38 ft, depth = 8.4 ft).

If a sinuous meandering channel is desired, then the maximum sinuosity for a stable channel can be calculated by dividing the valley slope by the calculated slope at minimum stream power.

$$\frac{0.0020}{0.001879} = 1.06$$

The stable channel dimensions for base width and depth are values calculated at minimum stream power. Any additional sinuosity would result in an aggrading stream. Thus, the only stable solutions occur with sinuosities between 1.0 and 1.06.

If one objective of this channel is flood control then it is best to design a compound channel to achieve both maximum channel stability and flood control benefits. The low-flow channel would be designed based on stability concepts for the channel-forming discharge, while the width and depth of the overflow channels would be based on normal depth or backwater calculations in a compound channel for the design flood (NEH654.06).

The design channel should be checked for the full range of expected natural flow conditions. A sediment budget analysis should be conducted to determine if there will be long-term aggradation or degradation in the channel. A hydraulic analysis should also be conducted at a design flood flow to obtain critical velocities for design on in channel structures and bank protection. It may be necessary to revise the initial design and iterate on a final solution that meets additional project constraints.

Example 2: Stable channel analytical method—Continued

Figure 9–16 Sample output from USACE SAM

```
*******************************************************************************
*          SAMwin Software Registered to the US Army Corps of Engineers       *
*******************************************************************************
*                             HYDRAULIC CALCULATIONS                          *
*                                  Version 1.0                                *
*           A Product of the Flood Control Channels Research Program          *
* Coastal & Hydraulics Laboratory, USAE Engineer Research & Development Center *
*                              in cooperation with                            *
*                Owen Ayres & Associates, Inc., Ft. Collins, CO               *
*******************************************************************************
CALCULATE CHANNEL WIDTH, DEPTH AND SLOPE BY COPELAND METHOD.
CALCULATE INFLOWING SEDIMENT CONCENTRATION, PPM.

 INFLOWING WATER DISCHARGE, CFS =  2500.000
 BASE WIDTH                     =    22.00000
 CHANNEL SLOPE, FT/FT           =     0.00250000

               LEFT BANK           RIGHT BANK
 SIDE SLOPE =   2.200                 1.100
 n-VALUE    =   0.07000               0.07000

 CALCULATE STABLE CHANNEL DIMENSIONS.
 USING BROWNLIE'S RESISTANCE & TRANSPORT EQUATIONS

 MEDIAN BED SIZE ON BED, MM = 3.64849
 GRADATION COEFFICIENT  = 9.950
 VALLEY SLOPE    = 0.00200000

               LEFT                BANK RIGHT BANK
 SIDE SLOPE =   3.000              3.000
 n-VALUE    =   0.04500            0.04500

 STABLE CHANNELS FOR Q=2500.0, C mgl=210.8, D50=3.648
```

Example 2: Stable channel analytical method—Continued

Figure 9–16 Sample output from USACE SAM—Continued

	BOTTOM WIDTH	DEPTH	ENERGY SLOPE	CMPOSIT n-Value	HYD RADIUS	VEL	FROUDE NUMBER	SHEAR STRESS	BED REGIME*
	FT	FT	FT/FT		FT	FPS		#/SF	
1	5.	10.4	0.004550	0.0456	5.30	6.69	0.37	2.94	LO
2	10.	10.4	0.003240	0.0458	5.64	5.86	0.32	2.10	LO
3	15.	10.1	0.002711	0.0459	5.81	5.45	0.30	1.71	LO
4	20.	9.8	0.002422	0.0460	5.89	5.19	0.29	1.48	LO
5	25.	9.4	0.002241	0.0460	5.92	5.00	0.29	1.31	LO
6	30.	9.0	0.002121	0.0461	5.91	4.86	0.29	1.19	LO
7	35.	8.6	0.002039	0.0461	5.88	4.74	0.28	1.10	LO
8	40.	8.3	0.001981	0.0461	5.82	4.65	0.28	1.03	LO
9	45.	8.0	0.001940	0.0461	5.75	4.56	0.29	0.96	LO
10	50.	7.6	0.001913	0.0460	5.67	4.49	0.29	0.91	LO
11	55.	7.3	0.001895	0.0460	5.57	4.42	0.29	0.87	LO
12	60.	7.1	0.001884	0.0460	5.48	4.36	0.29	0.83	LO
13	65.	6.8	0.001880	0.0459	5.38	4.31	0.29	0.80	LO
14	70.	6.6	0.001880	0.0459	5.27	4.26	0.29	0.77	LO
15	75.	6.3	0.001883	0.0458	5.17	4.21	0.30	0.74	LO
16	80.	6.1	0.001890	0.0457	5.06	4.16	0.30	0.72	LO
17	85.	5.9	0.001899	0.0457	4.96	4.12	0.30	0.70	LO
18	90.	5.7	0.001911	0.0456	4.85	4.08	0.30	0.68	LO
19	95.	5.5	0.001924	0.0456	4.75	4.04	0.30	0.66	LO
20	100.	5.4	0.001939	0.0455	4.65	4.01	0.30	0.65	LO
	RESULTS AT MINIMUM STREAM POWER								
21	67.	6.7	0.001879	0.0459	5.32	4.28	0.29	0.78	LO

* REGIMES: LO=LOWER, TL=TRANSITIONAL-LOWER, TU=TRANSITIONAL-UPPER, UP=UPPER

Example 2: Stable channel analytical method—Continued

Figure 9–17 Stability curve slope versus base width, Q = 2,500 ft^3/s, bed-material sediment concentration = 211 mg/L. Brownlie resistance and sediment transport equations

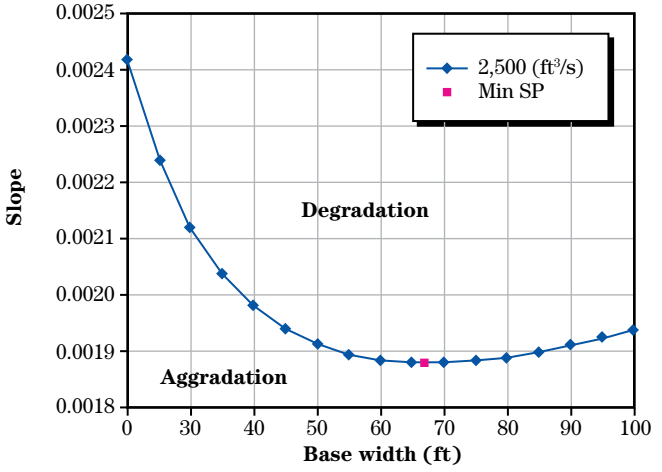
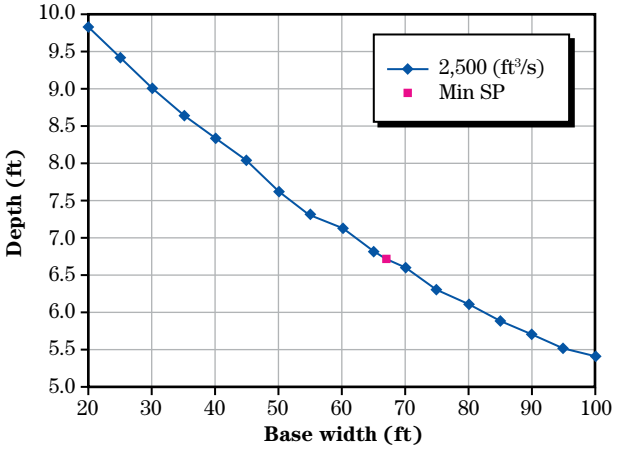


Figure 9–18 Stability curve depth versus base width, Q = 2,500 ft^3/s, bed-material sediment concentration = 211 mg/L. Brownlie resistance and sediment transport equations



Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method

Objective: Determine stable channel dimensions for single thread meandering channel that maximizes habitat benefits on an existing braided alluvial fan. The upstream natural stream comes out of a hillside watershed. The project reach is an alluvial fan with a braided channel that flows into a larger river downstream from the project reach. Note that the braided alluvial fan may be a naturally stable channel, but due to the wide shallow flow, water temperature is too high for certain fish species. Cross-sectional variability is negligible due to the lack of pools and riffles.

Given: Dimensions of the upstream natural channel reach are:

Base width = 55 ft
Side slopes
Left bank = 1.5H:1V
Right bank = 1.5H:1V
Side slope roughness coefficient = 0.08

Use Cowan method and add 0.01 to upstream channel roughness to account for channel irregularity.

Channel slope = 0.0065
Bed material—gravel
D_{84} = 19.7 mm
D_{50} = 6.9 mm
D_{16} = 0.76 mm
Channel-forming discharge = 1,500 ft^3/s

Design values for the single-thread channel:

Side slopes = 1V:2.5H
Side slope roughness coefficient = 0.05
Use Cowan method and add 0.005 to account for channel irregularity
Valley slope = 0.0055 (maximum design slope)

Solution: Solve the Limerinos resistance and Meyer-Peter and Müller sediment transport equations using the USACE SAM (Thomas, Copeland, and McComas 2003), HEC–RAS, or another program or spreadsheet. Example output is shown in figure 9–19. The gravel-bed equations are chosen because bedforms are not expected to be a factor. From this table, stability curves for slope and depth as a function of depth can be plotted (figs. 9–20 and 9–21).

The stability curves provide a family of solutions for width, depth, and slope that satisfy the resistance and sediment transport equations. Any combination of solutions on these curves will theoretically be stable in terms of aggradation and degradation. Note that a wide range of width solutions will satisfy sediment continuity requirements with a slope of about 0.0045. Selecting a design slope in this range will provide for a stable channel. If the extremal hypothesis is adopted, a unique solution is provided. In this case:

Base width = 150 ft
Depth = 1.9 ft
Slope = 0.004488

These channel dimensions also provide the maximum sinuosity with a stable channel. The sinuosity is calculated by dividing the valley slope by the calculated slope at minimum stream power. In this case:

$$\text{Sinuosity} = \frac{0.0055}{0.004488} = 1.23$$

Any additional sinuosity would result in an aggrading stream. Thus, the only stable solutions occur with sinuosities between 1.0 and 1.23.

If a straight channel is desired, then the channel slope would be set equal to the valley slope, 0.0055. Base width and depth, at a slope of 0.0055, can be read from the stability curves:

Base width = 60 ft
Depth = 3.4 ft

Hydraulic geometry relationships may be used to select an appropriate width. Ideally, a hydraulic geometry relationship could be developed from the study watershed or a regional hydraulic geometry relationship from physiographically similar watersheds might be available. Lacking one of these, figure 9–9, developed from North American gravel-bed rivers, can be used. Converting 1,500 cubic feet per second to 42.5 cubic meters per second, a bankfull width of 24 meters (79 ft) is obtained from the mean regression line. The hydraulic geometry relationship refers to top width, while the stable channel analytical method in the USACE SAM calculates base width. Figure 9–22 is a top width versus base width curve developed from

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method—Continued

the SAM output for the design channel which has side slopes of 1V:2.5H. A top width of 79 feet corresponds to a base width of 62 feet. Going back to the stability curve, this width would require a channel slope of about 0.0055. This is equal to the valley slope and, therefore, would be a straight channel. Using the maximum 90 percent single response limit from figure 9–9, a bankfull width of 43.5 meters or 142 feet is calculated. This corresponds to a base width of 132 feet and a stable channel slope of 0.0045. The design base width should be between 62 feet and 132 feet to satisfy both hydraulic geometry relationships and sediment continuity requirements. Decreasing the width provides for greater depths and more shade from trees on the banks, but it also decreases the sinuosity and channel variability that accompanies meandering. The following mean channel dimensions would be appropriate:

Base width	=	80 ft
Slope	=	0.005
Depth	=	2.9 ft
Sinuosity	=	1.1

The analogy method is another means of selecting an appropriate channel width. The reference reach used for the analogy method must be from a physiographically similar watershed. In this case, the upstream channel is not appropriate because the channel slope is significantly different from the slope in the design reach. The reference reach would need to be from a watershed with a similar sized drainage area that originates in the hills and flows onto an alluvial plain similar to the project reach. The key stability factor here is the abrupt change in slope between the upland stream and the alluvial fan stream. The reference reach would need to be stable and should have the favorable habitat characteristics desired in the project reach.

Other possible criteria for selecting the channel width could be constrained rights-of-way or minimum flow depths for habitat preservation. Minimum flow depths for a specified percent exceedance discharge can be determined by calculating normal depth for the proposed width.

The design channel should be checked for the full range of expected natural flow conditions. A sediment budget analysis should be conducted to determine if there will be long-term aggradation or degradation in the channel. A hydraulic analysis at a design flood flow should also be conducted to obtain critical velocities for design on inchannel structures and bank protection if necessary. It may be necessary to revise the initial design and iterate on a final solution that meets additional project constraints.

This example provides average channel dimensions of width, depth, and slope for the project channel. The planform layout is the next design parameter and is described in NEH654.12. Channel variability (riffles and pools) is also addressed in this chapter.

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method—Continued

Figure 9–19 Sample output from USACE SAM

```
*****************************************************************************
      SAMwin Software Registered to the US Army Corps of Engineers          *
*****************************************************************************
*                        HYDRAULIC CALCULATIONS                             *
*                              Version 1.0                                  *
*       A Product of the Flood Control Channels Research Program            *
* Coastal & Hydraulics Laboratory, USAE Engineer Research & Development     *
* Center *in cooperation with Owen Ayres & Associates, Inc., Ft. Collins, CO *
*****************************************************************************
***************************
 CALCULATE CHANNEL WIDTH, DEPTH AND SLOPE BY COPELAND METHOD.
 CALCULATE INFLOWING SEDIMENT CONCENTRATION, PPM.

 INFLOWING WATER DISCHARGE, CFS  =  1500.000
 BASE WIDTH, FT                 = 55.00000
 CHANNEL SLOPE, FT/FT           = 0.00650000

               LEFT BANK     RIGHT BANK
 SIDE SLOPE   = 1.500         1.500
 n-VALUE      = 0.08000       0.08000

 CALCULATE STABLE CHANNEL DIMENSIONS.
 USING MEYER-PETER-MULLER & LIMERINOS EQUATIONS

 MEDIAN BED SIZE ON BED, MM    =  6.87789
 GRADATION COEFFICIENT         =  5.971
 VALLEY SLOPE                  =  0.00550000

               LEFT BANK     RIGHT BANK
 SIDE SLOPE   = 2.500         2.500
 n-VALUE      = 0.05000       0.05000

 STABLE CHANNELS FOR Q=1500.0, C,mgL=1917., D50=6.878mm
```

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method—Continued

Figure 9–19 Sample output from USACE SAM—Continued

	BOTTOM WIDTH	DEPTH	ENERGY SLOPE	CMPOSIT n-Value	HYD RADIUS	VEL	FROUDE NUMBER	SHEAR STRESS
1	10.	6.0	0.020654	0.0494	3.54	10.03	0.72	7.71
2	20.	5.3	0.011705	0.0450	3.64	8.45	0.65	3.89
3	30.	4.7	0.008537	0.0416	3.54	7.67	0.62	2.49
4	40.	4.2	0.006974	0.0389	3.36	7.16	0.62	1.81
5	50.	3.7	0.006084	0.0368	3.16	6.77	0.62	1.42
6	60.	3.4	0.005534	0.0353	2.97	6.46	0.62	1.17
7	70.	3.1	0.005177	0.0341	2.79	6.20	0.62	1.00
8	80.	2.9	0.004938	0.0332	2.63	5.98	0.62	0.89
9	90.	2.7	0.004775	0.0325	2.48	5.79	0.62	0.80
10	100.	2.5	0.004664	0.0320	2.35	5.61	0.62	0.73
11	110.	2.4	0.004588	0.0316	2.24	5.46	0.63	0.68
12	120.	2.2	0.004539	0.0312	2.13	5.32	0.63	0.64
13	130.	2.1	0.004509	0.0309	2.04	5.19	0.63	0.60
14	140.	2.0	0.004493	0.0307	1.96	5.08	0.63	0.57
15	150.	1.9	0.004488	0.0305	1.88	4.97	0.63	0.55
16	160.	1.9	0.004492	0.0304	1.81	4.87	0.63	0.52
17	170.	1.8	0.004503	0.0302	1.75	4.78	0.63	0.50
18	180.	1.7	0.004519	0.0301	1.69	4.70	0.63	0.49
19	190.	1.7	0.004539	0.0300	1.63	4.62	0.63	0.47
20	200.	1.6	0.004563	0.0300	1.58	4.55	0.63	0.46
	RESULTS AT MINIMUM STREAM POWER							
21	150.	1.9	0.004488	0.0305	1.88	4.97	0.63	0.54

Example 3: Hydraulic design with hydraulic geometry and the stable channel analytical method—Continued

Figure 9–20 Stability curve slope versus base width, Q = 1,500 ft^3/s, bed-material sediment concentration = 1,917 mg/L. Limerinos resistance and Meyer-Peter and Müller sediment transport equations

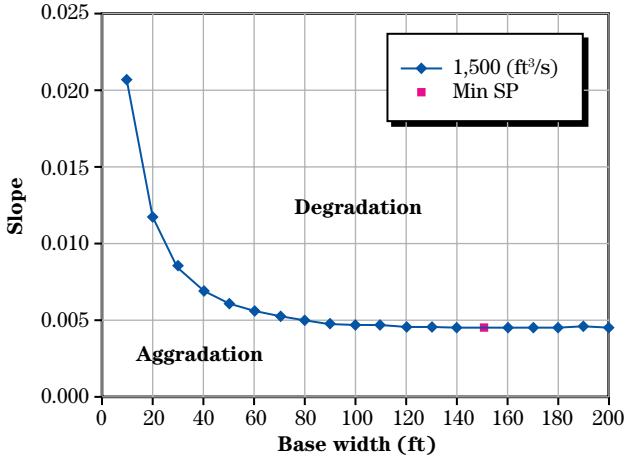


Figure 9–21 Stability curve depth versus base width, Q = 1,500 ft^3/s, bed-material sediment concentration = 1,917 mg/L. Limerinos resistance and Meyer-Peter and Müller sediment transport equations

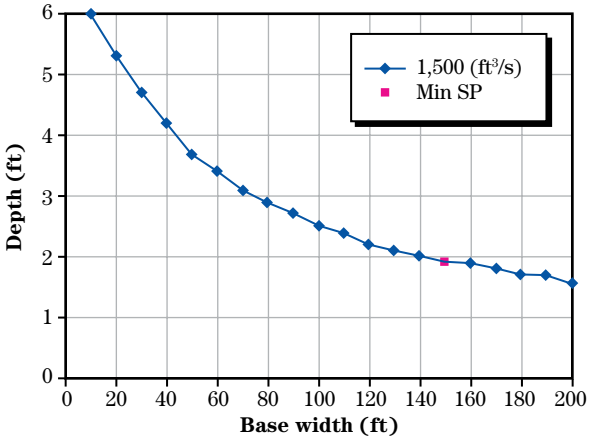
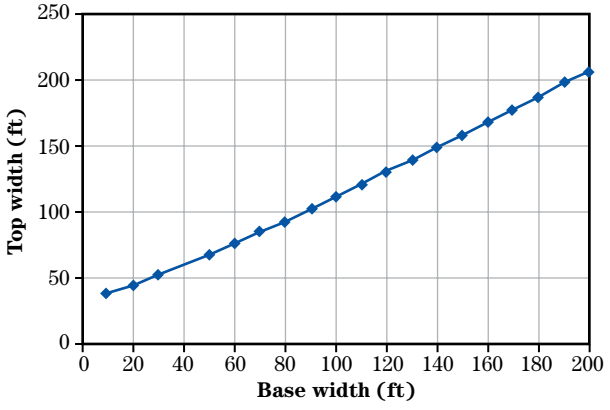


Figure 9–22 Top width verses base width for example problem which has 1V:2.5H side slopes



654.0911 Conclusion

Channels in which there is expected to be an exchange of the inflowing sediment load with the channel boundary should be designed using alluvial design methods. The design goal in an alluvial channel is to pass the inflowing sediment load without significant aggradation, degradation or planform change. Several techniques are available for the design of channels in an alluvial environment. They are the regime method, the analogy method, hydraulic geometry method, and the analytical method. All of these techniques have both advantages and disadvantages.

The analogy method is used to select design elements that are based on the premise that conditions in a reference reach with similar characteristics can be copied to the project reach. The hydraulic geometry method is similar to the analogy method insofar as that it is based on the premise that a river system tends to develop in a predictable way. The theory typically relates a dependent variable, such as width or slope, to an independent or driving variable, such as channel-forming discharge or drainage area. The regime method is similar to the hydraulic geometry, but is more appropriate for canal or drainage ditch type systems. The analytical method uses bed resistance and sediment transport equations to approximate a family of curves for width, slope, and depth for a range of potential stable configurations. These can be used indirectly with project constraints or in conjunction with the analogy or hydraulic geometry methods to estimate critical design elements.

All of the methods presented have advantages and disadvantages. Due to the high degree of uncertainty which is inherent to the nature of alluvial channels, many designers opt to use several methods. For example, during the assessment and design of proposed realignment, the family of curves calculated with the aforementioned analytical techniques can be used to provide another line of evidence which may give the designers more confidence in the chosen section, profile and planform.

All alluvial channel designs require analysis of channel stability. A stream is defined as stable when it has the ability to pass the incoming sediment load without significant degradation or aggradation and when its width, depth, and slope are fairly consistent over time. For design in an alluvial channel, it is suggested that an analytical sediment budget/assessment be conducted to compare the supply capabilities of the upstream reach to the sediment transport capacity of the design reach. Since bed-material sediment transport is significant under flows below, at, and above design flow in an alluvial channel, a sediment assessment should be done for a range of flows in any proposed realignment. Preparing sediment budgets is presented in NEH654.13.

United States
Department of
Agriculture

**Natural
Resources
Conservation
Service**

Part 654 Stream Restoration Design National Engineering Handbook

Chapter 10 Two-Stage Channel Design

Issued August 2007

Cover photo: Low gradient, nonincising channels and ditches may be modified by creating a narrow flood plain, thereby creating some ecological benefits, while minimizing the need for maintenance (clean-outs).

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 10 Two-Stage Channel Design

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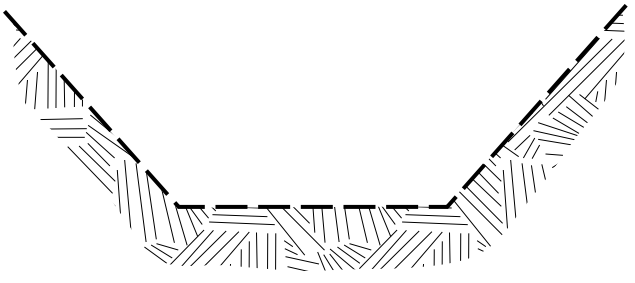
Chapter 10 Two-Stage Channel Design

654.1000 Purpose

Constructed channels are part of extensive portions of productive agricultural land in the United States. These channels provide important drainage and flood control functions. However, these agricultural channels are often constructed as traditional trapezoidal ditches using threshold design techniques. While this approach is suitable in some areas, channels of this design can require frequent and expensive maintenance in other parts of the country. In addition, natural ecological functions are normally not a consideration in the design of these channels. This chapter presents an alternative design to the conventional trapezoidal drainage channel. This two-stage channel system incorporates benches that function as flood plains and attempts to restore or create some natural alluvial channel processes. However, these two-stage channels are not an exact copy of natural streams, as the width of the benches is often small due to the confining geometry of the constructed channel. This chapter outlines measurement and analysis procedures that can be used to size two-stage channel systems that are more self-sustaining than conventional one-stage constructed channels. Although this chapter focuses primarily on an alternative design for constructed ditches, the technique may also have application in natural streams that have undergone incision or in streams where boundary constraints restrict restoration designs such as in urban or developed areas. A case study is also presented for a constructed two-stage ditch in Michigan.

This two-stage channel design approach is applicable to low gradient ditches and channels that are not undergoing incision.

Figure 10–1 Trapezoidal cross section of a constructed drainage ditch



654.1001 Introduction

Agricultural ditches and channels have long been used to provide important drainage and flood control. Historically, many of these drainageways are designed following threshold design techniques and result in a large, trapezoidal cross section. The primary purpose of the constructed channel is to convey water from agricultural fields.

Figure 10–1 illustrates the basic design configuration for a trapezoidal channel, and figure 10–2 is an example of one in Iowa. In many situations, the waterway does behave as a threshold channel, so this is an appropriate approach. However, when the waterway behaves as an alluvial channel, the ditch can be too entrenched and have overwidened bed widths. While the large section of a traditional agricultural drainage channel may provide sufficient flood conveyance, the more frequent discharges may not flow at a depth and velocity sufficient to move sediment through the reach. Deposition results, requiring maintenance to maintain the design flow capacity. As deposition occurs, bank stability may also become an issue as sediment deposits may force flows into one bank or the other. In addition, baseflows in this wide channel may have a depth which does not provide adequate aquatic habitat.

Figure 10–2 Drainage ditch constructed in north-central IA

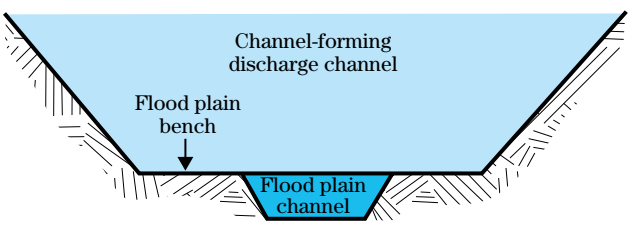
Fluvial processes at work in agricultural ditches functioning as alluvial channel systems often try to develop a flood plain that consists of low benches. While this deposition reduces flood capacity, these ditches show improved stability and improved habitat quality. This chapter presents a two-stage approach to design stable agricultural drainage channels (fig. 10–3). Specifically, the two stages are the:

- dominant discharge or channel-forming discharge channel
- flood plain bench or flood plain channel

This two-stage approach provides improved physical, as well as ecological performance. The channel-forming discharge channel provides the necessary sediment conveyance, while the flood plain channel provides for the design flood conveyance. By nesting the channel-forming discharge channel within the larger channel, the entire waterway is more stable.

The technique described herein uses bankfull discharge as representative of the channel-forming discharge. Therefore, the channel-forming discharge channel is referred to as the bankfull channel. The bankfull channel has also been referred to as the effective discharge channel. However, this is not necessarily accurate. There are no calculations made to define the effective discharge. Rather, this lower stage is assumed to be the bankfull discharge of a low-flow channel formed in a typical constructed ditch. The distinctions between bankfull, channel-forming, and effective discharge are addressed in more detail in NEH654.05. The differences between alluvial and threshold channels are addressed in NEH654.07.

Figure 10–3 Conceptual design for two-stage channel system



654.1002 Background

Highly modified channels drain extensive portions of productive agricultural land in the United States. Headwaters are typically the most modified. In some areas, virtually all of the natural channels have been deepened and straightened to facilitate the flow of water from agricultural subsurface drainage outlets and to maximize water conveyance. Work is done periodically to maintain the drainage function, which typically includes removal of woody vegetation, weeds, and deposited sediment. Ancillary work includes stabilizing bank slope failures and toe scour. Ditch form is a result of not only construction and maintenance, but also to verifying degrees, due to fluvial (flowing water) processes.

Ditch maintenance typically restores the ditch to a trapezoidal shape designed to transport large storm events (fig. 10–4). To facilitate drainage and reduce the frequency of over bank flows, trapezoidal ditches are designed to accommodate large flows (5- to 100+- year recurrence interval) within the ditch. Also the width of the ditch bottom is constructed wider than the channel bottom that would form by fluvial processes, thus, making the channel relatively wide and shallow. Therefore, the constructed ditch channel is often oversized for small flows and provides no flood plain for large flows.

In contrast to trapezoidal agricultural drainage ditches, integral parts of many natural stream channels are the flood plains. The flood plains of natural streams (except for those with steep bed slopes) are characterized by frequent, extensive over-bank flow. In dynamic equilibrium, a stream system depends on both the ability of the flood plain to dissipate the energy of high flows and to concentrate the energy of low flows to effectively create a balance in sediment transport, storage, and supply. In natural alluvial streams, fluvial processes work to size and maintain the dimensions of the bankfull channel based on the effective discharge (Ward and Trimble 2004).

In response to the construction of an oversized trapezoidal channel, alluvial channel processes often work to create a small bankfull channel by building a flood plain or bench within the confines of the ditch (fig. 10–4). If conditions allow, these benches can reach

a stable size, thickly vegetated with mostly grasses. This results in a two-stage channel. The small bankfull channel will often meander slightly within the ditch. The bankfull channel will usually have steep (1H:1V) sides and a bed consisting of material coarser than that of adjacent reaches where benches have not formed. Further details on fluvial processes in ditches are available form Landwehr and Rhoads (2003) and Ward, Mecklenburg, and Brown (2002). It is important to note that these deposits within a constructed trapezoidal ditch reduce the overall flood conveyance. As a result, the channel may no longer provide the designed flood protection.

(a) Advantages of a two-stage channel

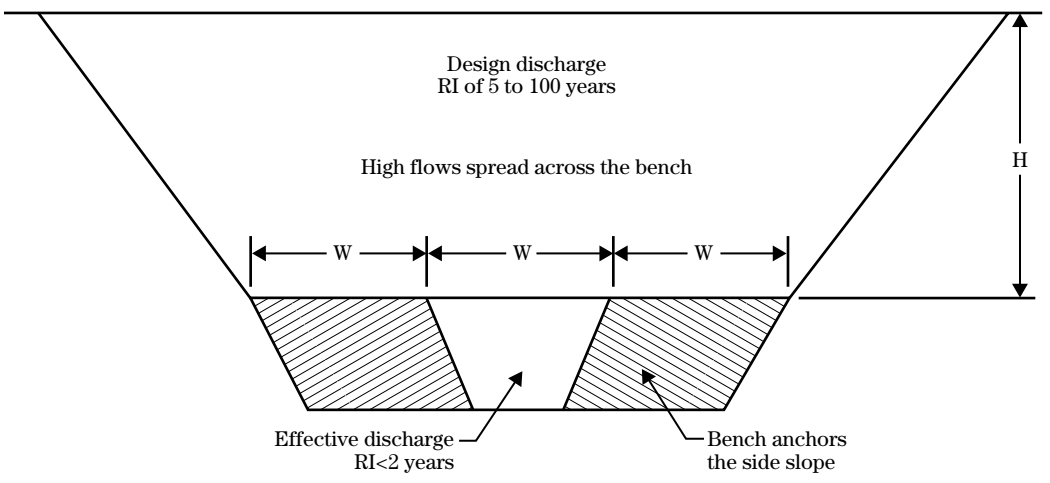
Benefits of a two-stage ditch over a conventional trapezoidal ditch are potentially both improved drainage function and ecological function. Drainage benefits may include increased ditch stability and reduced maintenance. Evidence and theory both suggest that ditches prone to filling with accumulated sediment may require less frequent dipping out if constructed in a two-stage form. Second, channel stability may be improved by a reduction in the erosive potential of larger flows as they are shallower and spread out across the bench (fig. 10–5).

Figure 10–4 Ditch before maintenance and after maintenance (MN)

(a) Before

(b) After

Figure 10–5 Two-stage ditch geometry with minimum sized benches



Stability of the ditch bank should be improved where the toe of the ditch bank meets the bench, rather than the ditch bottom. Here the water depth is effectively reduced, and the shear stress (erosive force) on the toe of the bank is less. Also, not being in contact with low flow, this bank material will be dryer and can be stabilized with vegetation using threshold design techniques as described in NEH654.08. Since a two-stage channel in an alluvial system will be more likely to retain its design shape, it is easier to predict its flood protection performance.

The biggest advantage of these two-stage channels is the ability to transport sediment more effectively. However, the two-stage ditch also has the potential to create and maintain better habitat than a conventional trapezoidal ditch. The narrow, deep bankfull channel provides better water depth during periods of low flow. Grass on the benches can provide some instream cover and shade. The substrate in the bankfull channel is improved as the two-stage form increases sediment conveyance and sorting, with fines deposited on the benches and coarser material forming the bed.

(b) Design of a two-stage channel

Design and construction of two-stage channels is different than that of traditional trapezoidal channels. The design of a two-stage agricultural ditch in an alluvial channel system involves correctly sizing the bankfull channel and minimum bench widths for the flood plain bench. The dimensions of the bankfull discharge or fluvial channel dictate the two-stage channel design. If properly sized, the bankfull channel will be maintained by fluvial processes and will reduce or possibly even eliminate large-scale channel maintenance. The flood plain bench serves as a flood plain for the smaller bankfull channel, but it acts more as a threshold channel. The upper stage must convey the channel-forming discharge and must have an adequate size to prevent design flood flows from overtopping the ditch banks and flooding surrounding land.

654.1003 Field measurements

Initial reconnaissance of the site area is recommended to establish a base knowledge of the project characteristics, surrounding area, and regional environment. The unique characteristics of the project site will generate the criteria for regional measurements. If searching for a reference reach, the watershed area, vegetation, soil, land use, and slope should correspond to the site in question.

Where a modification will be made to an existing channel, detailed measurements should be made of the channel profile and the dimensions of the bankfull channel and benches that have formed within the channel system. Often, the bankfull channel will be overwidened, and the benches will be intermittent and sloping. Guidance on performing such investigations is provided in NEH654.03. Conducting an onsite geomorphology study is a simple and reliable method, but is only adequate if:

- a bankfull channel and benches have formed
- the project length is short; the drainage area is relatively constant

A detailed survey along the reach of interest, or a reference reach, consists of measuring the profile, pattern, and dimension of the channel. The profile is the slope of the bed surface including all pools, riffles, and runs. The undulating elevations of the channel bed leads to questions of the true channel profile. To compensate for the bed slope variability, the water surface is also measured to represent the slope of the channel. The pattern of a reach measures the sinuosity of the bankfull channel. This is obtained using a compass and measuring the azimuth from magnetic north. The dimensions of the channel are obtained by surveying cross sections, either at increments along the reach or at representative cross sections. A laser level and survey tape, or a total station instrument, are often used. The distance from the left channel bank and the change in elevation are measured for each grade break across the channel cross section. A pebble count should be performed to estimate the mean bed particle size. Guidance for performing pebble counts is provided in NEH654 TS13A.

654.1004 Bankfull channel design

The first step in developing a two-stage design is determining the probable dimensions of the bankfull channel. This channel will carry most of the sediment in the channel. The width of the bankfull channel is a key design characteristic. It will determine the success in achieving the intended drainage effects, as well as ecological benefits. Channel design dimensions are determined by measuring the bankfull discharge features or calculating the effective discharge at the project site and then by creating a watershed specific regional curve for the project.

(a) Regional curve development

The probable dimensions of the bankfull channel can be empirically determined based on regional studies similar to those that are conducted for natural streams. Typically, for natural streams this knowledge is acquired by developing regional curves that relate the bankfull channel dimensions to drainage area. Traditional regional curves are created by performing numerous profile and cross-sectional surveys at locations with different drainage areas, which often include U.S. Geological Survey (USGS) stream gage sites as described in NEH654.05 and NEH654.09. The regional sites should be selected to provide fluvial information over a range of drainage areas that can be plotted to show channel dimension relationships to drainage area.

Measurements can be taken at the water surface and the bankfull fluvial features. Each gage station has a unique rating curve, which is a relationship between the gage reading and the streamflow rate. The bankfull height at each gage can be obtained by the measurements at bankfull and the waters surface along with the USGS real-time gage value. However, it is important to note that while each gage station may have a unique rating curve, the relationship between gage height and discharge is not necessarily unique. The rating curve may shift over the long term as the cross-sectional shape and/or elevation changes, and it may shift over the course of a hydrograph due to the unsteady loop effect or changing bedforms. If the rating curve is applicable, these values, combined with the width, will provide an additional point when creating a regional curve. An applicable and complete regional curve can be a valuable tool for two-stage channel design, as well as many other stream design activities.

Care must be taken with the use and development of the regional curves. The data used to develop a curve needs to be from physiographically similar basins. Drainage network patterns and the relative location of the channel site with respect to uplands are significant characteristics. The bed and bank characteristics used in the regional curve development should be the same as those at the project site. Issues related to the development and use of regional hydraulic geometry curves are described in more detail in NEH654.09.

Small watersheds that are drained by agricultural ditches can present particular challenges in the development of traditional regional curve data. In most parts of the Nation, there are a limited number of small-gaged watersheds, and these typically have short records or have been discontinued. Some additional difficulties associated with developing regional curves are that gages are often located at road crossings or the reach within the vicinity of the gage is highly modified.

(b) Rapid regional curve development

For two-stage channel design in many agricultural watersheds, an abbreviated rapid regional curve may be adequate. The method consists of finding ditches or streams with well-developed benches/flood plains and measuring at least the width and depth of the naturally formed bankfull channel. The selected channels must have reached a state of equilibrium and must be stable. Sites for any regional curve should also have similar characteristics to the project site and should come from physiographically similar watersheds. Several measurements should be taken for each range of drainage areas to verify that the measured feature is consistent with those across the watershed. Whenever possible, precision surveying instruments should be used to make the elevation and distance measurements.

Traditional regional curves are created by performing numerous detailed surveys at locations with different drainage areas. In contrast, a rapid regional curve channel dimension measurement consists of quickly

measuring visual fluvial features with a 100-foot tape and a telescoping leveling rod. The channel dimensions taken at a complementary range of several drainage areas should provide a sufficient spread for each log cycle. The bankfull dimensions of width and depth are measured where visible fluvial features are noticed. The drainage area for each measurement is acquired from a variety of methods such as calculating the area by hand using a planimeter or computer GIS software. The measured dimensions can be the plotted area versus drainage area, and a power regression equation can be fitted to the data. This equation can be used to estimate the bankfull channel design dimensions for a ditch, given the drainage area.

This rapid regional curve approach has been used on several watersheds in Ohio. Reportedly, this approach typically provides relationships between the bankfull channel and drainage areas with r^2 values of 0.8 or greater (Ward 2005; Ward et al. 2003).

(c) Reference reach

Measurements from a reference reach can provide valuable design guidance for the design of the bankfull channel. Typically, for natural streams this knowledge is acquired by conducting detailed surveys along a reach of interest and by conducting a detailed survey of a reference reach along the same stream or a similar nearby stream system. However, finding reference reaches can be a time consuming, costly, and frustrating activity. The attributes of the local subwatershed, such as the topography, soil and bedrock properties, vegetation on the banks and adjacent riparian zone, and size and characteristics of the active flood plain, can result in a variety of different stable channel dimensions for similar-sized drainage areas within a watershed or region. For a reference reach to be directly applicable, it must have similar climate, history, drainage area, and watershed conditions. More information on the identification and use of reference reaches is provided in NEH654.09 and 654.12.

654.1005 Flood plain channel design

The formation of benches in constructed ditches is the natural result of fluvial processes in most alluvial systems. The bench acts as a flood plain within the ditch to dissipate energy, reduce the erosive potential of high-flow volumes, and reduce the shear stress on the bank toe. In establishing two-stage geometry, it is often not cost effective or practical to form a flood plain as wide as fluvial processes would form under natural conditions. Large, deep agricultural ditches have often already been constructed to handle discharges from subsurface drainage systems. Making these large ditches even wider would result in extensive earth moving, high cost, and substantial losses in productive agricultural land. Therefore, the ability of these small flood plains (benches) to aid in developing a self-sustaining system is dependent on the establishment of dense grass cover on the benches and banks of the ditch. Also, the side slopes and depths of the ditch above the benches must satisfy geotechnical engineering requirements to provide bank stability.

In a designed two-stage channel, the elevations of the flood plain channel benches are dependent on correctly determining the size of the bankfull channel. The flooded width is defined as the total width across the ditch at the stage elevation where benches have formed and/or are anticipated to form. The two-stage width ratio is defined as the flooded width divided by the top width of the bankfull channel. Based on visual observations and modeling bed-load transport, two rules of thumb have been established:

- If the total width, when out-of-channel flow is initiated, is less than three times the top width of the bankfull channel, the benches might not fully develop, the benches are more likely to be unstable, and shear stresses on the bed and banks of the ditch will be high during large events.
- If the total width, when out-of-channel flow is initiated, is more than five times the top width of the bankfull channel, the channel will begin to exhibit a natural meander pattern that, at places in the ditch, is likely to cut into the banks of the ditch.

Therefore, when out-of-channel flow is initiated, the designed target total width should be between three to five channel widths (total bench sizes that are two to four times the channel width), if the objective is to provide adequate conveyance capacity and a more self-sustaining system, while maintaining a relatively straight ditch geometry. If the project goals or the stability requirements of the channel design require the development of a sinuous channel, a wider bench may be required. However, channel alignment design elements, such as are described in NEH654.12, will need to be addressed so that a stable planform is chosen.

654.1006 Flood conveyance

The overall conveyance capacities of the two-stage systems can be sized based on the probability of out-of-ditch flooding into adjacent areas. This probability is based on the recurrence interval storm event that the entire ditch can transport. Where possible, stream gage data should be used to determine the discharges associated with a prescribed recurrence interval. However, in most parts of the Nation, there are limited numbers of small watershed gages. Typically, these gages have short records or have been discontinued. Therefore, measured streamflows at locations without gages are determined from hydrologic models or from regional discharge curves. One source of regional discharge information is the National Flood Frequency (NFF) Program. The NFF Program includes 2,065 regression equations for 289 flood regions nationwide. These equations are contained in a Windows® program for estimating the magnitude and frequency of peak discharges for unregulated rural and urban watersheds. This program can be obtained at the following Web site:

http://water.usgs.gov/software/

Since most two-stage channels are of fairly uniform section and constant slope, a resistance equation, such as Manning's equation, can be used to calculate the depth corresponding to the design flow recurrence intervals.

654.1007 Spreadsheet tools for data analysis and design

Many of the calculations for two-stage channel assessment and design can be performed with the help of computer spreadsheets. One set has been developed by the Ohio Department of Natural Resources (ODNR). This suite of spreadsheet tools can be obtained from the following Web site:

http://www.ohiodnr.com/soilandwater/streammorphology.htm

These spreadsheets aid in designing a new bankfull channel together with various size benches, based on the following:

- a regional curve for the area
- cross-sectional data for a ditch or channel reach
- profile data for the reach that can include bed, water elevation, bench, and top of ditch data
- the D_{50} fraction of the bed material
- user-defined adjustments to the channel, bench, and ditch geometry

The channel width, depth, and cross-sectional area associated with the bankfull discharge at each location surveyed are entered into the spreadsheet to develop a regional curve. The calculated drainage area and bankfull discharge at each location are also entered into the same spreadsheet. A log-log plot is then made of each bankfull discharge dimension versus drainage area. A least-squares analysis is then used to fit a power regression line (a trend line) through each set of data and calculate the coefficients of the regional curve.

In the spreadsheet, stage-discharge relationships for each site are obtained based on Manning's equation. A separate Manning's n value is used for the bankfull channel and the vegetated benches and banks of the ditch. The roughness of the bed, banks, and benches vary seasonally based on winter conditions, vegetation growth, maintenance, and scour or deposition on these features. Therefore, the approach used only provides a general representation of roughness conditions. User-defined discharge versus recurrence interval data, or estimates based on the USGS Urban Method for Ohio, are used together with Manning's equation to calculate the flow stage associated with each recurrence interval. The ODNR channel design spreadsheet is programmed to obtain coefficients for recurrence intervals of 0.25, 0.5, 1.0, and 1.5 years.

In the spreadsheet, bed-load transport in the bankfull channel is calculated based on the probable discharges that will occur during a 100-year time period, the D_{50} of the bed material, and based on estimates obtained from the Meyer-Peter and Müller bed-load transport equation (Ward and Trimble 2004). Estimates are obtained for the bed load, recurrence interval of the bankfull discharge, and probable stage of the bankfull discharge.

While these spreadsheet tools have been developed to aid in the analysis of stream form and processes, they are best applied to alluvial stream systems that are a function of the bankfull discharge.

(a) Site selection and reconnaissance

Potential sites for regional curve measurements were marked on Michigan, Indiana, and Ohio State Gazetteers published by DeLorme. Sites were selected to provide data for several sized drainage areas within each log cycle. Due to the remoteness of the area from Columbus, Ohio, no preliminary reconnaissance was conducted. At each site, the widths and depths associated with grade breaks were measured using a 100-foot tape and a telescoping surveying rod. A more detailed and accurate survey was conducted at the Hillsdale ditch using a laser level, 100-foot tape, and a telescoping rod with a laser receiver. At that site, cross-sectional information was obtained every 100 feet, and the location of the thalweg was noted. On November 16, just prior to construction of the two-stage geometry, a pebble count was conducted for reaches 600 to 800 feet, 800 to 1,000 feet, and 1,000 to 1,200 feet.

(b) Regional curve

Data were obtained at 14 locations within a 600-square-mile drainage area. A regression analysis of the data (fig. 10–7) indicates that the bankfull discharge dimensions are highly correlated with drainage area.

The poorest correlation is with channel depth, perhaps because some of the channels were associated with streams that were highly connected to the flood plain, while others were associated with grade breaks and small bench formation in ditches. The Hillsdale ditch has a 4.5-square-mile drainage area, and its measured channel dimensions are located almost exactly on the regression lines.

(c) Discharge data

The regional curve analysis was extended to the USGS gage on the St. Joseph River near Newville, Indiana. Streamflow data for this gage was obtained from the following Web site:

http://water.usgs.gov/oh/nwis/rt

At this site, a laser level was used to determine the bankfull discharge and water surface elevation. Real-time gage data were available at the time the survey was performed and downloaded from the Internet. Due to deep flow conditions, it was only possible to estimate the width of the river by making a measurement on the road across the bridge.

(d) Discharge versus recurrence interval

An annual series of peak flow data for the period 1947 through 2002 were available for the gage on the St. Joseph River near Newville, Indiana. The Weibull method (Ward and Trimble 2004) was used to develop a plot of discharge versus recurrence interval data resulting in a high correlation (r^2=0.96) between peak discharge and recurrence interval.

Example 10–1: Hillsdale County case study

The Hillsdale County, Michigan, case study was conducted as part of a demonstration project for The Nature Conservancy, Upper St. Joseph River Project Office, and is funded with a grant from the Great Lakes Commission. The survey was conducted in July 2003, and an existing ditch was modified to a two-stage geometry in November 2003 (fig. 10–6). The project site is located in Hillsdale County, within the St. Joseph Watershed (MI). In 1997, the ditch was cleaned out as part of a maintenance action and in July 2003, had 0.5 to 2.0 feet of sediment deposits on the bed and had formed small intermittent benches.

Figure 10–6 Ditch before construction at location 1,000–1,200 ft, and after widening of small benches at location 1,600–2,100 ft (Hillsdale County, MI)

(a) Before

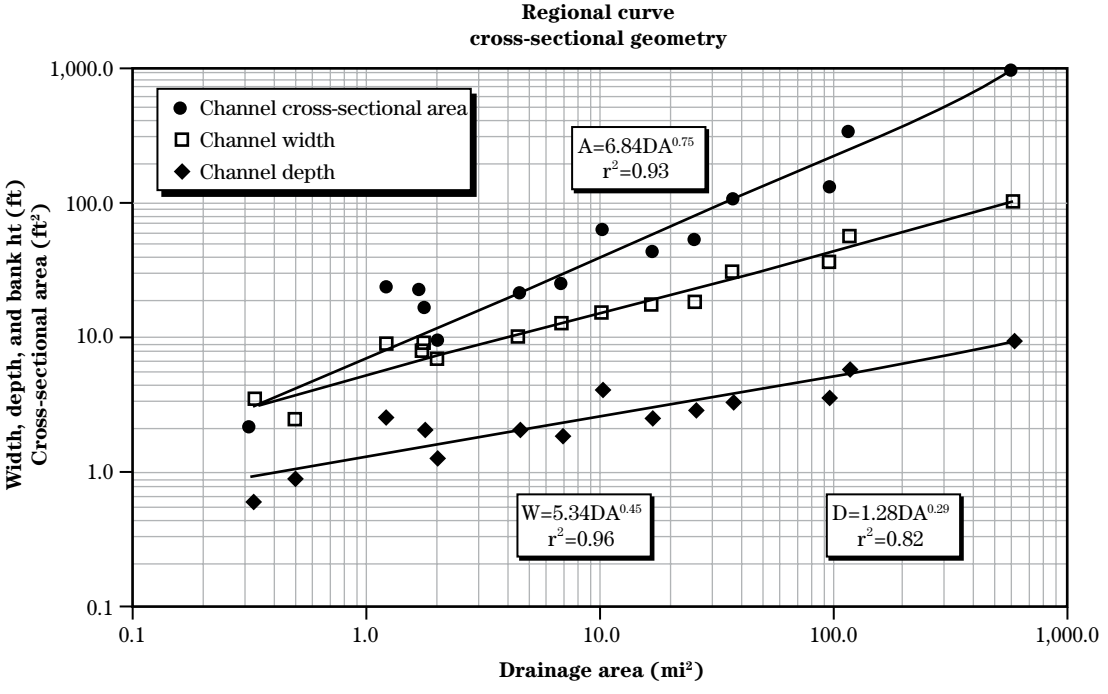
(b) After

On the day of the survey, the water depth at the gage was about 3.7 feet (based on the real-time measurement from the USGS NWIS Web site), and the bankfull discharge dimensions were measured on both banks at about 3.7 to 5.4 feet above the water elevation. The most dominant feature was a continuous approximately 20-foot high bench that was located on the left bank. A shorter bench at a similar elevation was located on the right bank. Therefore, it was estimated that the bankfull stage was 7.4 to 9.1 feet.

An approximate rating curve for the gage was created using data from the USGS Web site. The bankfull discharge was estimated from the survey data, and the rating curve to be between 740 and 1,330 cubic feet per second. While this is a wide range, an analysis of the recurrence interval curve indicates that this corresponds to a recurrence interval that is much more frequent than 1 year. This frequent occurrence is not surprising. In flat, poorly drained areas in the Midwest, where subsurface drainage is widely used, the bankfull discharge occurs frequently and primarily due to subsurface drainage discharges. Since this bankfull discharge is associated with high subsurface flows, it is usually associated with a recurrence interval that is much more frequent that one year (Ward 2005; Ward et al. 2003).

An analysis of the daily flow records shows that discharges within this range or larger occur on average 40 to 80 days annually. This range of flow seems to be too frequent to correspond to the bankfull or channel-forming discharge. However, an analysis of the daily flows exceeding 1,330 cubic feet per second revealed that, on average, they are associated with 1 to 13 discharge events annually, and the duration of these flows ranged from 1 to 49 days. High flows lasting many days typically occurred between November and April. On

Figure 10–7 Bankfull discharge channel dimensions for the St. Joseph River (OH) upstream of the gage near Newville, IN



average, annually, there are slightly more than five events, with an average duration of 8 days that exceeded 1,330 cubic feet per second.

A bankfull channel associated with very frequent flows is consistent with observation by Ward, Mecklenburg, and Brown (2002) in Wood County, Ohio. However, in a recent study, they noted that typically, only about 10 percent of the sediment is transported by flows that are less than double the mean discharge. For most of the gages, less than 25 percent of the sediment load is transported by flows that are 3 to 5 times the mean discharge. (Ward et al. 2003). For example: the mean annual discharge for the St. Joseph gage is about 540 cubic feet per second, so it is probable that a discharge of 1,330 cubic feet per second or higher corresponds to the bankfull discharge at the gage.

At the gage, the river was very entrenched the top of the bank corresponded to a stage of at least 16 to 18 feet (not measured). From further analysis, it was estimated that the out-of-bank discharge is 6,000 to 9,000+ cubic feet per second and corresponds to a 4- to 20-year recurrence interval flow. Therefore, at this location, the behavior of the river is similar to that of a ditch.

The results of discharge versus recurrence interval estimation analysis are presented in table 10–1 for this example. The gage data results were obtained from the regression equation. At the gage, the USGS Rural method gave similar results to the gage data. However, the USGS Urban method greatly overestimated the discharges, even though an annual precipitation of only 32 inches was used, rather than the 34 to 35 inches suggested by the annual precipitation map for Ohio (Ward and Trimble 2004). For the rural equation, a slope of 0.1 percent (5.2 ft/mi) and a storage value of 3 were used. At the Hillsdale ditch, the urban method also gave much higher estimates than the rural equation. However, if the urban method were calibrated based on the ratio of the urban to gage data results, the urban and rural methods gave similar results, except for a recurrence interval of 2 years. It was decided to base the analysis on the rural equation results. For this ditch, knowledge of the actual discharge versus recurrence interval has little influence on the design. The ditch is extremely large, and regardless of what estimates are used, the out-of-bank discharge is associated with a recurrence interval greater than 100 years.

Table 10–1 Discharge vs. recurrence interval results at the gage and the Hillsdale ditch

	St. Joseph gage			Hillsdale ditch		
Recurrence interval (years)	**Gage data**	**USGS Urban method**	**USGS Rural method**	**USGS Urban method**	**Urban calibrated**	**USGS Rural method**
2	4160	4902	5261	176	149	110
5	6337	9735	7365	302	197	171
10	7984	13920	8635	373	214	205
25	10162	22495	10148	546	247	246
50	11809	30121	11141	663	260	274
100	13456	37488	12148	786	282	301

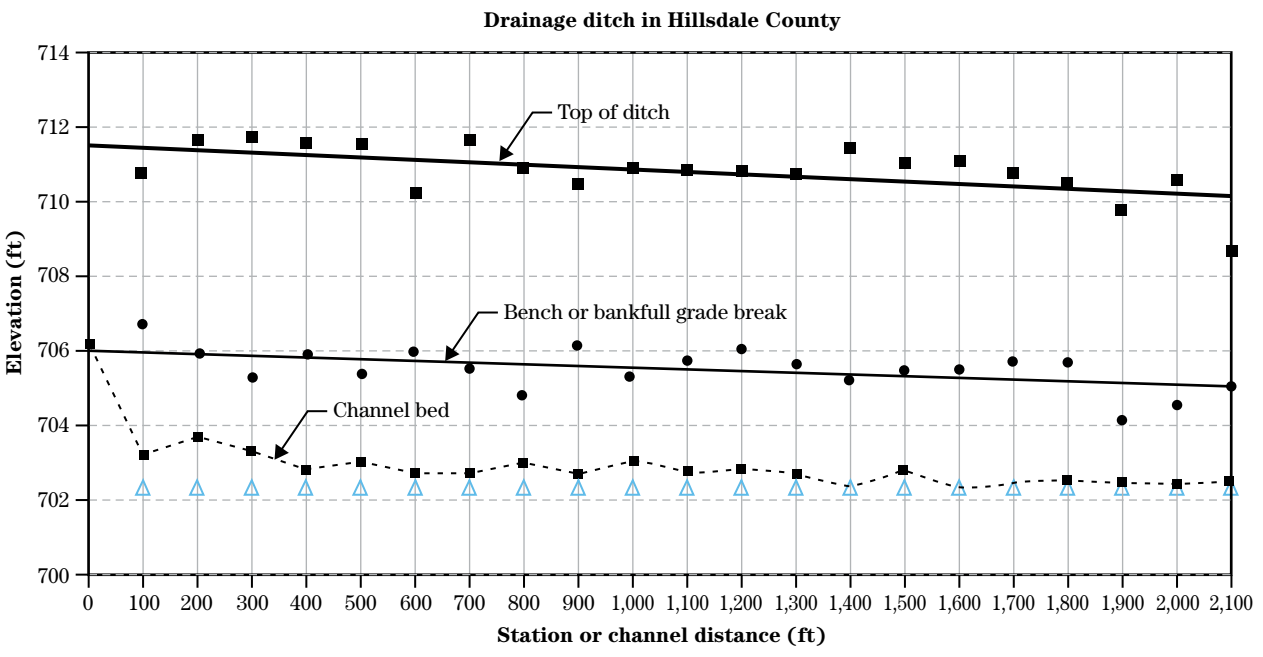
(e) Ditch geometry

A survey of a 2,100-foot length of ditch was performed on July 17, 2003. Station 1+00 is located 100 feet south of the upstream bridge, and station 21+00 is located near the southeast corner of the field on the right bank. This location is close to the Michigan-Ohio state line and the point where the ditch enters a wooded area.

Working conditions in the ditch were difficult because of steep slopes, dense vegetation, and deep deposits of fine sediment in the bottom of the ditch. Therefore, cross-sectional data were obtained by locating a person with a rod and receiver on each side of the ditch and stretching a tape between the two people. Each person took elevation and position data on their side of the ditch and part of the bankfull channel. Notes were made indicating the location of the thalweg in the bankfull channel, and the water depth. Thalweg data were then used in place of conducting a separate profile survey. Also, grade-break data and top-of-bank data were extracted from the cross-sectional data to obtain profiles of these features. Profiles of the various features are shown in figure 10–8. All elevation data are relative to an arbitrary datum. For most of the ditch, the bed slope varied from 0.05 to 0.2 percent.

A typical cross section is shown in figure 10–9, with a possible new design with a 4:1 overtopped width to channel-width ratio. For a 4:1 ratio, the total width of the benches is three times the width of the bankfull channel. The top width of the bankfull channel is 10 feet, the mean depth is 1.8 feet, and the maximum depth is 2.3 feet. Based on an analysis of the data, the maximum stable size of this channel might be 12.3 feet wide, with a mean depth of 2.2 feet, and a maximum depth of 2.8 feet. The channel has a 0.1 percent slope, the channel dimensions estimated from the regional curve (top width of 10 ft and maximum depth of 2.3 ft), and an over bench flow width to channel width ratio of 4:1. The 0.2-year recurrence interval discharge almost fills the small channel, the 1.6-year recurrence interval discharge fills the channel to a depth of about 4.5 feet, and the stage for the 100-year discharge is just over 5.5 feet. For these conditions, fine sediment will be flushed from the bankfull channel, and substrate with a mean size of about 3 to 4 millimeters will be established.

Figure 10–8 Profiles of the bed, bench, and top of ditch



This ditch was cleaned out about 6 years prior to this survey and only exhibited intermittent small bench formations along much of its length. There was up to 2 feet of sediment deposited on the bed in the first few hundred feet, perhaps because of the culvert configuration and a rapid change in bed elevation. Further downstream from the bridge, the bench formations improved, the depth of the sediment deposits decreased, and in places (1,000 to 1,400 ft), clean, coarse substrate was observed in the bottom of the bankfull channel.

(f) Bed material and bed-load transport

The measured D_{50} and D_{84} for reaches 600 to 800 feet, 800 to 1,000 feet, and 1,000 to 1,200 feet were <1 millimeter, <1 millimeter and 12 millimeters, and 3 millimeters and 10 millimeters, respectively. More than 80 percent of the bed material was clay and silt where there were only small intermittent benches (600–800 ft). In the next two reaches, the bench development was more pronounced, and the main channel was narrower, resulting in the coarse substrate sizes. Pebble counts for the last 200 feet (1,000–1,200 ft, fig 10–6a) had the coarsest substrate, widest benches, and narrowest bankfull channel.

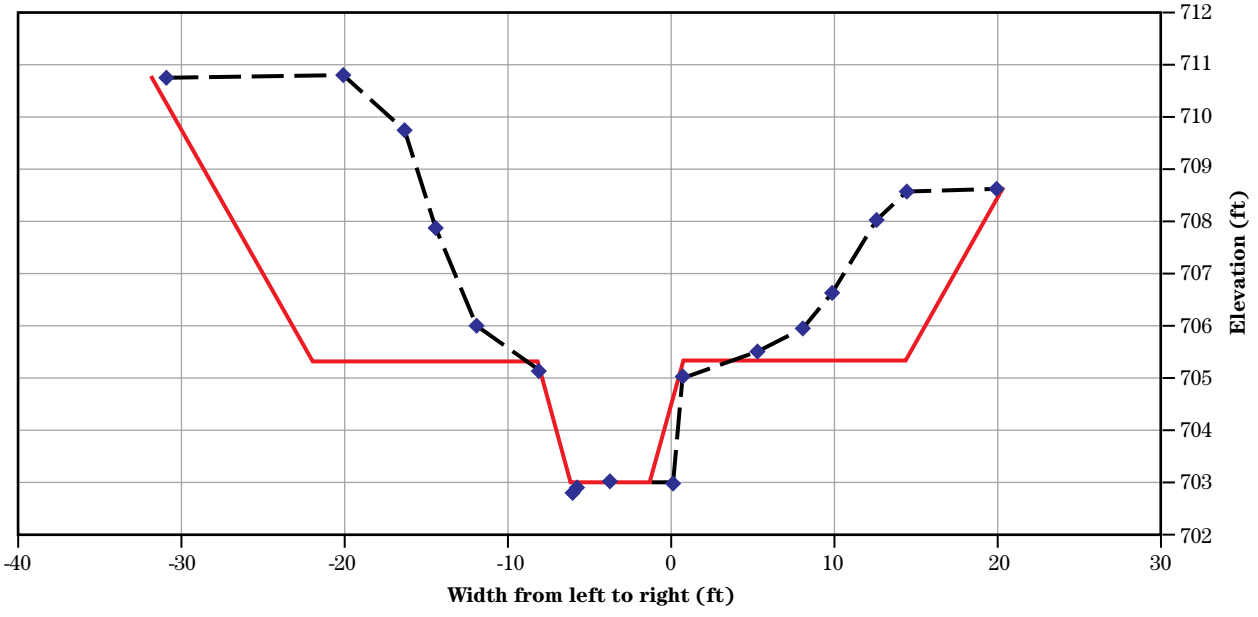
The mean bed-material size is associated with the tractive force (mean shear stress) on the bed and can be estimated as (Ward and Trimble 2004):

$$D_{50} = 1000ds$$

where:
D_{50} = particle size (mm)
d = flow depth (m)
s = bed slope (ft/ft)

Therefore, a 0.6-meter (2.0 ft) bankfull discharge depth in a channel with a bed slope of 0.1 percent might result in a D_{50} of 6 millimeters. The bed slope varies from 0.05 to 0.2 percent, and the bankfull discharge depth is 1.8 to 2.5 feet, so the probable D_{50} is about 3 to 13 millimeters. This is in good agreement with the measured D_{50} and D_{84} of 3 millimeters and 10 millimeters in the channel at reach 1,000 to 1,200 feet, where fluvial benches have formed.

Figure 10–9 Pre- (dashed line) and post- (solid line) maintenance geometries at a location with a grade break, but weak bench formation at the elevation where it was determined that a bench would naturally form. The existing main channel has a similar geometry to the projected geometry.



While this equation is readily applied, it should be noted that it contains some inherent assumptions. Its use assumes that sufficient coarse material is available to form the armor layer. If sufficient coarse material is not available, then this approach may not be advisable.

Actual bed-load transport is difficult to quantify because of the complexity of the system and the lack of any sediment transport data. Relative bed-load estimates were obtained by relating bed-load transport to the current channel conditions: a bed slope of 0.1 percent and a D_{50} of 2 millimeters. The geometry for current conditions was approximated as a cross-sectional area that is three times the area predicted by the regional curve and an over-bench flow width to channel width ratio of 1.5. The results of the bed-load transport analysis results are summarized in table 10–2. It is anticipated that fluvial processes will establish a coarse substrate with a mean particle size of 4 to 8 millimeters, and bed-load transport will be less than half current rates. Following the flushing of deposited fines, and bench building by fluvial processes, the total sediment export will primarily be a function of conservation practices on the landscape and ditch instability problems upstream of this reach.

(g) Discussion

In establishing two-stage geometry, the ditch is widened at the elevation that corresponds to existing bench features or the elevation at which these benches are predicted to form from fluvial processes. Vegetation is left along the fringe of the existing channel, and no work is done to reshape or narrow the current channel. The benches will vegetate quickly, and it is anticipated that the channel will adjust its shape as a function of fluvial processes.

A much debated and often controversial issue is the type of vegetation that should be established on the benches and at the top of the ditches. Trees provide many benefits in natural stream systems and are particularly important for the aquatic biota. However, in straightened, channelized systems, grass might provide better overall benefits. Often trees will affect the ability of nature to establish stable benches, as much of the stability of these systems depends on the dense grass cover that quickly establishes, in the absence of trees. A way of viewing these systems is to think of the small bankfull channels as meadow streams (Rosgen type E channels) that lack the sinuosity that occurs in natural systems. Therefore, trees will often provide the most benefit if they are set back from a grass buffer at the top of the two-stage system or at locations where there is a wide, well-attached flood plain—something that is rarely found in watersheds with extensive networks of agricultural ditches. Constructing wide benches with a 10:1 or larger flood-width ratio might be considered, but that approach will be very expensive in locations where the main function of the ditches is primarily to convey discharges from subsurface drainage systems. In those situations, the ditches must be more than 5 feet deep and sometimes are more than 10 feet deep.

Table 10–2 Relative bed-load transport for various channel conditions

Geometry	Bed slope 0.05%		Bed slope 0.1%		Bed slope 0.2%	
	Relative bed load	D_{50} (mm)	Relative bed load	D_{50} (mm)	Relative bed load	D_{50} (mm)
Current	0.36	2	1	2	2.45	2
3:1 bench ratio	0.06–0.18	4–6	0.28–0.49	6–8	1.03–1.38	8–10
4:1 bench ratio	0.04–0.16	4–6	0.25–0.45	6–8	0.96–1.30	8–10

The primary costs of two-stage ditches are associated with the increased ditch width required. This increased width requires additional initial earthwork. Costs for construction increase with both watershed size and ditch depth and might range from $5 to $20 per linear foot.

Creating a low bench typically requires the top width of the ditch to be greater than what would be required for a traditional trapezoidal channel. It is important to note that the wider ditch top width results in the surrendering of surrounding agricultural production land. To offset landowner costs, the potential for including the bench width in buffer conservation programs should be considered. Buffers have typically been measured from the top of the ditch. Alternatively measuring from the top of the small channel to include the bench and the main side slope of the ditch is preferable from a water-quality perspective and profitability perspective.

In many locations, a do nothing approach might be considered. Removing benches will only provide an increase in the conveyance capacity of the ditch. However, this improvement might only be temporary. If subsurface drains are free flowing and not blocked by bench formations, constructing wider benches may provide limited benefit and could disrupt a functional system that currently provides aquatic and terrestrial habitat and water quality benefits.

654.1008 Conclusion

In channelized ditches and streams that are entrenched and have overwidened bed widths, alluvial channel processes try to develop a flood plain that consists of low benches. Often, these ditches show improved stability. The techniques presented in this chapter are based on observations and analysis of the behavior and evolution of traditional trapezoidal earth channels.

The elevation of the benches and size of the bankfull channel can be determined from regional curves that relate channel dimensions to drainage area. The design approach considers the magnitude and design frequency of discharges for both stages of the channel. It is anticipated that total bench widths that are two to four times the bankfull channel width will result in a stable geometry and a channel with low sinuosity. The overall conveyance capacities of these two-stage systems can be sized based on the probability of out-of-ditch flooding into adjacent areas.

Construction of a two-stage channel system requires a significant capital investment to create a wider design top width. However, it is anticipated that two-stage systems will have improved conveyance capacity, be more self-sustaining, and create and maintain improved aquatic habitat.

United States
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**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 11 Rosgen Geomorphic Channel Design

Issued August 2007

Cover photo: Stream restoration project, South Fork of the Mitchell River, NC, three months after project completion. The Rosgen natural stream design process uses a detailed 40-step approach.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 11 Rosgen Geomorphic Channel Design

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Chapter 11 Rosgen Geomorphic Channel Design

654.1100 Purpose

This chapter outlines a channel design technique based on the morphological and morphometric qualities of the Rosgen classification system. While this approach is written in a series of steps, it is not a cookbook. This approach is often referred to as the Rosgen design approach. The essence for this design approach is based on measured morphological relations associated with bankfull flow, geomorphic valley type, and geomorphic stream type. This channel design technique involves a combination of hydraulic geometry, analytical calculation, regionalized validated relationships, and analogy in a precise series of steps. While this technique may appear to be straightforward in its application, it actually requires a series of precise measurements and assessments. It is important for the reader to recognize that the successful application of this design approach requires extensive training and experience.

The contents of this chapter were submitted to the technical editors of this handbook as a manuscript titled Natural Channel Design Using a Geomorphic Approach, by Dave Rosgen, Wildland Hydrology, Fort Collins, Colorado. This material was edited to fit the style and format of this handbook. The approaches and techniques presented herein are not universally applicable, just as other approaches and techniques presented in this handbook are not necessarily appropriate in all circumstances. However, the Rosgen Geomorphic Approach for Natural Channel Design has been implemented in many locations and is cited as the methodology of choice for stream restoration by several state and local governments.

654.1101 Introduction

River restoration based on the principles of the Rosgen geomorphic channel design approach is most commonly accomplished by restoring the dimension, pattern, and profile of a disturbed river system by emulating the natural, stable river. Restoring rivers involves securing their physical stability and biological function, rather than the unlikely ability to return the river to a pristine state. Restoration, as used in this chapter, will be used synonymously with the term rehabilitation. Any river restoration design must first identify the multiple specific objectives, desires, and benefits of the proposed restoration. The causes and consequences of stream channel problems must then be assessed.

Natural channel design using the Rosgen geomorphic channel design approach incorporates a combination of analog, empirical, and analytical methods for assessment and design. Because all rivers within a wide range of valley types do not exhibit similar morphological, sedimentological, hydraulic, or biological characteristics, it is necessary to group rivers of similar characteristics into discreet stream types. Such characteristics are obtained from stable reference reach locations by discreet valley types, and then are converted to dimensionless ratios for extrapolation to disturbed stream reaches of various sizes.

The proper utilization of this approach requires fundamental training and experience using this geomorphic method. Not only is a strong background in geomorphology, hydrology, and engineering required, but the restoration specialist also must have the ability to implement the design in the field. The methodology is divided into eight major sequential phases:

I Define specific restoration objectives associated with physical, biological, and/or chemical process.

II Develop regional and localized specific information on geomorphologic characterization, hydrology, and hydraulics.

III Conduct a watershed/river assessment to determine river potential; current state; and the nature, magnitude, direction, duration, and consequences of change. Review land

use history and time trends of river change. Isolate the primary causes of instability and/or loss of physical and biological function. Collect and analyze field data including reference reach data to define sedimentological, hydraulic, and morphological parameters. Obtain concurrent biological data (limiting factor analysis) on a parallel track with the physical data.

IV Initially consider passive restoration recommendations based on land use change in lieu of mechanical restoration. If passive methods are reasonable to meet objectives, skip to the monitoring phase (VIII). If passive efforts and/or recovery potential do not meet stated multiple objectives, proceed with the following phases.

V Initiate natural channel design with subsequent analytical testing of hydraulic and sediment transport relations (competence and capacity).

VI Select and design stabilization/enhancement/vegetative establishment measures and materials to maintain dimension, pattern, and profile to meet stated objectives.

VII Implement the proposed design and stabilization measures involving layout, water quality control, and construction staging.

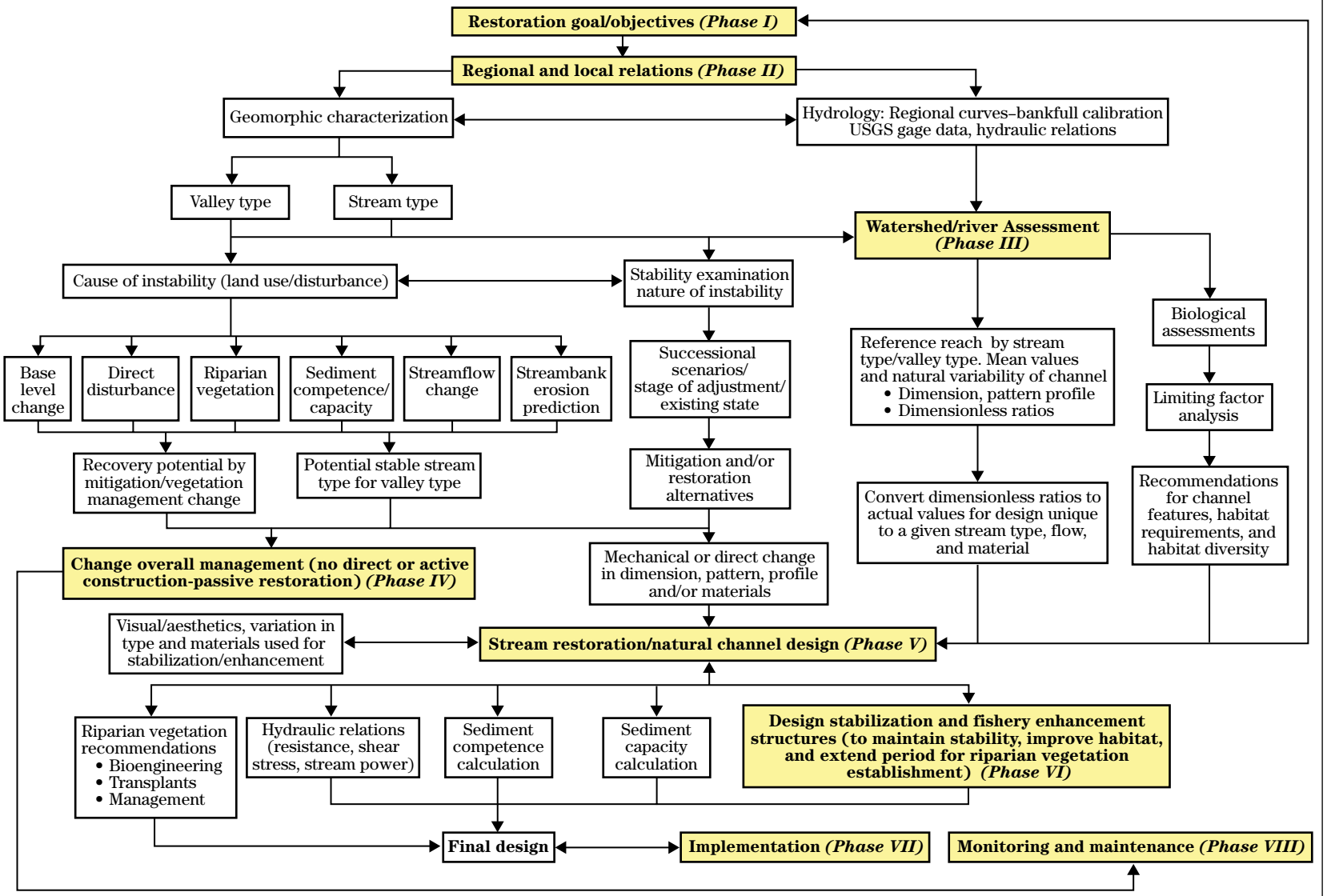
VIII Design a plan for effectiveness, validation, and implementation monitoring to ensure stated objectives are met, prediction methods are appropriate, and the construction is implemented as designed. Design and implement a maintenance plan.

The conceptual layout for the phases of the Rosgen geomorphic channel design approach is shown in figure 11–1. The various phases listed above are indicated on this generalized layout. The flowchart is indicative of the full extent and complexity associated with this method.

Because of the complexity and uncertainty of natural systems, it becomes imperative to monitor each restoration project. The following are three objectives of such monitoring:

- Ensure correct implementation of the design variables and construction details.
- Validate the analog, empirical, and analytical methods used for the assessment and design.
- Determine effectiveness of the restoration methods to the stated physical and biological restoration objectives.

Figure 11–1 River restoration using Rosgen geomorphic channel design approach



654.1102 Restoration phases

(a) Phase I—Restoration objectives

It is very important to obtain clear and concise statements of restoration objectives to appropriately design the solution(s). The potential of a certain stream to meet specific objectives must be assessed early on in the planning phases so that the initial restoration direction is appropriate. The common objectives are:

- flood level reduction
- streambank stability
- reduce sediment supply, land loss, and attached nutrients
- improve visual values
- improve fish habitat and biological diversity
- create a natural stable river
- withstand floods
- be self-maintaining
- be cost-effective
- improve water quality
- improve wetlands

It is essential to fully describe and understand the restoration objectives. The importance of formulating clear, achievable, and measurable objectives is described in detail in NEH654.02. Often the objectives can be competing or be in conflict with one another. Conflict resolution must be initiated and can often be offset by varying the design and/or the nature of stabilization methods or materials planned.

The assessment required must also reflect the restoration objectives to ensure various related processes are thoroughly evaluated. For example, if improved fishery abundance, size, and species are desired, a limiting factor analysis of habitat and fish populations must be linked with the morphological and sedimentological characteristics.

(b) Phase II—Developing local and regional relations in geomorphic characterization, hydrology, and hydraulics

Geomorphic characterization

The relations mapped at this phase are the geomorphic characterization and description levels for stream classification (Rosgen 1994, 1996). Valley types (table 11–1) are mapped prior to stream classification to ensure reference reach data are appropriately applied for the respective valley types being studied. Morphological relations associated with stream types are presented in figures 11–2 (Rosgen 1994) and 11–3 (Rosgen 1996) and summarized in table 11–2. In natural channel design using the Rosgen geomorphic channel design approach, it is often advantageous to have an undisturbed and/or stable river reach immediately upstream of the restoration reach. Reference reach data are obtained and converted to dimensionless ratio relations to extrapolate channel dimension, pattern, profile, and channel material data to rivers and valleys of the same type, but of different size. If an undisturbed/stable river reach is not upstream of the restoration reach, extrapolation of morphological and dimensionless ratio relations by valley and stream type is required for both assessment and design.

An example of the form used to organize reference reach data, including dimensionless ratios for a given stream type, is presented in table 11–3. Specific design variables use reference reach data for extrapolation purposes, assuming the same valley and stream type as represented. These relations are only representative of a similar stable stream type within a valley type of the disturbed stream.

Hydrology

The hydrology of the basin is often determined from regional curves constructed from long-term stream gage records. Relationships of flow-duration curves and flood-frequency data are used for computations in both the assessment and design phases. Stream Hydrology is also addressed in NEH654.05. Relations are converted to dimensionless formats using bankfull discharge as the normalization parameter. Bankfull discharge and dimensions associated with stream gages are plotted as a function of drainage area for extrapolation to ungaged sites in similar hydro-physiographic provinces. A key requirement in the development of

Table 11–1 Valley types used in geomorphic characterization

Valley types	Summary description of valley types
I	Steep, confined, V-notched canyons, rejuvenated side slopes
II	Moderately steep, gentle-sloping side slopes often in colluvial valleys
III	Alluvial fans and debris cones
IV	Gentle gradient canyons, gorges, and confined alluvial and bedrock-controlled valleys
V	Moderately steep, U-shaped glacial-trough valleys
VI	Moderately steep, fault, joint, or bedrock (structural) controlled valleys
VII	Steep, fluvial dissected, high-drainage density alluvial slopes
VIII	Wide, gentle valley slope with well-developed flood plain adjacent to river and/or glacial terraces
IX	Broad, moderate to gentle slopes, associated with glacial outwash and/or eolian sand dunes
X	Very broad and gentle valley slope, associated with glacio- and nonglacio-lacustrine deposits
XI	Deltas

Figure 11–2 Broad-level stream classification delineation showing longitudinal, cross-sectional, and plan views of major stream types

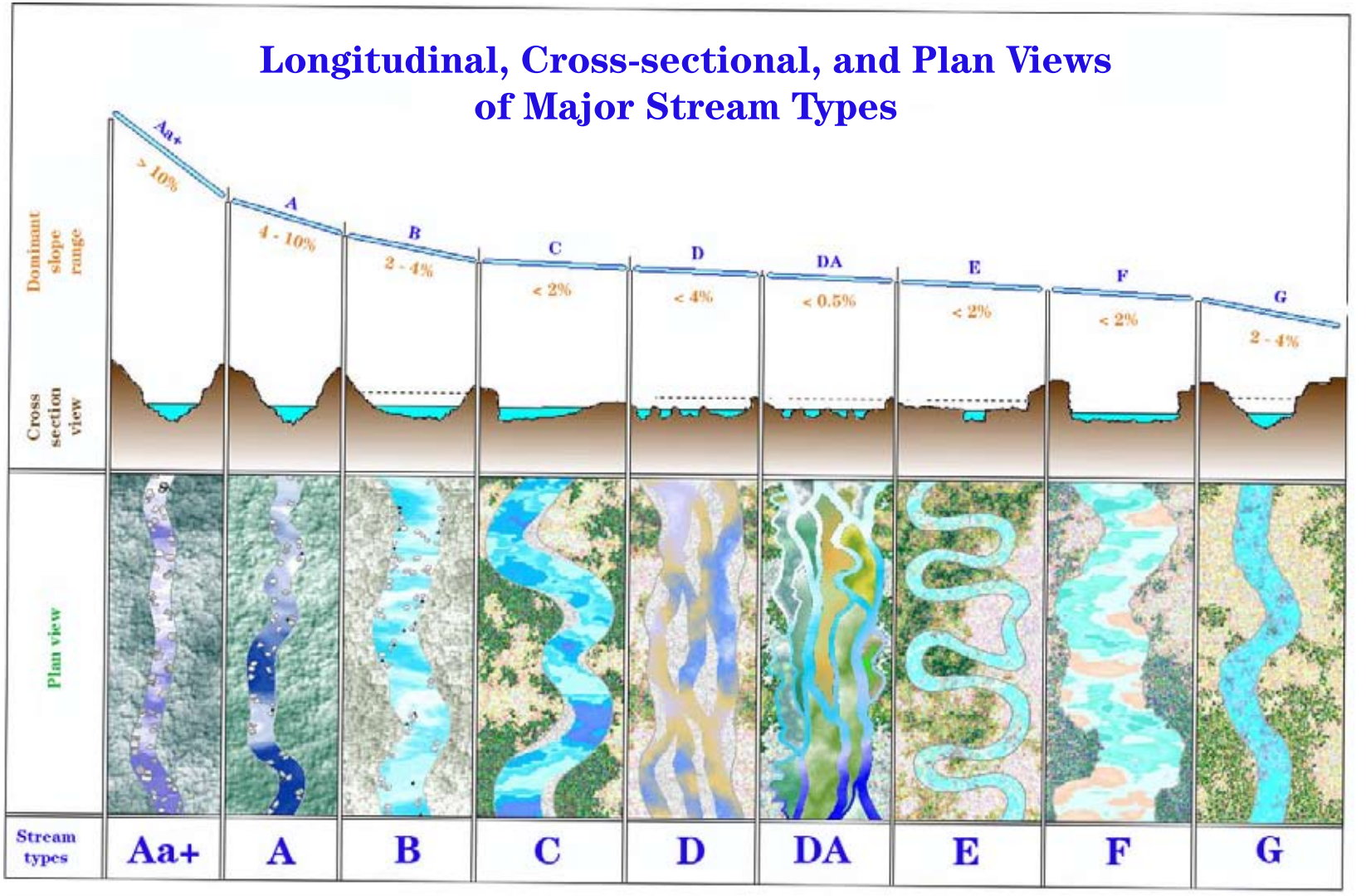
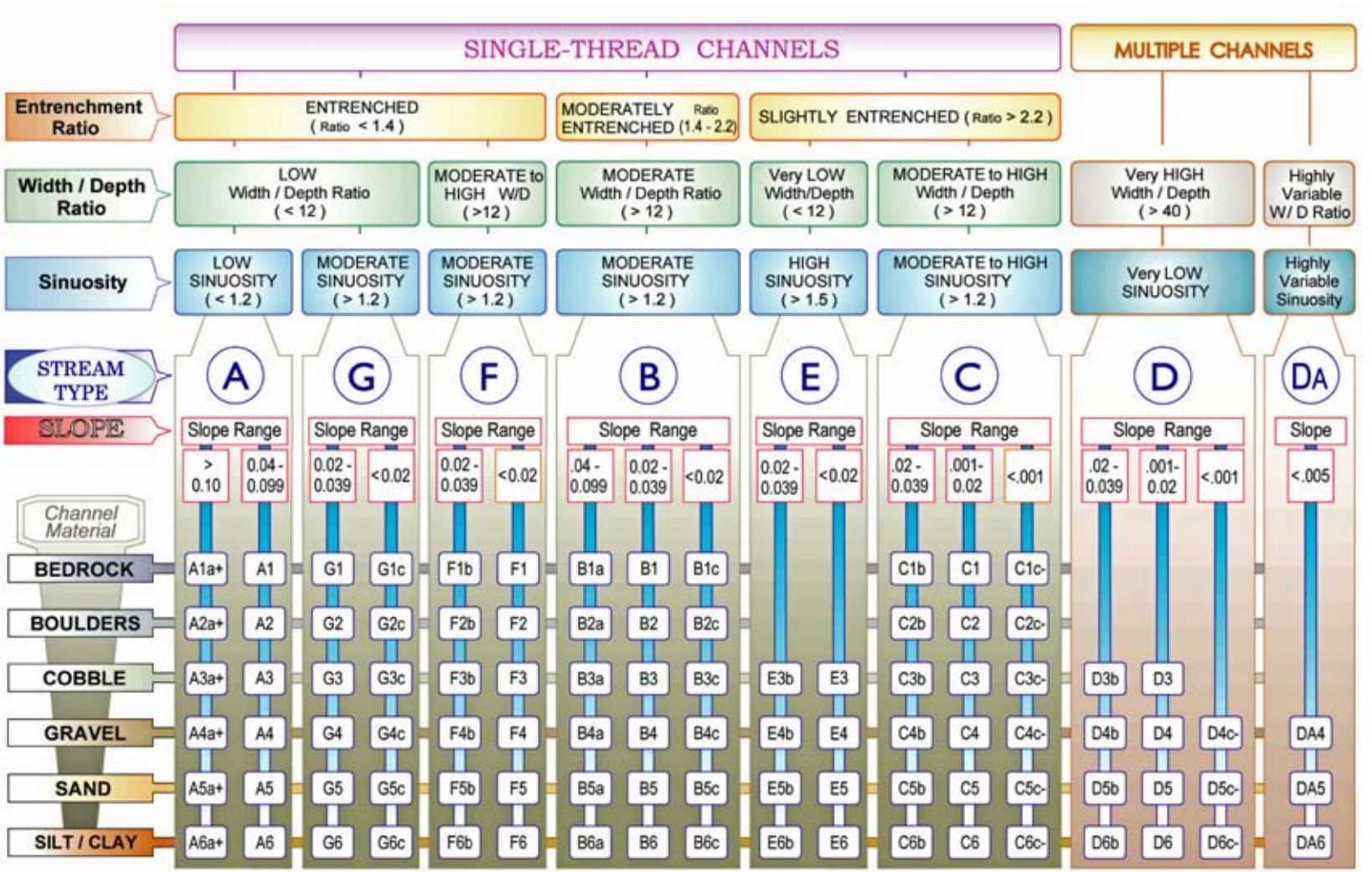


Figure 11–3 Classification key for natural rivers



KEY to the ROSGEN CLASSIFICATION of NATURAL RIVERS. As a function of the *"continuum of physical variables"* within stream reaches, values of ***Entrenchment*** and ***Sinuosity*** ratios can vary by +/- 0.2 units; while values for ***Width / Depth*** ratios can vary by +/- 2.0 units.

Table 11–2 General stream type descriptions and delineative criteria for broad-level classification (level 1)

Stream type	General description	Entrenchment ratio	W/d ratio	Sinuosity	Slope	Landform/ soils/features
Aa+	Very steep, deeply entrenched, debris transport, torrent streams	<1.4	<12	1.0 to 1.1	>.10	Very high relief. Erosional, bedrock, or depositional features; debris flow potential. Deeply entrenched streams. Vertical steps with deep scour pools; waterfalls
A	Steep, entrenched, cascading, step-pool streams. High energy/ debris transport associated with depositional soils. Very stable if bedrock or boulder-dominated channel	<1.4	<12	1.0 to 1.2	.04 to .10	High relief. Erosional or depositional and bedrock forms. Entrenched and confined streams with cascading reaches. Frequently spaced, deep pools in associated step-pool bed morphology
B	Moderately entrenched, moderate gradient, riffle dominated channel with infrequently spaced pools. Very stable plan and profile. Stable banks	1.4 to 2.2	>12	>1.2	.02 to .039	Moderate relief, colluvial deposition and/or structural. Moderate entrenchment and width-to-depth ratio. Narrow, gently sloping valleys. Rapids predominate with scour pools
C	Low gradient, meandering, point bar, riffle/pool, alluvial channels with broad, well-defined flood plains	>2.2	>12	>1.2	<.02	Broad valleys with terraces, in association with flood plains, alluvial soils. Slightly entrenched with well-defined meandering channels. Riffle/ pool bed morphology
D	Braided channel with longitudinal and transverse bars. Very wide channel with eroding banks	n/a	>40	n/a	<.04	Broad valleys with alluvium, steeper fans. Glacial debris and depositional features. Active lateral adjustment with abundance of sediment supply. Convergence/divergence bed features, aggradational processes, high bed load and bank erosion
DA	Anastomizing (multiple channels) narrow and deep with extensive, well-vegetated flood plains and associated wetlands. Very gentle relief with highly variable sinuosities and width-to-depth ratios. Very stable streambanks	>2.2	Highly variable	Highly variable	<.005	Broad, low-gradient valleys with fine alluvium and/or lacustrine soils. Anastomized (multiple channel) geologic control creating fine deposition with well-vegetated bars that are laterally stable with broad wetland flood plains. Very low bed-load, high wash load sediment
E	Low gradient, meandering riffle/pool stream with low width-to-depth ratio and little deposition. Very efficient and stable. High meander width ratio	>2.2	<12	>1.5	<.02	Broad valley/meadows. Alluvial materials with flood plains. Highly sinuous with stable, well-vegetated banks. Riffle/pool morphology with very low width-to-depth ratios
F	Entrenched meandering riffle/pool channel on low gradients with high width-to-depth ratio	<1.4	>12	>1.2	<.02	Entrenched in highly weathered material. Gentle gradients with a high width-to-depth ratio. Meandering, laterally unstable with high bank erosion rates. Riffle/pool morphology
G	Entrenched gully step-pool and low width-to-depth ratio on moderate gradients	<1.4	<12	>1.2	.02 to .039	Gullies, step-pool morphology with moderate slopes and low width-to-depth ratio. Narrow valleys, or deeply incised in alluvial or colluvial materials (fans or deltas). Unstable, with grade control problems and high bank erosion rates

Table 11–3 Reference reach summary data form

River Reach Summary Data

Channel dimension	Mean riffle depth (d_{bkf})		ft	Riffle width (W_{bkf})		ft	Riffle area (A_{bkf})		ft^2
	Mean pool depth (d_{bkfp})		ft	Pool width (W_{bkfp})		ft	Pool area (A_{bkfp})		ft^2
	Mean pool depth/mean riffle depth		d_{bkfp}/(d_{bkf})	Pool width/riffle width		W_{bkfp}/W_{bkf}	Pool area/riffle area		A_{bkfp}/A_{bkf}
	Max riffle depth (d_{mbkf})		ft	Max pool depth (d_{mbkfp})		ft	Max riffle depth/mean riffle depth		
	Max pool depth/mean riffle depth						Point bar slope		
	Streamflow: estimated mean velocity at bankfull stage (u_{bkf})					ft/s	Estimation method		
	Streamflow: estimated discharge at bankfull stage (Q_{bkf})					ft^3/s	Drainage area		mi^2

	Geometry	Mean	Min.	Max.		Dimensionless geometry ratios	Mean	Min.	Max.
Channel pattern	Meander length (Lm)				ft	Meander length ratio (Lm/W_{bkf})			
	Radius of curvature (Rc)				ft	Radius of curvature/riffle width (Rc/W_{bkf})			
	Belt width (W_{blt})				ft	Meander width ratio (W_{blt}/W_{bkf})			
	Individual pool length				ft	Pool length/riffle width			
	Pool to pool spacing				ft	Pool to pool spacing/riffle width			

Channel profile	Valley slope (VS)			ft/ft	Average water surface slope (S)		ft/ft	Sinuosity (VS/S)		
	Stream length (SL)			ft	Valley length (VL)		ft	Sinuosity (SL/VL)		
	Low bank height (LBH)	start		ft	Max riffle depth	start	ft	Bank height ratio (LBH/max riffle depth)	start	
		end		ft		end	ft		end	

	Facet slopes	Mean	Min.	Max.		Dimensionless geometry ratios	Mean	Min.	Max.
Channel profile	Riffle slope (S_{rif})				ft/ft	Riffle slope/average water surface slope (S_{rif}/S)			
	Run slope (S_{run})				ft/ft	Run slope/average water surface slope (S_{run}/S)			
	Pool slope (S_p)				ft/ft	Pool slope/average water surface slope (S_p/S)			
	Glide slope (S_g)				ft/ft	Glide slope/average water surface slope (S_g/S)			
	Feature midpoint[a/]	**Mean**	**Min.**	**Max.**		**Dimensionless geometry ratios**	**Mean**	**Min.**	**Max.**
	Riffle depth (d_{rif})				ft	Riffle depth/mean riffle depth (d_{rif}/d_{bkf})			
	Run depth (d_{run})				ft	Run depth/mean riffle depth (d_{run}/d_{bkf})			
	Pool depth (d_p)				ft	Pool depth/mean riffle depth (d_p/d_{bkf})			
	Glide depth (d_g)				ft	Glide depth/mean riffle depth (d_g/d_{bkf})			

	Geometry		Reach[b/]	Riffle[c/]	Bar		Reach[b/]	Riffle[c/]	Bar	
Channel materials	% Silt/clay					D_{16}				mm
	% Sand					D_{35}				mm
	% Gravel					D_{50}				mm
	% Cobble					D_{84}				mm
	% Boulder					D_{95}				mm
	% Bedrock					D_{100}				mm

a/ Minimum, maximum, mean depths are the average midpoint values except pools which are taken at deepest part of pool
b/ Composite sample of riffles and pools within the designated reach
c/ Active bed of a riffle

such relations is the necessity to field-calibrate the bankfull stage at each gage within a hydro-physiographic province (a drainage basin similar in precipitation/runoff relations due to precipitation/elevation, lithology and land uses).

Regional curves—The field-calibrated bankfull stage is used to obtain the return period associated with the bankfull discharge. Regional curves of bankfull discharge versus drainage area are developed (fig. 11–4) (adapted from Dunne and Leopold 1978)). To plot bankfull dimensions by drainage area, the U.S. Geological Survey (USGS) 9–207 data (summary of stream discharge measurements at the gage) are obtained to plot the at-a-station hydraulic geometry relations (fig. 11–5 (adapted from Rosgen 1996; Rosgen and Silvey 2005)). These data are then converted to dimensionless hydraulic geometry data by dividing each value by their respective bankfull value. These relations are used during assessment and design to indicate the shape of the various cross sections from low flow to high flow. In the development of the dimensionless hydraulic geometry data, current meter measurements must be stratified by stream type (Rosgen 1994, 1996) and for specific bed features such as riffles, glides, runs, or pools.

Figure 11–4 Regional curves from stream gaging stations showing bankfull discharge (ft^3/s) vs. drainage area (mi^2)

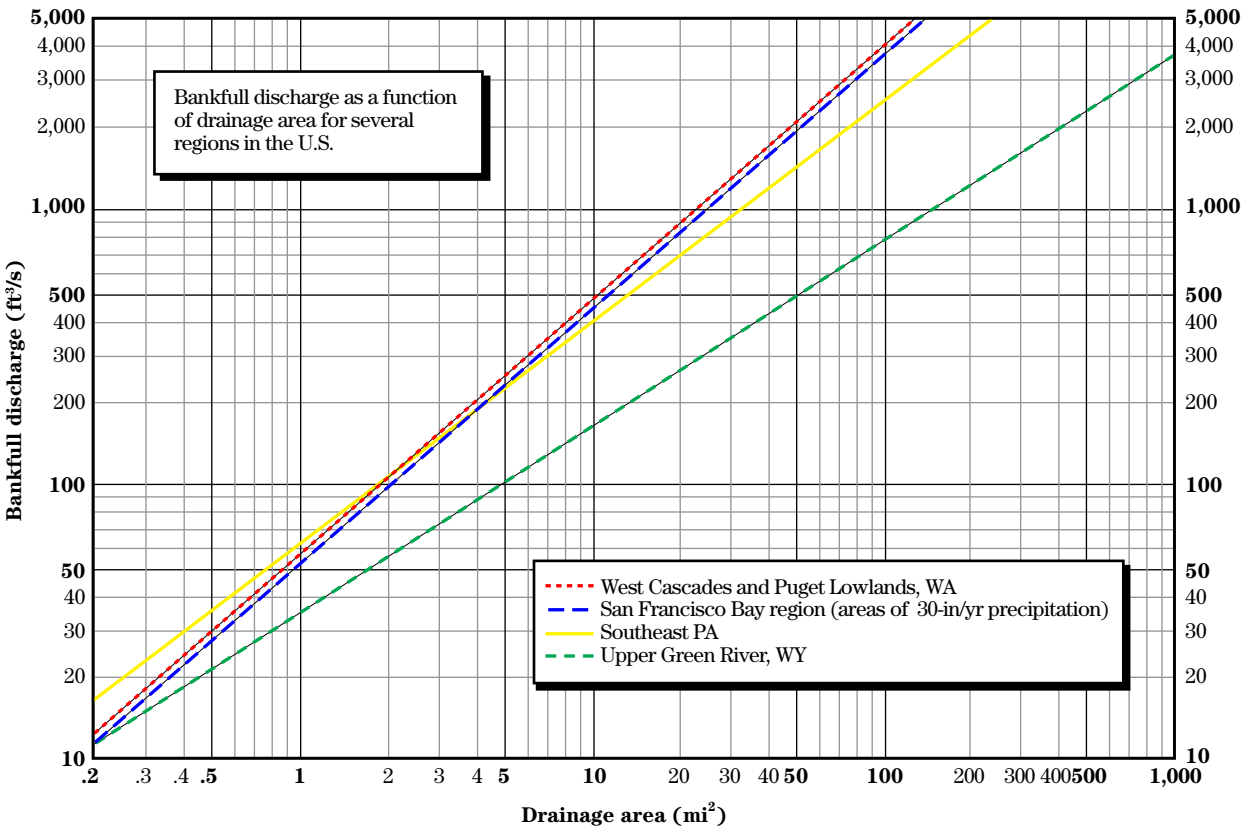
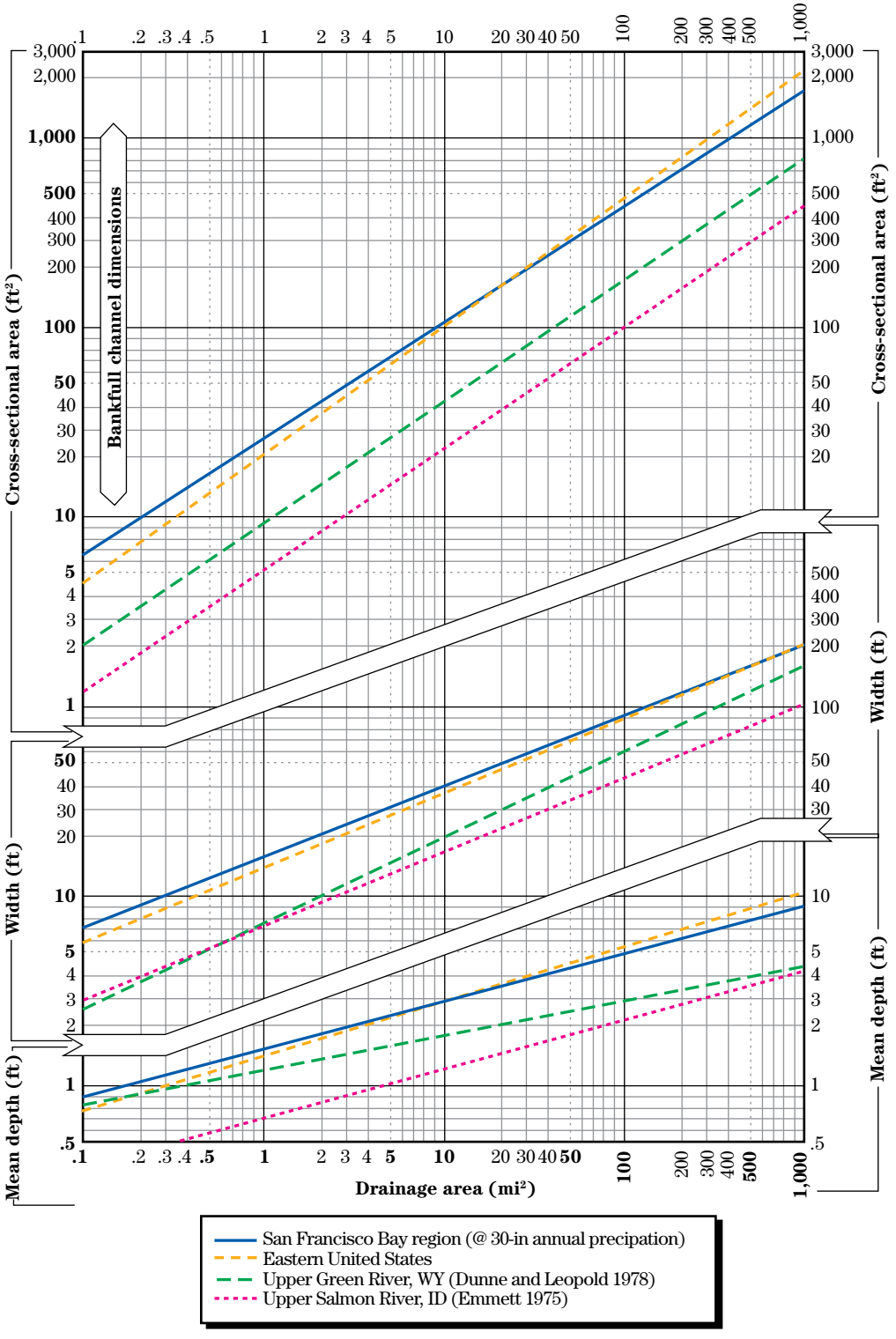


Figure 11–5 Regional curves from stream gage stations showing bankfull dimensions (width, depth, and cross-sectional area) vs. drainage area (mi²)



Hydraulic relations

Hydraulic relations are validated using resistance equations for velocity prediction at ungaged sites. (Stream Hydraulics is addressed in more detail in NEH654.06) Validation is accomplished by back calculating relative roughness (R/D_{84}) and a friction factor (u/u^*) from actual measured velocity for a range of streamflows including bankfull:

$$u = \left[2.83 + 5.66 \log\left(\frac{R}{D_{84}}\right)\right] u^* \qquad \text{(eq. 11–1)}$$

where:

u = mean velocity (ft/s)
R = hydraulic radius
D_{84} = diameter of bed material of the 84th percentile of riffles
u^* = shear velocity $(gRS)^{1/2}$
g = gravitational acceleration
S = slope

Measured velocity, slope, channel material, and hydraulic radius data from various Colorado rivers using this friction factor (u/u^*) and relative roughness (R/D_{84}) relation are shown in figure 11–6 (Rosgen, Leopold, and Silvey 1998; Rosgen and Silvey 2005).

Manning's *n* (roughness coefficient) can also be back-calculated from measured velocity, slope, and hydraulic radius. Another approach to predict velocity at ungaged sites is to predict Manning's *n* from a friction factor back-calculated from relative roughness shown in figure 11–7 (Rosgen, Leopold, and Silvey 1998; Rosgen and Silvey 2005). Manning's *n* can also be estimated at the bankfull stage by stream type as shown in the relationship from gaged, large streams in figure 11–8. Vegetative influence is also depicted in these data (Rosgen 1994).

Dimensionless flow-duration curves—Flow-duration curves (based on mean daily discharge) are also obtained from gage stations then converted to dimensionless form using bankfull discharge as the normalization parameter (fig. 11–9 (Emmett 1975)). The purpose of this form is to allow the user to extrapolate flow-duration curves to ungaged basins. This relationship is needed for the annual suspended and bed-load sediment yield calculation along with channel hydraulic variables.

(c) Phase III—Watershed and river assessment

Land use history is a critical part of watershed assessment to understand the nature and extent of potential impacts to the water resources. Past erosional/depositional processes related to changes in vegetative cover, direct disturbance, and flow and sediment regime changes provide insight into the direction and detail for assessment procedures required for restoration. Time series of aerial photos are of particular value to understand the nature, direction, magnitude, and rate of change. This is very helpful, as it assists in assessing both short-term, as well as long-term river problems.

Assessment of river stability and sediment supply

River stability (equilibrium or quasi-equilibrium) is defined as the ability of a river, over time, in the present climate to transport the flows and sediment produced by its watershed in such a manner that the stream maintains its dimension, pattern, and profile without either aggrading or degrading (Rosgen 1994, 1996, 2001d). A stream channel stability analysis is conducted along with riparian vegetation inventory, flow and sediment regime changes, limiting factor analysis compared to biological potential, sources/causes of instability, and adverse consequences to physical and biological function. Procedures for this assessment are described in detail by Rosgen (1996, 2001d) and in Watershed Assessment and River Stability for Sediment Supply (WARSSS) (Rosgen 1999, 2005).

It is important to realize the difference between the dynamic nature of streams and natural adjustment processes compared to an acceleration of such adjustments. For example, bank erosion is a natural channel process; however, accelerated streambank erosion must be understood when the rate increases and creates a disequilibrium condition. Many stable rivers naturally adjust laterally, such as the "wandering" river. While it may meet certain local objectives to stabilize high risk banks, it would be inadvisable to try to "control" or "fix in place" such a river.

In many instances, a braided river and/or anastomizing river type is the stable form. Designing all stream systems to be a single-thread meandering stream may not properly represent the natural stable form. Valley types are a key part of river assessment to understand

Figure 11–6 Relation of channel bed particle size to hydraulic resistance with river data from a variety of eastern and western streams

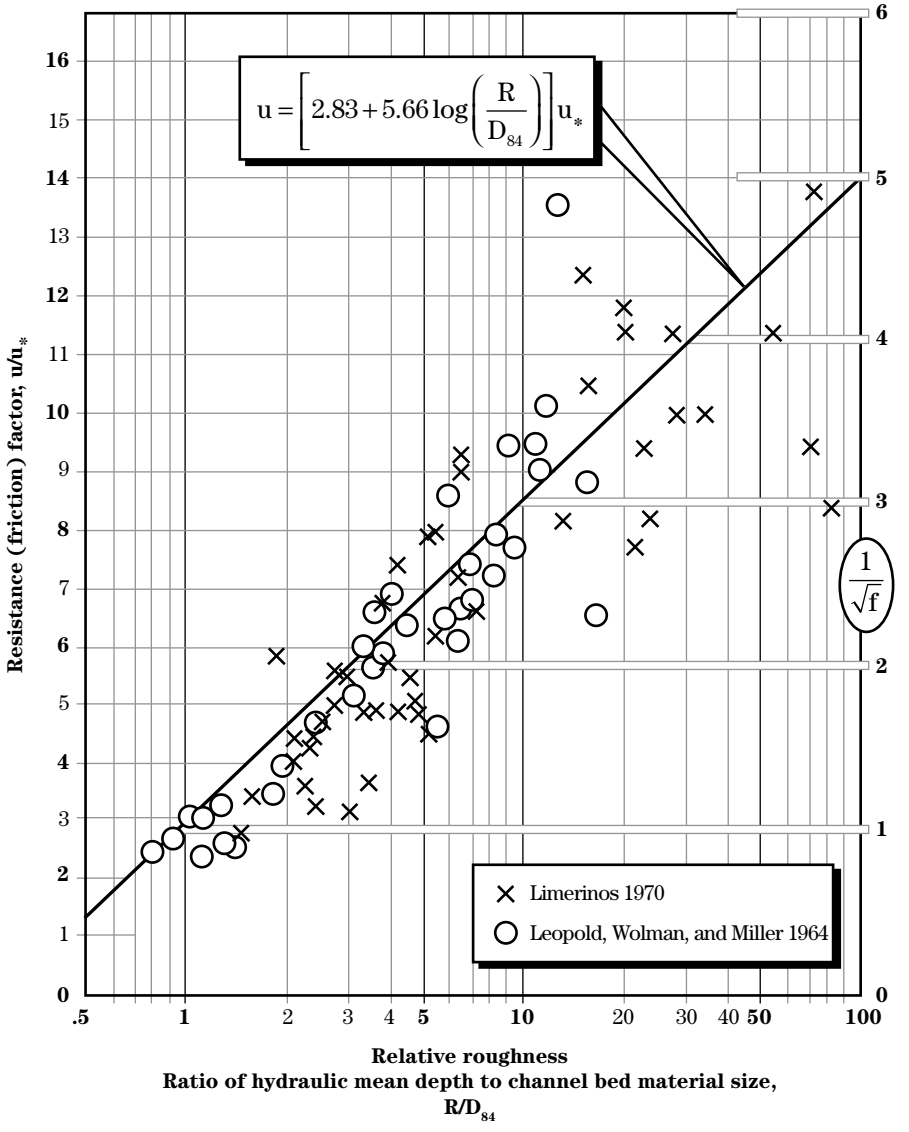


Figure 11–7 Prediction of Manning's n roughness coefficient

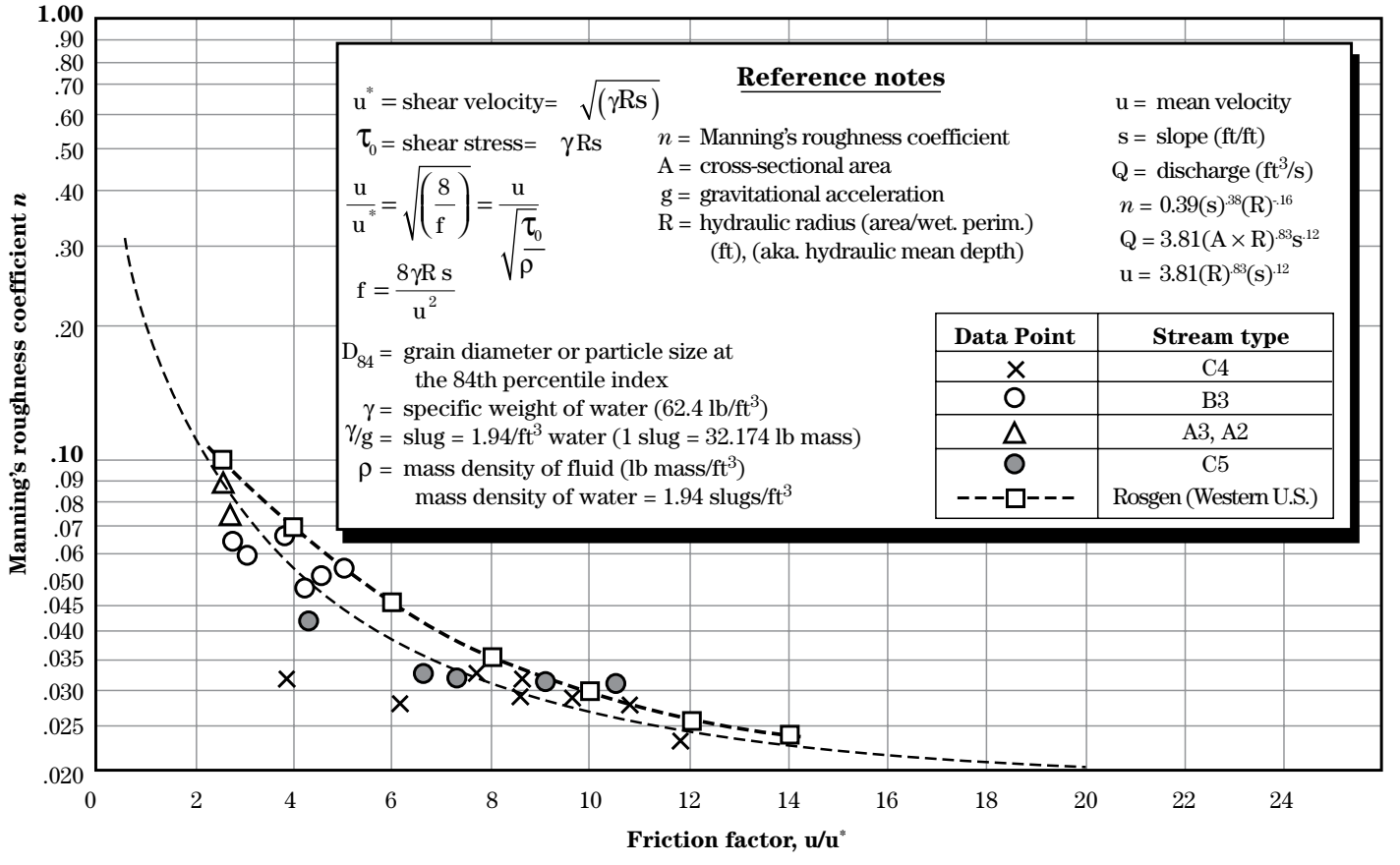
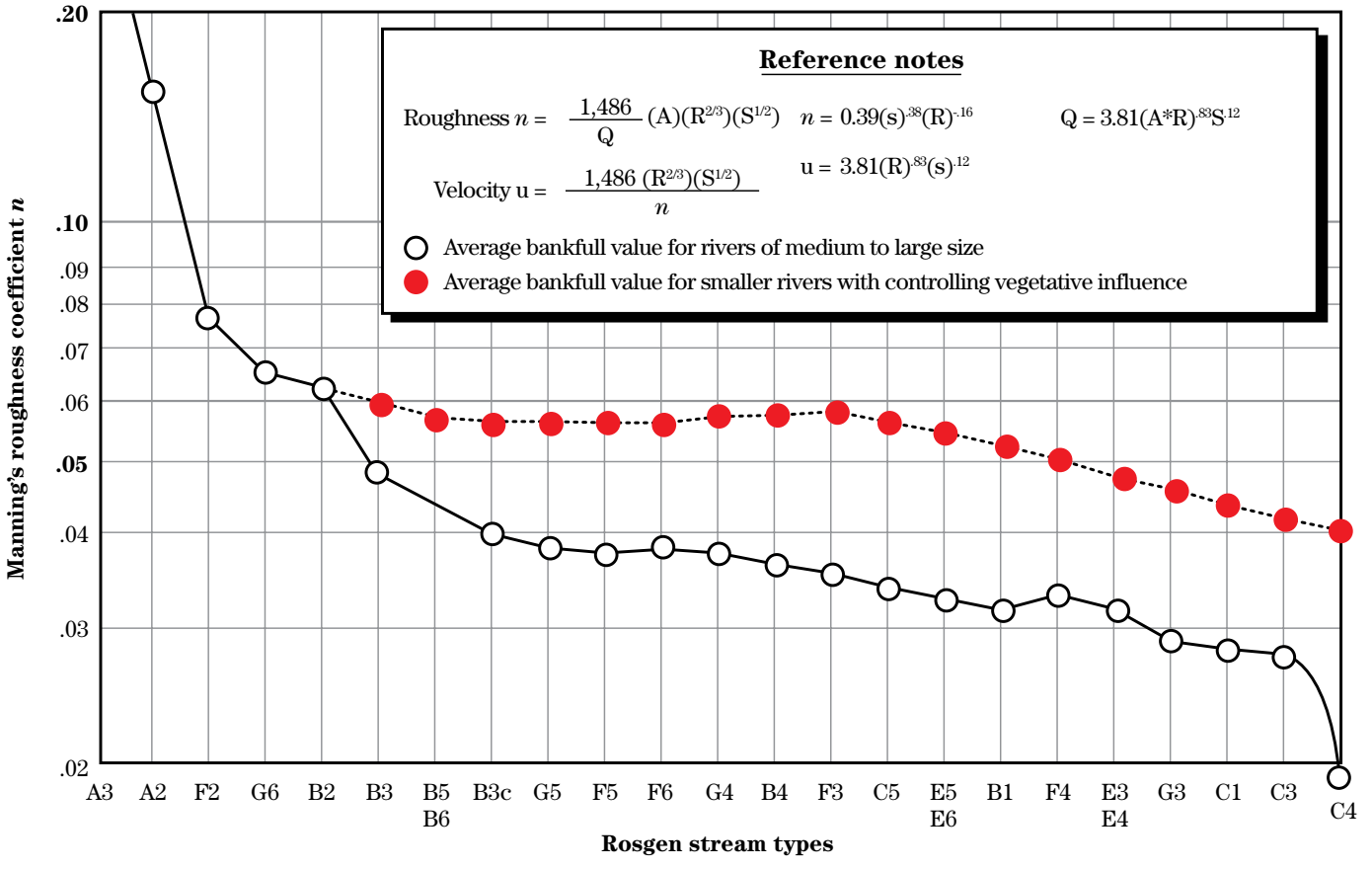


Figure 11–8 Bankfull stage roughness coefficients (n values) by stream type for 140 streams in the United States and New Zealand

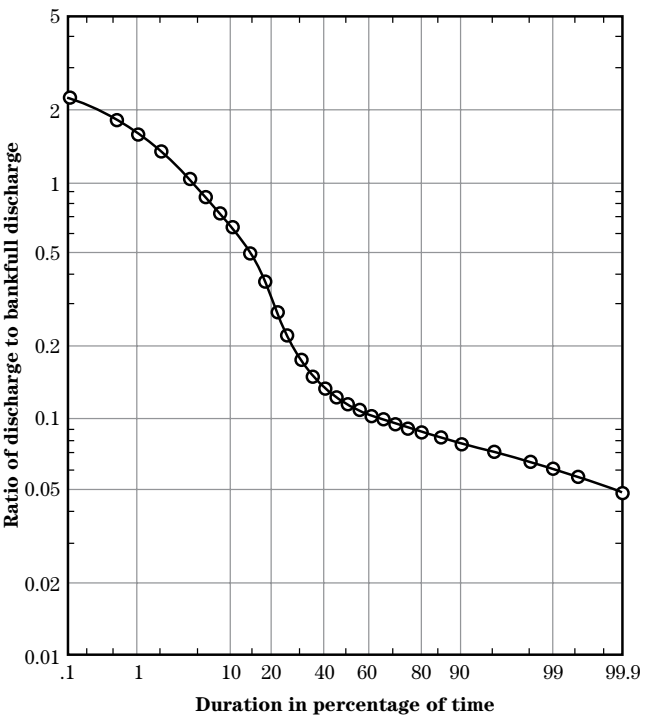


which stream types are stable within a variety of valley types in their geomorphic settings. Reference reaches that represent the stable form have to be measured and characterized only for use in similar valley types. This prevents applying good data to the *wrong* stream type.

Time-trend data using aerial photography is very valuable at documenting channel change. Field evidence using dendrochronology, stratigraphy, carbon dating, paleochannels, or evidence of avulsion and avulsion dates can help the field observer to understand rate, direction, and consequence of channel change.

The field inventory and the number of variables required to conduct a watershed and river stability assessment is substantial. The flowchart in figure 11–10 represents a general summary of the various elements used for assessing channel stability as used in this methodology. The assessment effort is one of the key procedural steps in a sound restoration plan, as it identifies the causes and consequences of the problems leading to loss of physical and biological river function. Some of the major variables are described to provide a *general* overview.

Figure 11–9 Dimensionless flow-duration curve for streamflow in the upper Salmon River area

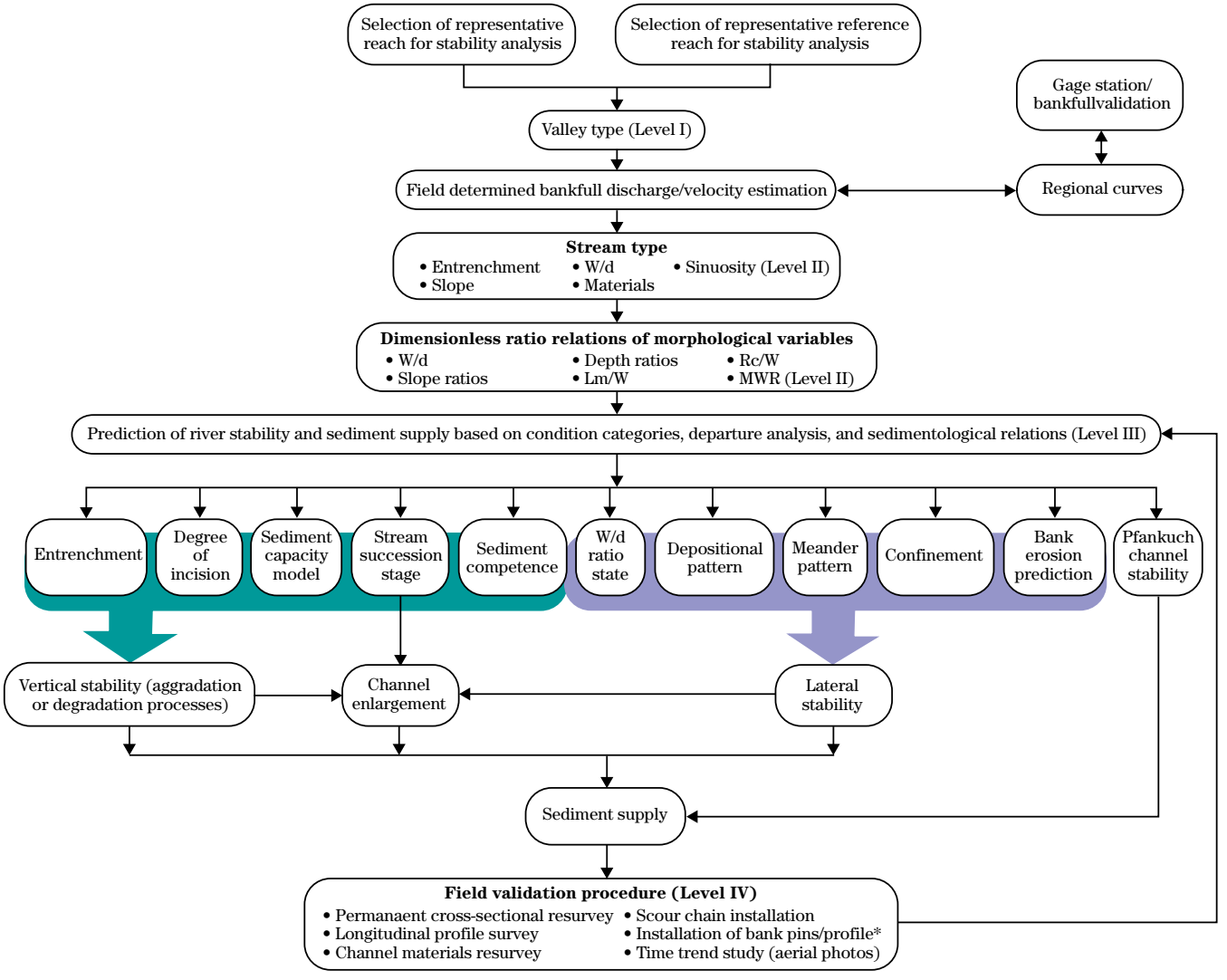


Streamflow change—Streamflow alteration (magnitude, duration, and timing) due to land use changes, such as percent impervious cover, must be determined at this phase. Streamflow models, such as the unit hydrograph approach, must be calibrated by back-calculating what precipitation probability generates bankfull discharge for various antecedent soil moisture and runoff curve numbers. It is critical to separate bankfull discharge from flood flows, as each flow category, including flood flow, has a separate dimension, pattern, and profile. This varies by stream type and the lateral and vertical constraints imposed within the valley (or urban "valley").

Flow-duration curves by similar hydro-physiographic provinces from gaged stations are converted to bankfull dimensionless flow duration for use in the annual sediment yield calculation. Snowmelt watershed flow prediction output (Troendle, Swanson, and Nankervis 2005) is generally shown in flow-duration changes, rather than an annual hydrograph. Similar model outputs using flow-duration changes are shown in Water Resources Evaluation of Nonpoint Silvicultural Sources (U.S. Environmental Protection Agency (EPA) 1980).

Sediment competence—Sedimentological data are obtained by a field measurement of the size of bar and bed material, bed-load sediment transport, suspended sediment transport, and bankfull discharge measurements at the bankfull stage. Sediment relations are established by collecting energy slope, hydraulic radius, bed material, bar material, and the largest particle produced by the drainage immediately upstream of the assessment reach. Critical dimensionless shear stress is calculated from field data to determine *sediment competence* (ability to move the largest particle made available to the channel). Procedures for this field inventory are presented in Andrews (1984) and Rosgen (2001a, 2001d, 2005). Potential aggradation, degradation, and channel enlargement are predicted for the disturbed reach, comparing the required depth and slope necessary to transport the largest size sediment available. These calculations can be accomplished by hand, spreadsheet, or by commercially available computer programs.

Figure 11–10 Generalized flowchart of application of various assessment levels of channel morphology, stability ratings, and sediment supply



*Optional: sediment measurements (largest size moved at bankfull, D_l)

Changes in channel dimension, pattern, and profile are reflected in changes of velocity, depth, and slope. These changes in the hydraulic variables are reflected in values of shear stress. Shear stress is defined as:

$$\tau = \gamma RS \qquad \text{(eq. 11–2)}$$

where:
τ = bankfull shear stress (lb/ft^2)
γ = specific weight of water = 62.4 lb/ft^3
R = hydraulic radius of riffle cross section (ft)
S = average water surface slope (ft/ft)

Use the calculated value of τ (lb/ft^2) and the Shields diagram as revised with the Colorado data (fig. 11–11 (Rosgen and Silvey 2005)) to predict the moveable particle size (mm) at bankfull shear stress.

Another relationship used in assessment and in design is the use of dimensionless shear stress (τ^*_{ci}) to determine particle entrainment. Dimensionless shear stress is defined as:

$$\tau^* = 0.0834\left(\frac{D_{50}}{\hat{D}_{50}}\right)^{-0.872} \qquad \text{(eq. 11–3)}$$

where:
τ^* = dimensionless shear stress
D_{50} = median diameter of the riffle bed (from 100 count in the riffle or pavement sample)
$\hat{D}_{50}$ = median diameter of the bar sample (or subpavement sample)

If the ratio $\frac{D_{50}}{\hat{D}_{50}}$ is between the values of 3.0 and 7.0, calculate the critical dimensionless shear stress using equation 11–3 (modifications adapted from Andrews 1983, 1984; Andrews and Erman 1986).

If the ratio $\frac{D_{50}}{\hat{D}_{50}}$ is **not** between the values of 3.0 and 7.0, calculate the ratio $\frac{D_{max}}{D_{50}}$

where:
D_{max} = largest particle from the bar sample (or the subpavement sample)
D_{50} = median diameter of the riffle bed (from 100 count in the riffle or the pavement sample)

If the ratio $\frac{D_{max}}{D_{50}}$ is between the value of 1.3 and 3.0, calculate the critical dimensionless shear stress:

$$\tau^* = 0.0384\left(\frac{D_{max}}{D_{50}}\right)^{-0.887} \qquad \text{(eq. 11–4)}$$

Once the dimensionless shear stress is determined, the bankfull mean depth required for entrainment of the largest particle in the bar sample (or subpavement sample) is calculated using equation 11–5:

$$d_{bkf} = 1.65\tau^* \frac{D_{max}}{S} \qquad \text{(eq. 11–5)}$$

where:
d_{bkf} = required bankfull mean depth (ft)
1.65 = submerged specific weight of sediment
τ^* = dimensionless shear stress
D_{max} = largest particle from bar sample (or subpavement sample) (ft)
S = bankfull water surface slope (ft/ft)

The bankfull water surface slope required for entrainment of the largest particle can be calculated using equation 11–6:

$$S = 1.65\tau^* \frac{D_{max}}{d_{bkf}} \qquad \text{(eq. 11–6)}$$

Equations 11–5 and 11–6 are derived from the basic Shields relation.

If the protrusion ratios are out of the usable range as stated, another option is to calculate sediment entrainment using dimensional bankfull shear stress (eq. 11–2 and fig. 11–11).

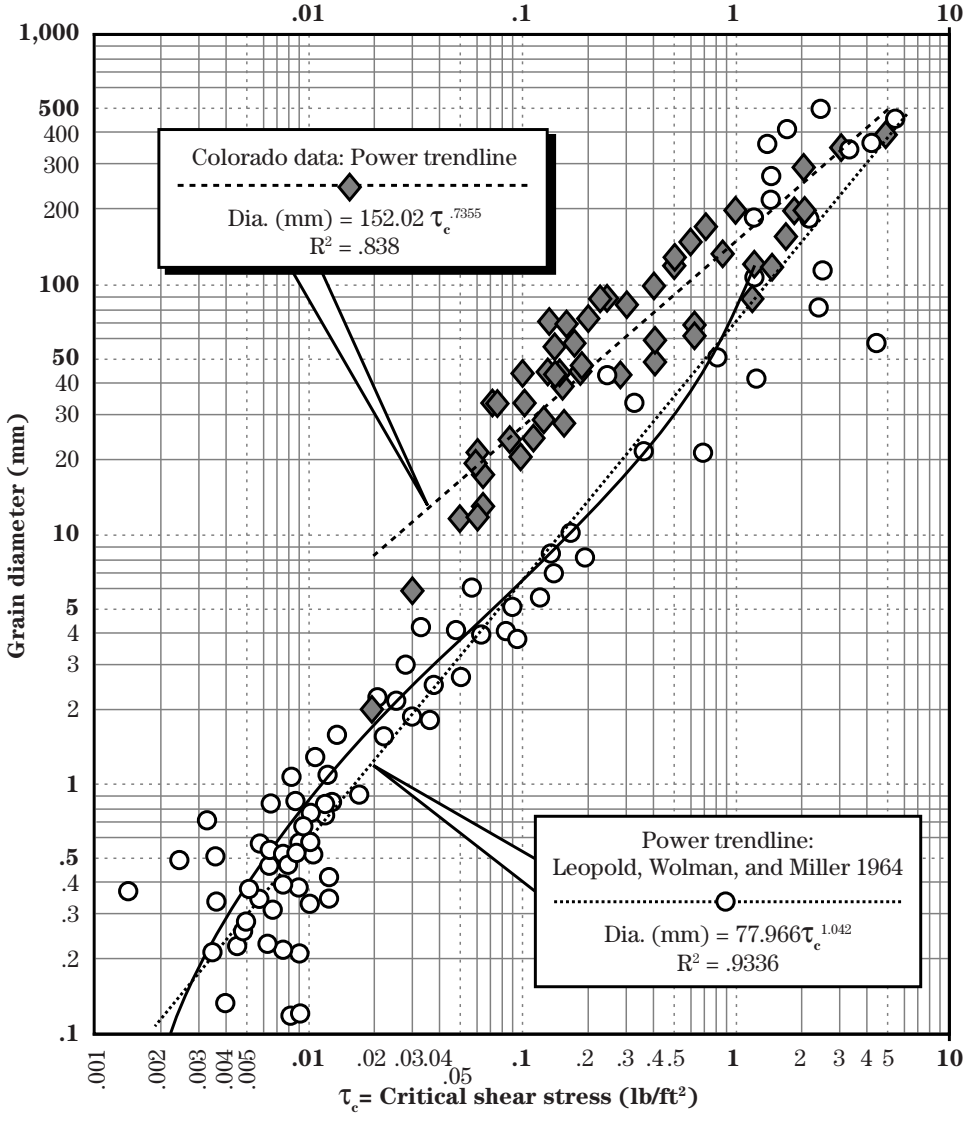
Sediment capacity—In addition to sediment competence, sediment capacity is important to predict river stability. Unit stream power is also utilized to determine the distribution of energy associated with changes in the dimension, pattern, profile, and materials of stream channels. Unit stream power is defined as shear stress times mean velocity:

$$\omega = \tau u \qquad \text{(eq. 11–7)}$$

where:
ω = unit stream power (lb/ft/s)
τ = shear stress (lb/ft^2)
u = mean velocity (ft/s)

Predicted sediment rating curves are converted to unit stream power for the same range of discharges by individual cells to demonstrate reduction or increase in coarse sediment transport.

Figure 11–11 Relation between grain diameter for entrainment and shear stress using Shields relations



Laboratory and field data on critical shear stress required to initiate movement of grains (Leopold, Wolman, and Miller 1964). The solid line is the Shields curve of *the threshold of motion;* transposed from the θ versus R_g form into the present form, in which critical shear is plotted as a function of grain diameter.

○ Leopold, Wolman, and Miller 1964
◆ Colorado data (Wildland Hydrology)

The use of reference dimensionless sediment rating curves by stream type and stability rating, (Troendle et al. 2001), as well as hydrology and hydraulic data, are all needed for the stability and design phases. Additional information will be presented in the respective sequential, analytical steps of each phase of the procedure. Local suspended sediment and bed-load data can be converted to regional sediment curves by plotting bankfull and suspended sediment data by drainage area. Examples of suspended sediment data plotted by 1.5-year recurrence interval discharge/drainage area for many regions of the United States as developed from USGS gage data by the U.S. Department of Agriculture (USDA), Agricultural Research Service (ARS) are presented in Simon, Dickerson, and Heins (2004). These relations can be used if a direct measurement of bankfull sediment cannot be obtained for subsequent analysis. Caution should be exercised in using an arbitrary bankfull value without field calibration of the bankfull discharge. The 1.5-year recurrence interval discharge is often greater than the actual bankfull value in wet climates and urban areas.

The disadvantage of using various suspended and bed load equations for the Rosgen geomorphic channel design methodology is the difficulty of determining sediment supply for sediment rating curves. It is common in the use of these models to have predicted values of many orders of magnitude different than observed values. The use of developed dimensionless ratio sediment rating curves for both suspended (less wash load) and bed load by stream type and stability is the improvement of predicted versus observed values. Results of an independent test of predicted versus observed values for a variety of USGS gage sites are shown in figures 11–12, 11–13, and 11–14. These figures show that predicted sediment rating curves match observed values for a wide range of flows. The model for bed-load transport reflects sediment transport based on changes in the channel hydraulics from a reference condition.

Validation of sediment competence or entrainment relations can also assist in the development and application of subsequent analysis. These data can be collected by installing scour chains and actual measurements of bed-load transport grain size for a given shear stress using Helley-Smith bed-load samplers. Plotting existing data collected by others in this manner can also help in developing a data base used in later analysis.

The use of reference dimensionless ratio sediment rating curves (bed load and suspended less wash load) requires field measured bankfull sediment and dis-

Figure 11–12 Comparison of predicted sediment rating curve to observed values from the Tanana River, AK, using the Pagosa Springs dimensionless ratio relation

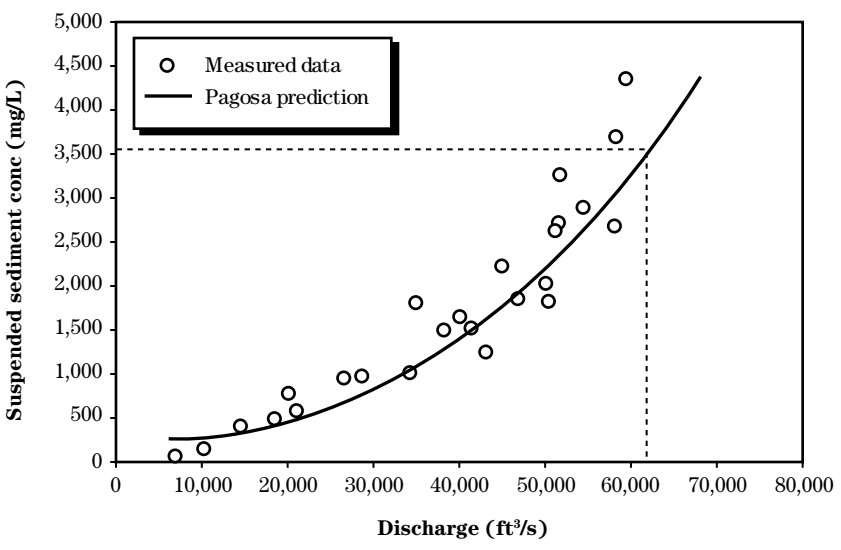
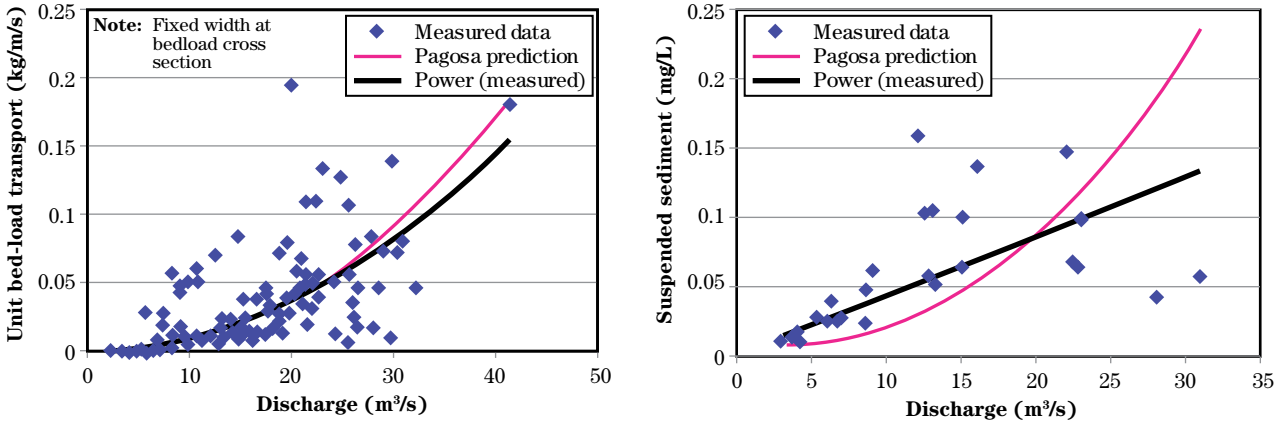
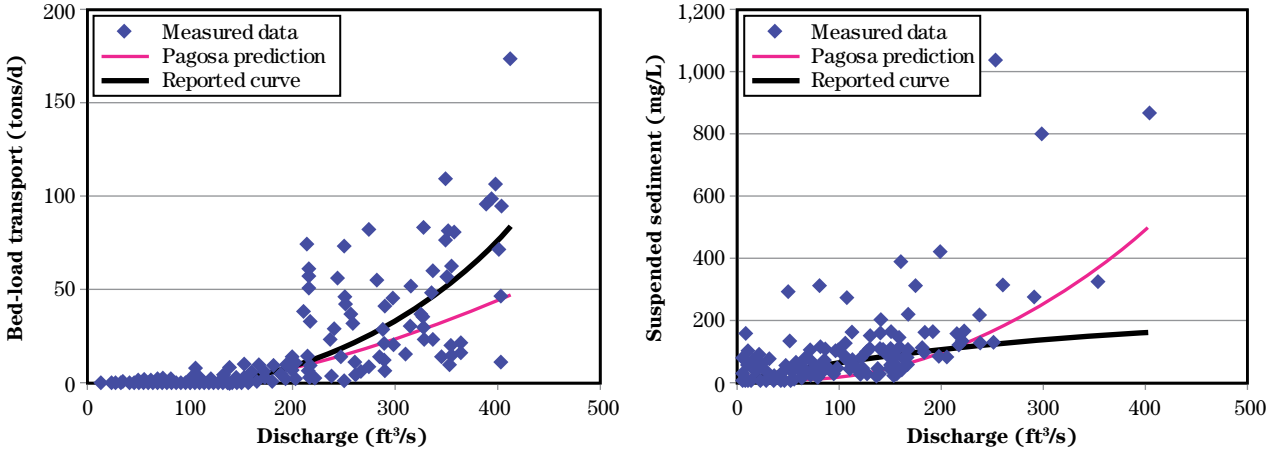


Figure 11–13 Predicted vs. measured sediment data using reference dimensionless rating curve (data from Leopold and Emmett 1997; Ryan and Emmett 2002)

East Fork River near Big Sandy, WY (from Leopold and Emmett 1997)



Little Granite Creek near Bondurant, WY (from Ryan and Emmett 2002)



Maggie Creek (F4)—NV

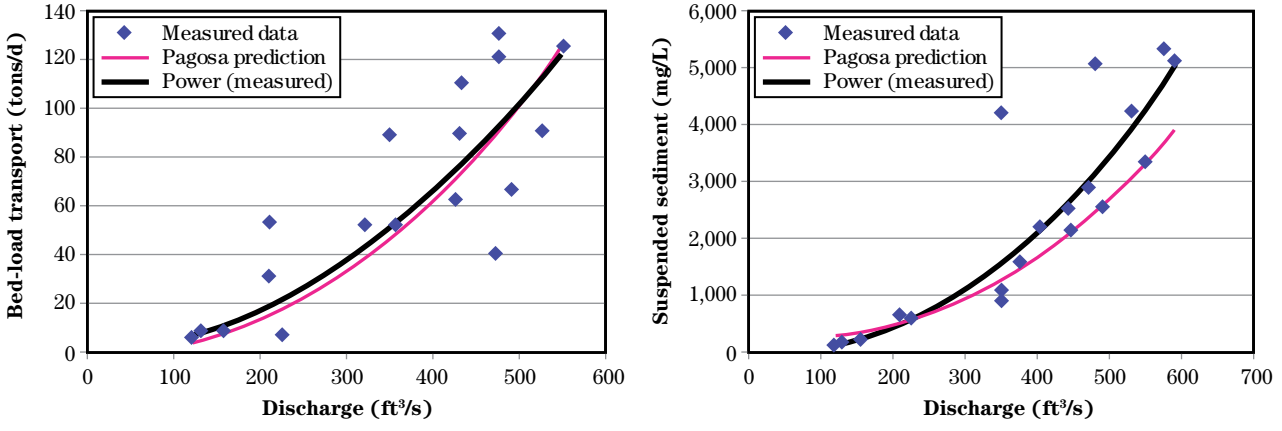
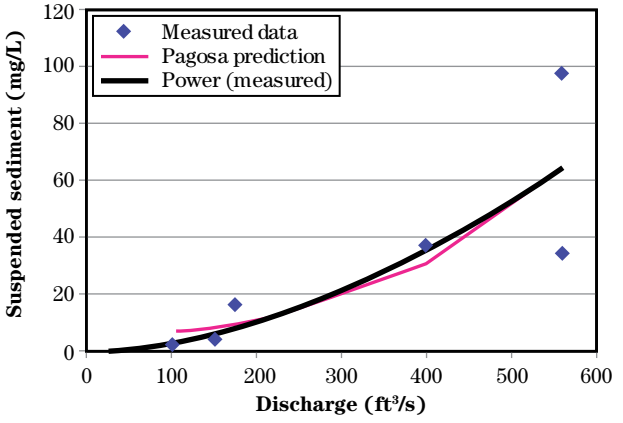
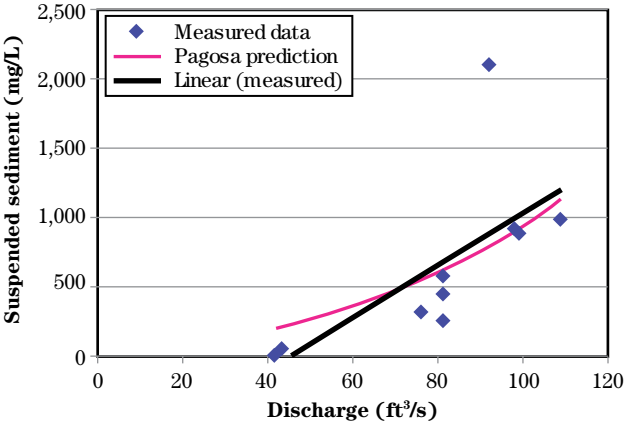


Figure 11–14 Predicted vs. measured suspended sediment data using dimensionless reference curve (data from Emmett 1975)

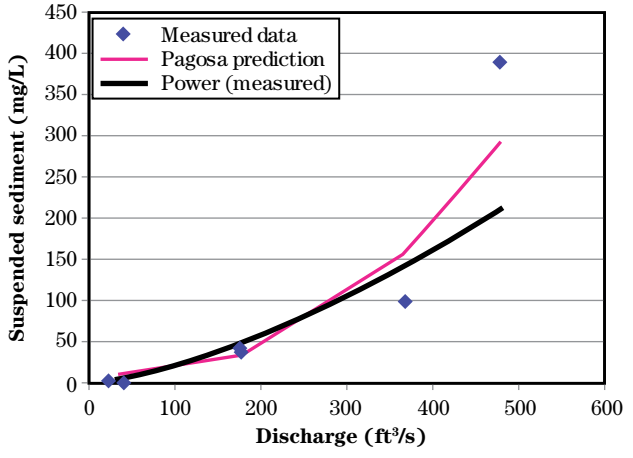
Warm Springs Creek near Clayton, ID 13297000



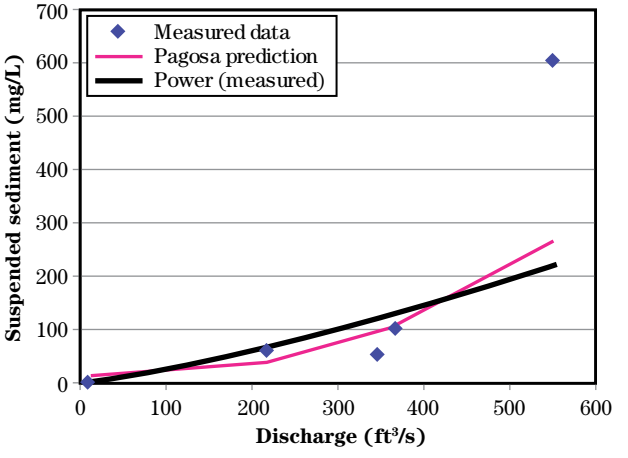
Upper Salmon Watershed 13297250



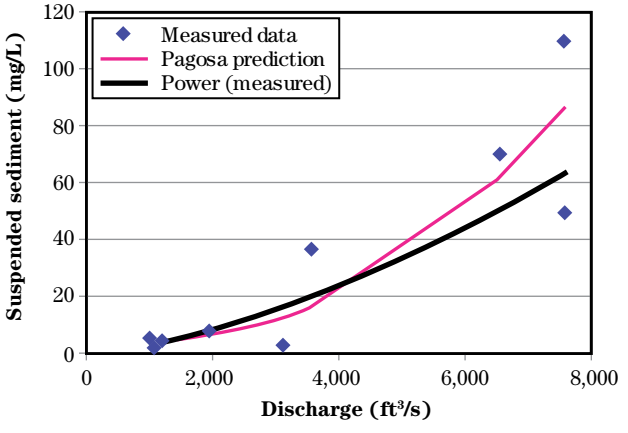
Upper Salmon Watershed 13297340



Upper Salmon Watershed 13297360



Upper Salmon Watershed 13297380



Upper Salmon Watershed 13297425

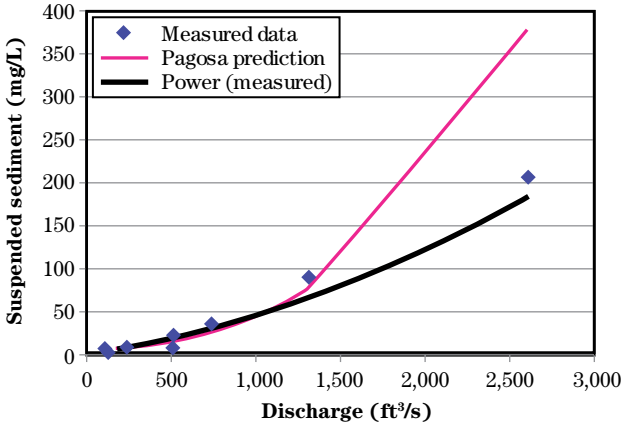
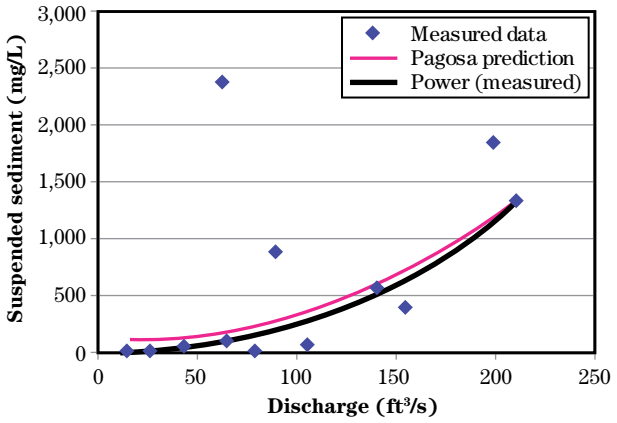
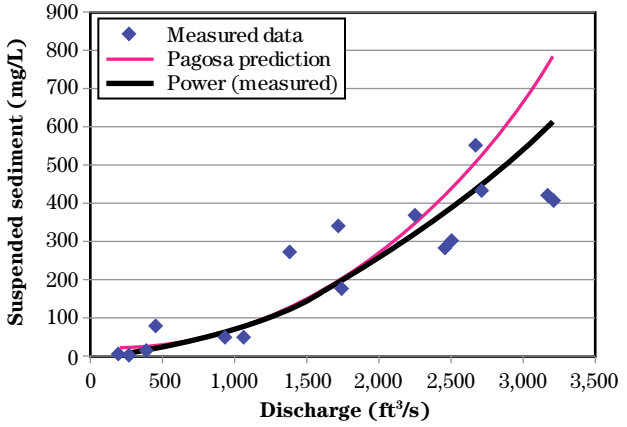


Figure 11–14 Examples of predicted vs. measured suspended sediment data using dimensionless reference curve (data from Emmett 1975)—Continued

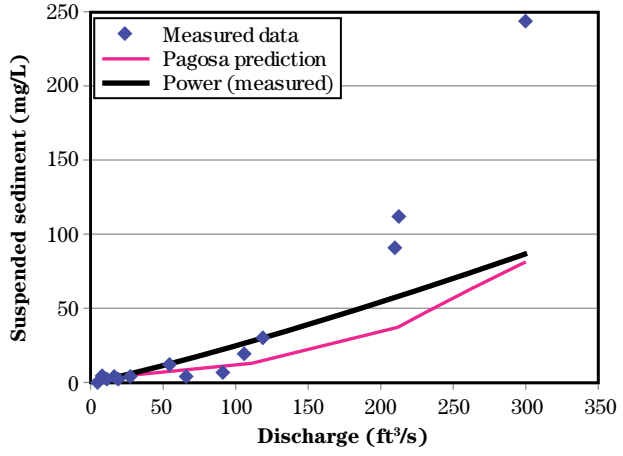
Big Boulder Creek near Clayton, ID 13297500



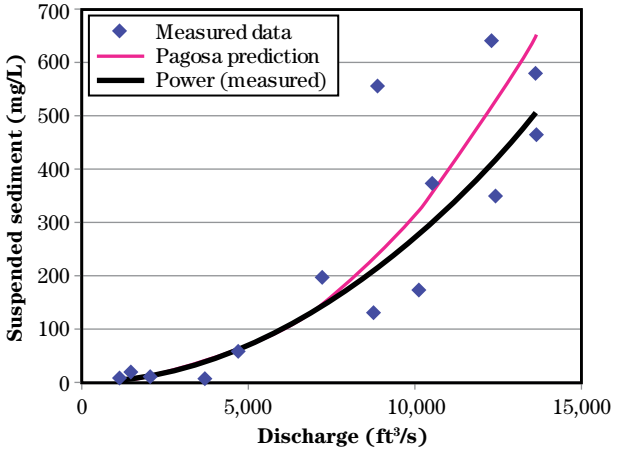
East Fork Salmon River near Clayton, ID 13298000



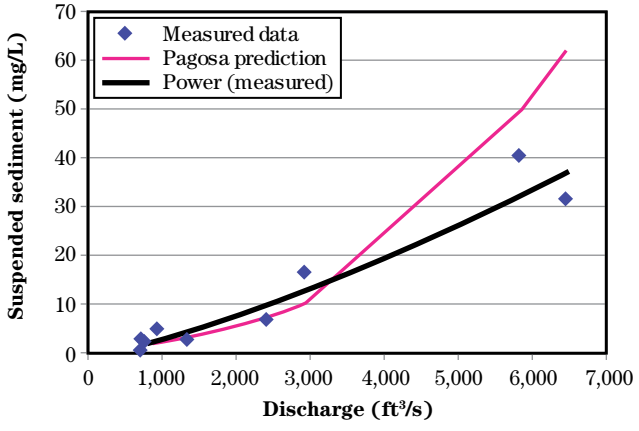
Little Boulder Creek near Clayton, ID 13297450



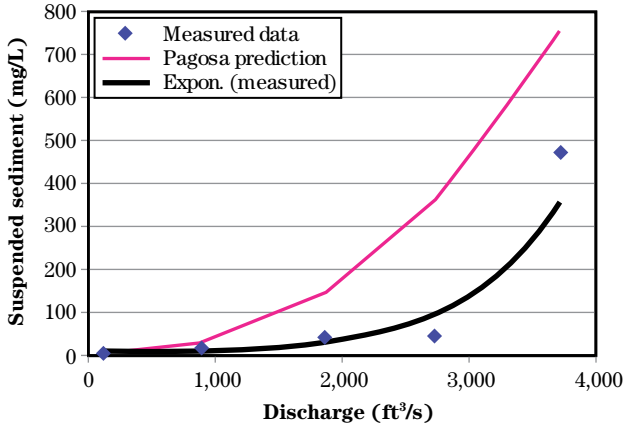
Salmon River near Challis, ID 13298500



Salmon River below Yankee Fork 13296500



Yankee Fork Salmon River near Clayton, ID 1329600



charge. Regional bankfull sediment relations versus drainage area may be substituted if actual bankfull measurements are impossible to obtain, but must be extrapolated from streams of similar lithology, stream type, and stability. Examples of such relations using 1.5-year recurrence interval discharge for suspended sediment are shown in Simon, Dickerson, and Heins (2004). Dimensionless flow-duration curves are also used to produce total annual sediment yield once dimensionless ratio sediment and flow-duration curves are converted to dimensional relations. The examples of predicted sediment rating curves to observed values using a dimensionless sediment rating curve were presented in figures 11–12 to 11–14. Changes in unit stream power (eq. 11–7) are calculated to determine changes in transport rate due to change in depth, slope, and/or velocity. Dimensionless flow-duration curves are used to generate total annual sediment yield from the generated sediment rating curves and bed-load transport by unit stream power.

Streambank erosion—Streambank erosion rate (lateral erosion rate and sediment, tons/yr) is predicted as part of the river stability assessment. The influence of vegetative change, direct disturbance, and other causes of bank instability is quantitatively assessed. One of the major consequences of stream channel instability is accelerated streambank erosion and associated land loss. Fish habitat is adversely affected not only due to increased sediment supply but also by changes in pool quality, substrate materials, imbrication, and other physical habitat loss. Water temperatures are also adversely affected due to increases in width-to-depth ratio due to lateral accretion. The prediction methodology is presented in Rosgen (1996) and in Rosgen (2001d) utilizing a Bank Erodibility Hazard Index (BEHI) and Near Bank Stress (NBS) calculations.

Successional stages of channel evolution—A useful tool at this phase is the determination of various stream type scenarios and stages of channel evolution as depicted in figure 11–15. It is imperative to identify the present stage of the stream and predict the direction and consequence of change. The various stages and scenarios depicted in figure 11–15 assist the observer in this assessment. River channels undergo morphological change due to various disturbance and/or recovery (Rosgen 1996, 2001d, 2005). The assessment phase must identify current states and scenarios. For each state within a scenario, there are specific morphological, sedimentological, hydraulic, and biological relations depicted. The associated interpretations of these relations assist in river assessments.

River stability analysis—Additional stability variables are required for assessment, including the influence of large woody material, flow regime, depositional features, meander patterns, riparian vegetation, and channel stability ratings by stream type, and are summarized in the form shown in table 11–4.

Figure 11–15 Various stream type succession scenarios

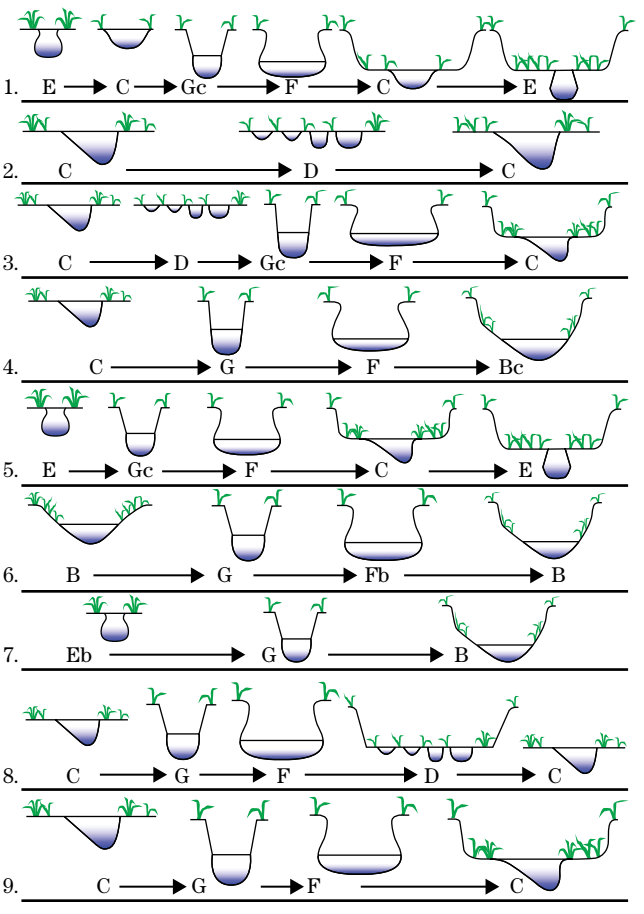


Table 11–4 Stream channel stability assessment summary form

Stream					**Stream**												**Date**				**Observers**							
Level III variables	Stream type		Flow regime		Stream size		Stream order		Meander pattern								Depositional pattern				Debris/channel blockage							
	Riparian vegetation		Current composition/density				Potential composition/density										Altered channel state (dimension, pattern, profile, materials)											
Channel dimension	Mean bankfull depth (ft)				Mean bankfull width (ft)				Cross section area (ft^2)				Remarks															
Channel dimension relationships	Width/depth ratio (W/D)				Reference condition width/depth ratio (W/D_{ref})				(W/D)/ (W/D_{ref})				Circle		Stable		Moderately unstable		Unstable		Highly unstable							
Channel pattern	Mean (range)		MWR				Lm/W_{bkf}						Rc/W_{bkf}						Sinuosity									
River profile and bed features	Circle		Riffle/pool		Step/pool		Plane bed		Convergence/divergence				Dune/antidunes/smooth bed															
	Max bankfull depth (ft)		Riffle		Pool		Depth ratio (max/mean)		Riffle		Pool		Pool to pool spacing				Slope											
																	Valley				Average bankfull							
Channel stability rating	Pfankuch rating								Pfankuch adjusted by stream type (use potential/reference reach)																			
Bank erosion summary	Length of reach studied (ft)				Annual streambank erosion rate								Curved used				Remarks											
							(ton/yr)				ton/yr/ft																	
Degree of confinement	Reference		MWR/				Unconfined				Moderately confined						Confined				Severely confined							
	MWR		Reference MWR				(1.0–0.80)				(0.79–0.30)						(0.29–0.1)				(<0.1)							
Lateral stability	Circle		Stable		Moderately unstable				Unstable		Highly unstable (accelerated lateral erosion)																	
Sediment capacity	Sufficient capacity				Insufficient capacity																							
Stream channel scour/deposition	Largest particle-bar sample (mm)				τ_{ci}		Existing $depth_{bkf}$				Required $depth_{bkf}$				Existing $slope_{bkf}$				Required $slope_{bkf}$									
Degree of incision	Bank height ratio				Stable (no incision)		Slightly incised		Moderately incised		Deeply incised		Width of flood prone area (ft)						Entrenchment ratio									
Channel enlargement	Circle		Stable		Slight		Moderate		Extensive																			
Stream successional stage		→		→		→		→		→			Existing stream state (type)						Potential stream state (type)									
Vertical stability	Circle		Stable				Aggradation				Degradation																	
Sediment supply (channel source)	Circle		Very high		High		Moderate		Low		Score		Remarks/causes															

Base-level change—A key part of channel stability analysis. Degree of channel incision (lowering of local base level) is determined by the ratio of the lowest bank height divided by maximum bankfull depth, called the bank height ratio. A stream may not be entrenched (vertically constrained), but may be partially incised, leading to entrenchment. A grade-control structure requirement is often associated with partially incised channels (Rosgen 1997a).

Direct disturbance and riparian vegetation—The direct disturbance of stream channels must be offset by correcting dimension, pattern, profile, and often channel materials. Levees adjacent to both banks should be set back allowing room for a flood plain. Riparian vegetation change is not only a major cause of instability and loss of function, but is a key solution in restoration and natural channel design. Riparian vegetation reestablishment should contain the correct overstory and understory species to be compatible for a self-sustaining, long-term solution.

Biological assessments—Biological assessments that describe fish species, food chains, diversity with broad categories of ecoregions, and stream types (habitat units) are currently collected with the assessment level for identifying biological potential. Limiting factor analysis provides information that identifies specific problems that may be corrected by changed management and/or restoration.

It is readily apparent that this procedure involves extensive field observations and an extensive data base followed by a thorough and detailed analysis. All of this must be completed prior to restoration planning, as it forms much of the foundation for what follows.

It is important to understand the various causes of instability responsible for loss of physical and biological function and corresponding loss of value. Recommendations that follow are critically linked to land uses, disturbance regime, and other problem sources. The flowchart (fig. 11–10) depicts the assessment criteria of channel stability.

(d) Phase IV—Passive recommendations for restoration

A first priority in restoration is to seek a natural recovery solution based on changes in the variables causing the instability and/or loss of physical and biological function. Changes in land use management can influence riparian vegetation composition, density and vigor, flow modifications (diversions, storage, and reservoir release schedule modifications based on the operational hydrology), flood control measures, road closures/stabilization, hillslope erosional processes, and other process influences of river stability. Often, a change in management strategies can be very effective in securing stability and function. This often has to be determined based on the recovery potential of various stream types and the short- and long-term goals associated with the stated objectives (including costs). The alternative of self-stabilization is always a key consideration in any stability assessment. The time-trend aerial photography from phase III may help to provide insight into stream recovery potential following disturbance.

Successional stages of channel adjustment (fig. 11–15) can also assist at looking at natural recovery potential. It is very important to ensure that objectives are met through effectiveness monitoring required to provide the documentation on the nature, magnitude, rate, and consequences of natural recovery. If natural recovery potential is poor and/or does not meet specific objectives, phase V would be appropriate (Rosgen geomorphic channel design methodology).

(e) Phase V—The stream restoration and natural channel design using the Rosgen geomorphic channel design methodology

Phase V involves combining the results of the previous phases. A good design can only follow a good assessment. It is preferred not to patch symptoms, but rather provide solutions to restoration that will offset the cause of the problem and allow for the river to be self-maintaining. The practitioner must be very familiar with the processes involved in hydrology, hydraulics, sedimentology, geomorphology, soil science, aquatic habitat, and riparian vegetation. Due to the inherent complexity, it is usually necessary to obtain technical

assistance for assessment and design, depending on the practitioner's experience and training.

The conceptual, generalized flowchart shown in figure 11–16 depicts the general sequence of the mixed use of analog, empirical, and analytical methods in this design procedure. The early sequence is required to determine the existing valley type and potential stream type of the stable form. The proposed channel type must be converted to a dimension, pattern, and profile to initially test whether the hydraulic and sediment relations associated with the watershed are compatible prior to advancing through all of the procedural steps.

The watershed and river assessment that predicts the consequence of streamflow, sediment supply, and channel change is reflected in figure 11–17. The procedure is incorporated into the following sequential analysis steps.

Figure 11–16 Generalized flowchart representing Rosgen geomorphic channel design utilizing analog, analytical, and empirical methodologies

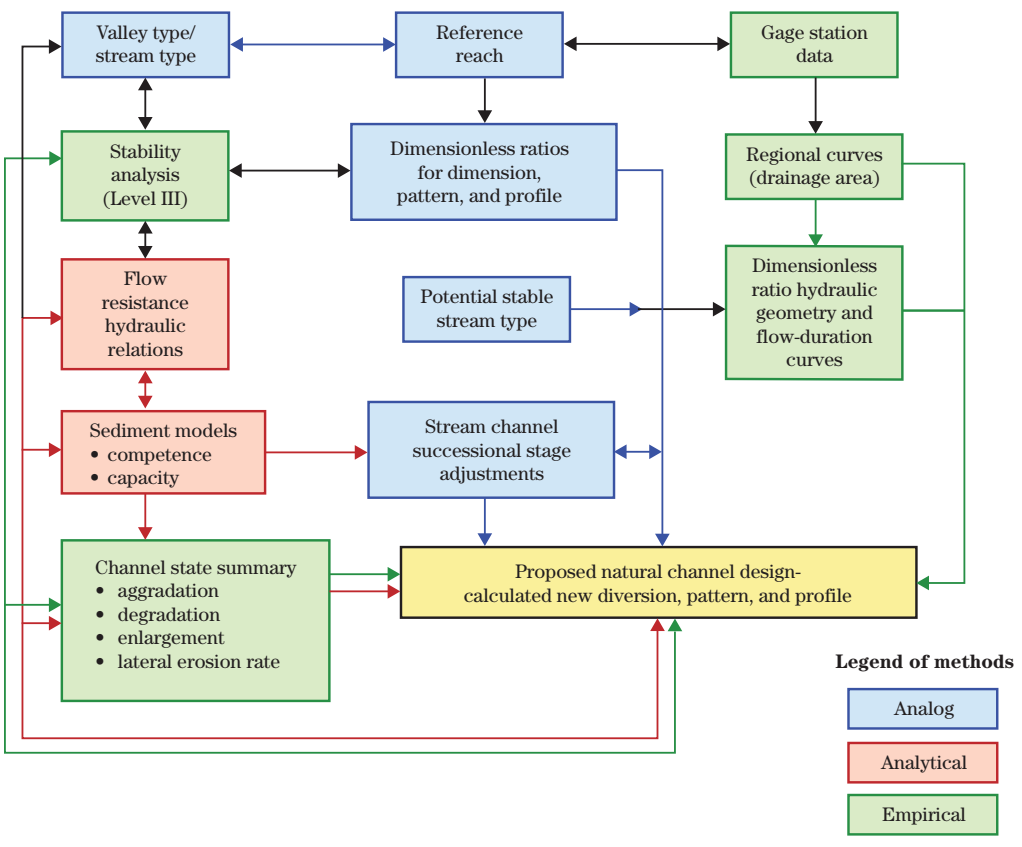
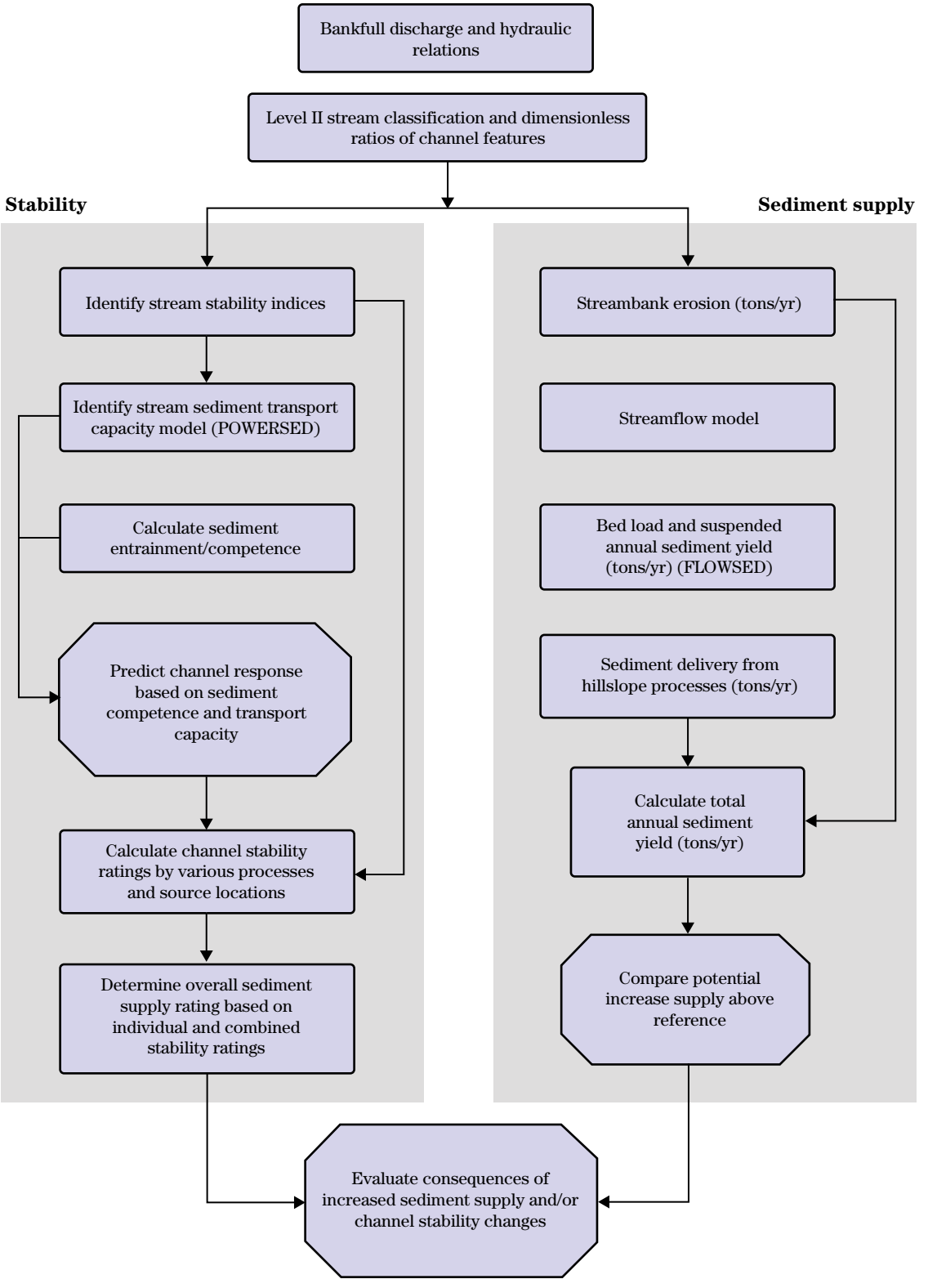


Figure 11–17 Flowchart for determining sediment supply and stability consequences for river assessment



The procedural sequence utilized in the Rosgen geomorphic channel design methodology is shown in the following operational steps:

Step 1 Obtain and/or verify regional curves (bankfull discharge, cross-sectional area, width and depth versus drainage area). The regional curves must be located in the same hydro-physiographic province as that of the restoration reach.

Step 2 Obtain hydraulic geometry (USGS 9–207 forms, summary of current meter measurements) from the gage station stratified by stream type and bed features.

Step 3 Create dimensionless hydraulic geometry by dividing all values by the bankfull value.

Step 4 Obtain flow-duration curves from the gage station for a representative hydro-physiographic region.

Step 5 Create dimensionless flow-duration curve by dividing all flow values by the bankfull discharge.

Step 6 Identify the valley type for the restoration reach(s). Identify stream type(s) of the restoration reach.

Step 7 Obtain corresponding reference reach data for the same valley and stream type. The reference reach is not required to be located within the same watershed or hydro-physiographic province. Examples of the dimensionless ratio and other reference reach data by stream type/valley type are presented in table 11–3.

Step 8 Complete and/or review the stability examination data for the restoration reach (fig. 11–10 and table 11–4). Evaluate variables/states that represent instability relations (width, depth, and slope values that do not meet sediment transport requirements).

Step 9 Select appropriate scenario of successional stages of channel adjustment for channel evolution scenario (fig. 11–15). This determines the stream type of the current state and the potential state to match the valley type. (This step is completed in the stability phase, phase III).

Step 10 Obtain drainage area (mi^2) for the restoration reach.

Step 11 Obtain bankfull cross-sectional area (A_{bkf}) from the regional curves (step 1).

Step 12 Obtain reference reach width-to-depth ratio associated with the stable design stream type commensurate with the valley type (step 7).

Step 13 Calculate design bankfull channel width of riffle reach:

$$W_{bkf} = \left[\left(\frac{W_{bkf}}{d_{bkf}}\right)_{ref} A_{bkf}\right]^{\frac{1}{2}} \qquad \text{(eq. 11– 8)}$$

Step 14 Calculate mean riffle depth:

$$d_{bkf} = \frac{A_{bkf}}{W_{bkf}} \quad \text{or} \quad \left[\frac{W_{bkf}}{\left(\frac{W_{bkf}}{d_{bkf}}\right)_{ref}}\right] \qquad \text{(eq. 11–9)}$$

Step 15 Calculate meander wavelength (Lm) for average and range of values. Obtain meander length ratio average and range of values, where:

$$MLR = \left[\left(\frac{Lm}{W_{bkf}}\right)_{ref}\right] \text{ from reference reach data (step 7, table 11–3).}$$

$$Lm = \left[\left(MLR_{ref}\right)\right] W_{bkf} \text{ (from step 13)} \qquad \text{(eq. 11–10)}$$

Step 16 Calculate belt width (W_{blt}) for average and range of values from meander width ratios (MWR).

$$MWR = \left[\left(\frac{W_{blt}}{W_{bkf}}\right)_{ref}\right] \quad \text{(step 7, table 11–3).}$$

$$W_{blt} = [(MWR)_{ref}]\ W_{bkf} \qquad \text{(eq. 11–11)}$$

Step 17 Calculate radius of curvature (Rc) for average and a range of values from ratio of radius of curvature ratio. (step 7, table 11–3).

$$Rc = \left[\left(\frac{Rc}{W_{bkf}}\right)_{ref}\right] W_{bkf} \qquad \text{(eq. 11–12)}$$

Step 18 Obtain an aerial photo depicting vegetation, channel features and terrain character. Layout the range of values for meander length (Lm), belt width (W_{blt}) and radius of curvature (Rc) on aerial photo or detailed topographic map. Adjust pattern to utilize terrain features and existing vegetation where possible within the range of the

pattern variables. Once the preliminary layout is complete, measure stream length (SL) of the proposed channel. Measure valley length (VL) by following the fall line of the valley, rather than straight line segments between meanders.

Step 19 Calculate sinuosity (k) of the proposed channel where:

$$k = \frac{SL}{VL} \qquad \text{(eq. 11–13)}$$

Step 20 Calculate valley slope (S_{val}). Measure the water surface elevation difference (DE) between the same bed features along the fall line of the valley using valley length (VL), where:

$$S_{val} = \frac{DE}{VL} \qquad \text{(eq. 11–14)}$$

Step 21 Calculate proposed channel average slope (S):

$$S = \frac{S_{val}}{k} \qquad \text{(eq. 11–15)}$$

Step 22 Calculate bankfull channel velocity (u_{bkf}) and check design bankfull discharge with velocity, cross-sectional area (continuity) regional curves:

$$uA = Q \qquad \text{(eq. 11–16)}$$

$$\frac{Q}{A} = u \quad \text{Compare to regional curve (step 1)} \qquad \text{(eq. 11–17)}$$

Steps 23 through 26 Predict stream competence (entrainment) by utilizing particle entrainment computations. A general flowchart depicting the procedural steps is shown in figure 11–18.

First, obtain bar sample gradation from field sampling and sieving procedure upstream of the proposed restoration (Rosgen 1996). A field procedure for bar sampling, pavement/subpavement sample and wet-sieving onsite is presented in tables 11–5 and 11–6. The user is advised to review additional details of particle size sampling by Bunte and Abt (2001). Sediment sampling is also addressed in NEH654 TS13A. Bar samples are field-sieved and recorded in the entrainment worksheet (table 11–7).

The sediment competence computations that determine bed stability (aggradation/degradation) are completed and summarized in table 11–8. This method has shown consistency when actual bed-load/scour chain data are compared to predicted values. Use the value of the largest particle in the bar sample (or subpavement sample), D_{max} in millimeters, and the revised Shields diagram to predict the shear stress required to initiate movement of the largest particle in the bar and/or subpavement (fig. 11–11).

If the protrusion ratios described in equations 11–3 or 11–4 are outside the ranges indicated in table 11–8, the user should use the shear stress equation (eq. 11–2) and apply it with a revised Shields relation using Colorado data or local data if available (fig. 11–11).

$$\tau^* = 0.0834\left(\frac{D_{50}}{\hat{D}_{50}}\right)^{-0.872} \qquad \text{(eq. 11–3)}$$

$$\tau^* = 0.0384\left(\frac{D_{max}}{D_{50}}\right)^{-0.887} \qquad \text{(eq. 11–4)}$$

$$\tau = \gamma RS \qquad \text{(eq. 11–2)}$$

A grain size corresponding with shear stress is selected to determine what sizes the river can potentially move. Based on measured bed-load sizes, in a heterogeneous mixture of bed material comprised of a mixture of sand to gravel and cobble, the previously published Shields relation generally underestimates particle sizes of heterogeneous bed material in the shear stress range of 0.05 pounds per square foot to 1.5 pounds per square foot. The Shields relationship is appropriately used for entrainment sizes below and/or above this value range. Without this adjustment, most computations underestimate the largest sizes of heterogeneous bed material moved during bankfull discharge. The measured data in figure 11–11 indicate the magnitude of the underestimate of particle size entrainment from comparing published relations to measured values.

To determine the ability of the existing stream reach to transport the largest clast size of the bed-load sediment, it is necessary to calculate the bankfull dimensionless shear stress (τ^*). This calculation determines the depth and slope necessary to mobilize and transport the largest particle made available to the channel. The dimensionless shear stress at bankfull stage is used in the entrainment

Figure 11–18 Generalized flowchart depicting procedural steps for sediment competence calculations

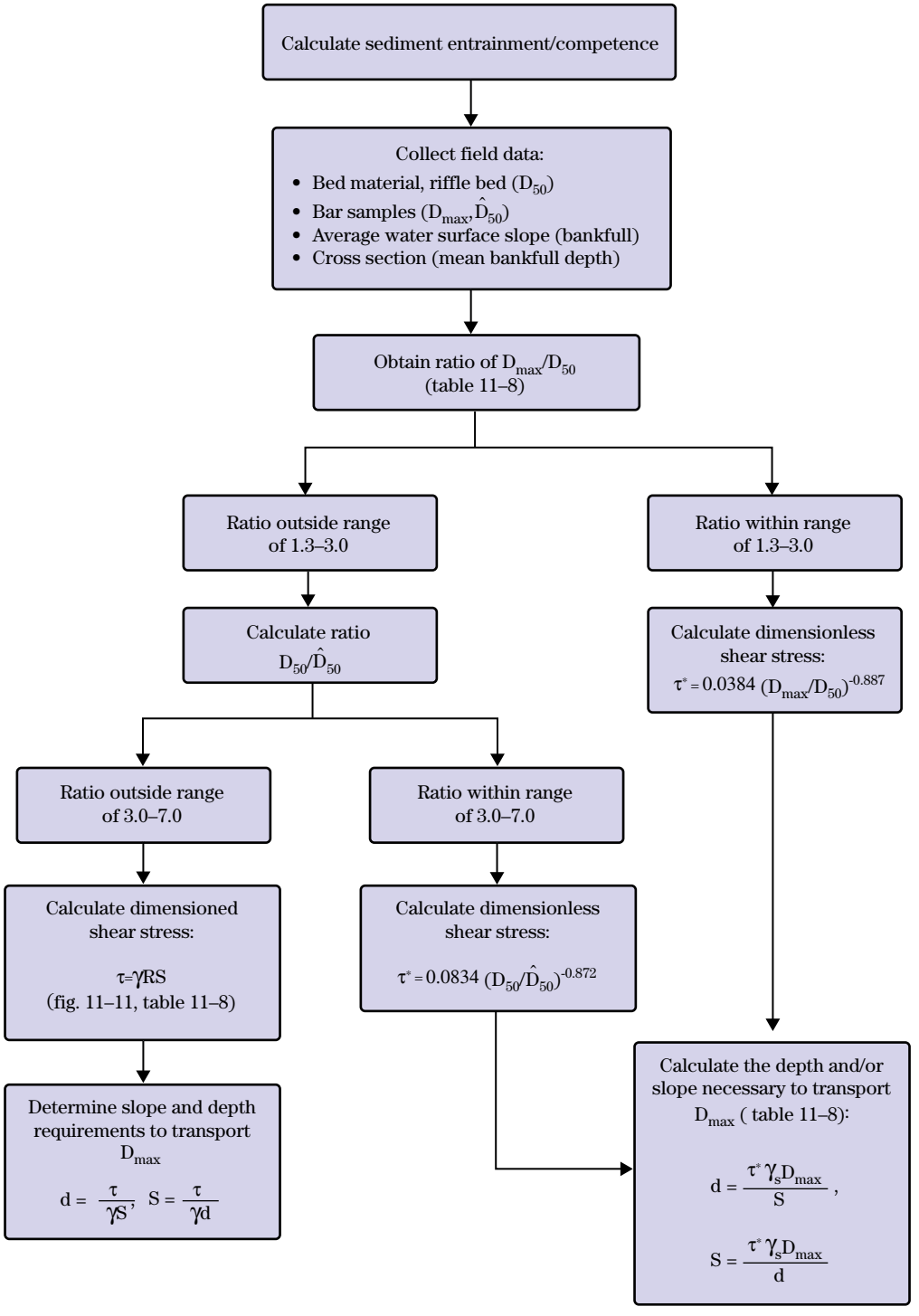


Table 11–5 Field procedure for bar samples*

Bar sample field procedure
Collect sediment core samples from point bars along the project and reference reaches. At least one sample should be collected from each reach associated with a change in stream type. Conduct a critical shear stress analysis using the following procedures:
Locate a sampling point on the downstream a third of a meander bend. The sample location on the point bar is halfway between the thalweg elevation (the point of maximum depth) and the bankfull stage elevation. Scan the point bar in this area to determine the sampling location by observing the maximum particles on the surface of the bar.
Place a 5-gallon bottomless bucket at the sampling location over one of the representative larger particles that are observed on the lower third of the point bar. Remove the two largest particles from the surface covered by the bottomless bucket. Measure the intermediate axis for each particle and individually weigh the particles. Record these values. The largest particle obtained is D_{max}, the largest particle from the bar sample. Push the bottomless bucket into the bar material. Excavate the materials from the bottomless bucket to a depth that is equal to twice the intermediate axis width of the largest surface particle. Place these materials in a bucket or bag for sieving and weighing.
For fine bar materials, follow the directions above, except that when the bottomless bucket is pushed into the bar material, excavate materials from the bucket to a depth of 4 to 6 inches. Place these materials in a bucket or bag for sieving and weighing.
Wet-sieve the collected bar materials using water and a standard sieve set with a 2-millimeter screen size for the bottom sieve. Weigh the bucket with sand after draining off as much water as possible. Subtract the tare weight of the bucket to obtain the net weight of the sand.
Weigh the sieved materials and record weights (less tare weight) by size class. Be sure to include the intermediate axis measurements and individual weights of the two largest particles that were collected.
Determine a material size class distribution for all of the collected materials. The data represents the range of channel materials subject to movement or transport as bed-load sediment materials at bankfull discharge.
Plot data; determine size-class indices, D_{16}, D_{35}, D_{50}, D_{84}, D_{95}. The D_{100} should represent the actual intermediate axis width and weight (not the tray size) when plotted. The largest size measured will be plotted at the D_{100} point (Note: $D_{100} = D_{max}$). The intermediate axis measurement of the second largest particle will be the top end of the catch range for the last sieve that retains material (use the record data in the entrainment worksheet, table 11–7).
Survey a typical cross section of a riffle reach at a location where the stream is free to adjust its boundaries. Plot the survey data. Determine the hydraulic radius of the cross section.
Conduct a Wolman Pebble Count (100 count in riffle) of the bed material in the coarsest portion of the wetted riffle area (active channel). The pebble count should be conducted at multiple transects that represent the riffle. Plot data and determine the size-class indices.

*Sediment sampling is also addressed in NEH654 TS13A.

Table 11–6 Field procedure for pavement/sub-pavement samples

Pavement/subpavement sample field procedure (alternate procedures for obtaining a pavement/sub-pavement sample if you are unable to collect a bar sample)
Locate a sampling point in the same riffle where cross-sectional survey was conducted. The sampling point should be to the left or right of the thalweg, not in the thalweg, in a coarse-grain size portion of the riffle.
Push a 5-gallon bottomless bucket into the riffle at the sampling location to cut off the streamflow. The diameter of the bucket (sample size) should be at least twice the diameter of the largest rock on the bed of the riffle.
Remove the pavement material (surface layer only) by removing the smallest to the coarsest particles. Measure the intermediate axis and weight of the largest and second largest particles. Record these values. Place the remaining pavement materials into a bucket or bag for sieving and weighing.
Remove the sub-pavement material to a depth that is equal to twice the intermediate axis width of the largest particle in the pavement layer, or at least 150-millimeter depth. Caution: if a coarser bed material persists under the sub-pavement, it generally is material remnant of the previous bed. Stop at this condition and do not excavate deeper, even if the depth is not at twice the maximum pavement particle diameter. This residual layer is generally not associated with the size distribution of bed load transported at the bankfull stage. Collect the sub-pavement materials into a separate bucket or a bag. Measure the intermediate axis and weight of the two largest particles in the sub-pavement sample. Record these values. Sieve and weigh the remaining sub-pavement materials. The sub-pavement sample is the equivalent of the bar sample; therefore, use the largest particle from the sub-pavement sample in lieu of the largest particle from a bar sample in the entrainment calculations. Note: If the largest particle collected from the sub-pavement is larger than the pavement layer, the largest rock should be discarded from the sub-pavement layer. Drop back to the next largest particle size to determine the largest particle size to be used in the entrainment calculation.
Wet-sieve the collected pavement materials and then the subpavement materials using water and a standard sieve set with a 2-millimeter screen size for the bottom sieve. Weigh the bucket with sand after draining off as much water as possible. Subtract the tare weight of the bucket to obtain the net weight of the sand.
Weigh the sieved materials and record weights (less tare weight) by size class for both the pavement and sub-pavement samples. Be sure to include the mean intermediate axis width and individual net weights of the two largest particles that were collected (table 11–7).
Determine a material size-class distribution for the materials. The subpavement data represent the range of channel materials subject to movement or transport as bed-load sediment materials at bankfull discharge.
Plot data; determine size-class indices, D_{16}, D_{35}, D_{50}, D_{84}, D_{95}. The D_{100}, should represent the actual intermediate axis width and weight (not the tray size) when plotted. The largest size measured will be plotted at the D_{100} point. (Note: $D_{100} = D_{max}$). The intermediate axis measurement of the second largest particle will be the top end of the catch range for the last sieve that retains material.
The pavement material size class distribution may be used to determine the D_{50} of the riffle bed instead of doing the 100 count in the riffle bed.
Determine the average bankfull slope (approximated by the average water surface slope) for the study reach from the longitudinal profile.
Calculate the bankfull dimensionless shear stress required to mobilize and transport the largest particle from the bar sample (or sub-pavement sample). Use the equations and record the data in the entrainment worksheet (table 11–8).

Table 11–7 Bar sample data collection and sieve analysis form

Point / Side BAR-BULK MATERIALS SAMPLE DATA: Size Distribution Analysis

Party:

Location:

Date:

Notes:

SUB SAMPLES	Sieve SIZE		Sieve SIZE		Sieve SIZE		Sieve SIZE		Sieve SIZE		Sieve SIZE		Sieve SIZE		Sieve SIZE		Sieve SIZE	
	Tare Weight		Tare Weight		Tare Weight		Tare Weight		Tare Weight		Tare Weight		Tare Weight		Tare Weight		Tare Weight	
	Sample Weights		Sample Weights		Sample Weights		Sample Weights		Sample Weights		Sample Weights		Sample Weights		Sample Weights		Sample Weights	
	Total	Net	Total	Net	Total	Net	Total	Net	Total	Net	Total	Net	Total	Net	Total	Net	Total	Net
1																		
2																		
3																		
4																		
5																		
6																		
7																		
8																		
9																		
10																		
11																		
12																		
13																		
14																		
15																		
Net Wt. Total																		
% Grand Tot.																		
Accum. % =<																		

SURFACE MATERIALS DATA
(Two Largest Particles)

No.	Dia.	WT.
1		
2		

Bucket
+ Materials
Weight_____

Bucket
Tare
Weight_____

Materials
Weight_____
(Materials less than:
_____mm.)

Be Sure to Add Separate Material Weights to Grand Total

GRAND TOTAL SAMPLE WEIGHT

NOTES										

Table 11–8 Sediment competence calculation form to assess bed stability (steps 23–26)

Stream:		Reach:			
Observers:			Date:		
Enter required information					
	D_{50}	Riffle bed material D_{50} (mm)			
	$\hat{D}_{50}$	Bar sample D_{50} (mm)			
	D_{max}	Largest particle from bar sample (ft)		**(mm)**	304.8 mm/ft
	S	Existing bankfull water surface slope (ft/ft)			
	d	Existing bankfull mean depth (ft)			
1.65	γ_s	Submerged specific weight of sediment			
Select the appropriate equation and calculate critical dimensionless shear stress					
	$D_{50}/\hat{D}_{50}$	Range: 3 – 7	Use equation 1:	$\tau^* = 0.0834\left(\frac{D_{50}}{\hat{D}_{50}}\right)^{-0.872}$	
	D_{max}/D_{50}	Range: 1.3 – 3.0	Use equation 2:	$\tau^* = 0.0384\left(\frac{D_{max}}{D_{50}}\right)^{-0.887}$	
	τ^*	Bankfull dimensionless shear stress		Equation used:	
Calculate bankfull mean depth required for entrainment of largest particle in bar sample					
	d	Required bankfull mean depth (ft)		$d = \frac{\tau^* \gamma_s D_{max}}{S}$	
	Circle:	Stable Aggrading Degrading			
Calculate bankfull water surface slope required for entrainment of largest particle in bar sample					
	S	Required bankfull water surface slope (ft/ft)		$s = \frac{\tau^* \gamma_s D_{max}}{d}$	
	Circle:	Stable Aggrading Degrading			
Sediment competence using dimensional shear stress					
	Bankfull shear stress $\tau = \gamma dS$ (lb/ft^2) (substitute hydraulic radius, R, with mean depth, d)				
	Moveable particle size (mm) at bankfull shear stress (fig. 11-11)				
	Predicted shear stress required to initiate movement of D_{max} (mm) (figure 11-11)				
	Predicted mean depth required to initiate movement of D_{max} (mm)			$d = \frac{\tau}{\gamma S}$	
	Predicted slope required to initiate movement of D_{max} (mm)			$S = \frac{\tau}{\gamma d}$	

analysis for both the reference reach and project reach. This analysis of the reference, stable condition is compared to the potentially disturbed reach. To maintain stability, a stream must be competent to transport the largest size of sediment and have the capacity to transport the load (volume) on an annual basis. These calculations provide a prediction of sediment competence as required in steps 23 through 26.

Step 27 Compute sediment transport capacity. Following this analysis, the depth and/or slope may need to be adjusted by recalculating steps 14 through 27.

FLOWSED and POWERSED are sediment supply/sediment transport models that predict the following:

- total annual suspended sediment yield
- total annual suspended sand sediment yield
- total annual bed-load sediment yield
- potential aggradation and/or degradation
- flow-related annual sediment yield due to changes in streamflow magnitude and duration

The models are based on the use of dimensionless reference sediment rating and flow-duration curves. The normalization parameters include:

- bankfull discharge
- bankfull stage bed load
- suspended and suspended sand sediment

The appropriate dimensionless sediment curves are selected for representative stream types and stability ratings. The dimensionless flow-duration curves are developed from representative hydrophysiographic province data from USGS stream gage data.

The FLOWSED model reflects sediment supply and generates the total annual sediment yield for both suspended and bed load. Changes in flow are also reflected in flow-duration curves and corresponding sediment yield. To determine annual sediment yield, near-bankfull stage values must be field measured to convert dimensionless sediment and flow-duration curves to actual values.

The POWERSED model compares sediment transport capacity from a stable, reference condition by predicting transport rate change due to channel hydraulics. The hydraulics reflect potential change in morphological variables such as channel width, depth, and slope. The corresponding changes in flow resistance are used to predict velocity, shear stress, and unit stream power (velocity multiplied by shear stress). Sediment rating curves from the FLOWSED model are converted from discharge to unit stream power for a wide range of flows. Revised values of annual sediment transport can then be compared to the reference condition from the subsequent change in the hydraulic geometry of the stream channel and corresponding response in sediment transport. Any flow modifications can also be simulated by revised flow-duration curves.

Detailed descriptions and model tests are provided for FLOWSED/POWERSED in Rosgen (2006). This analysis is complicated and detailed. However, it can be computed by spreadsheet or commercially available computer programs (RIVERMorph® 4.0). The basis of the calculations and model descriptions, however, are described to better understand how the models work. Table 11–9 lists the data required to run the FLOWSED and POWERSED models. With these data, the user can generate average annual sediment yields (tons/yr).

Table 11–9 Data required to run the FLOWSED and POWERSED supply/sediment transport models

Data requirements for FLOWSED/POWERSED
• Background reference data (flow and sediment)
– Dimensionless suspended sediment rating curves by stream type or stability
– Dimensionless bed-load rating curves by stream type or stability
– Dimensionless flow duration (from local or representative hydro-physiographic province)
– Momentary maximum bankfull discharge
– Mean daily bankfull discharge (the mean daily discharge the day bankfull occurs at a gage station)
– Flow-duration curves indicating change in flow regime (increase and/or decrease)
• Field measured values (for both reference and impaired condition)
– Cross section
– Longitudinal profile
– Pebble count on active riffle bed to obtain D_{50} and D_{84} of bed material
– Stream classification (level II)
– Pfankuch channel stability rating
– Measured bankfull discharge (ft^3/s)
– Measured suspended sediment (mg/L)
– Measured suspended sand sediment (mg/L)
– Measured bed-load sediment (kg/s) (Helley-Smith bed-load sampler)

FLOWSED

The FLOWSED model is graphically depicted in figures 11–19 and 11–20. The procedure in table 11–10 and accompanying worksheet depicted in table 11–11 provide a more detailed understanding of the model. The following provides insight into the basis of the model.

Predict runoff response—Several applicable models for runoff exist, including TR–55, WRENSS (EPA 1980), the unit hydrograph approach (U.S. Army Corps of Engineers (USACE) 1998b), and others (EPA 1980; Troendle, Swanson, and Nankervis 2005). This step also considers operational hydrology from reservoirs, diversions, and other flow modifications that influence the magnitude, duration, and timing of streamflow. The input variables for most models are precipitation data, a vegetation alteration map by aspect and elevation, drainage area computations, percent of drainage area in impervious condition, and similar data specified based on the specific model being selected. The output from these models needs to be in the form of flow-duration curves. Flow-duration curves must represent reference conditions (full hydrologic utilization or recovery) and existing departures from reference. Because few stream gages are located on smaller watersheds, dimensionless ratio procedures become essential for data extrapolation in flow models. The data are entered into the flow-duration portion of the FLOWSED worksheet (table 11–11).

Develop dimensionless flow-duration curves—If a water yield model or operational hydrology data with actual flow-duration curve data are not available, it will be necessary to utilize dimensionless flow-duration curves. This information is obtained from gage station data and made dimensionless by dividing the mean daily discharge data by bankfull discharge. Bankfull discharge data are divided into all of the ranges of mean daily discharge and then plotted; see figures 11–9 and 11–21 as an example of the application for Weminuche Creek. The user must develop dimensionless flow-duration curves from gaging stations that represent a hydro-physiographic region similar to the impaired stream being assessed. If the user is applying these relations to a stormflow-generated hydrograph, rather than snowmelt (as in the case of Weminuche Creek), the following changes are recommended:

- Convert bankfull discharge (momentary maximum discharge in ft^3/s) to mean daily bankfull. This is accomplished by obtaining the mean daily discharge on the day during which bankfull discharge occurs. This ratio of mean daily discharge divided by momentary maximum discharge is used to develop the dimensionless flow-duration curves for a stormflow-dominated region. For example, if the mean daily discharge from a gage in a stormflow-dominated hydrograph was 125 cubic feet per second, but bankfull was 550 cubic feet per second, the ratio is 0.227. This ratio would be multiplied by the bankfull discharge from the regional curves or from a flood-frequency curve relation to convert bankfull discharge from a momentary maximum to a mean daily discharge value.

- Divide the mean daily discharge values by mean daily bankfull to establish the dimensionless relations similar to those in figures 11–9 and 11–21.

- Convert from dimensionless to dimensioned mean daily bankfull values. The momentary maximum value must be adjusted by the appropriate ratio, then multiplied by the appropriate ratio value in the dimensionless flow-duration curve. The dimensioned flow-duration curve data are entered into the FLOWSED worksheet (table 11–11). This would be done separately for reference or baseline conditions, and then would be compared to impaired or impacted watershed conditions to calculate annual streamflow and sediment yield.

FLOWSED—Continued

Collect bankfull discharge, suspended sediment, and bed-load sediment—This step is eventually used to convert the reference dimensionless sediment rating curves to actual values. It is very important to capture the bankfull discharge and have several data points to compute an average of the flow and sediment values due to the high spatial and temporal variability of sediment movement. Field methods and equipment used should follow the procedures outlined in book 3, chapter C2 of Field Methods for Measurement of Fluvial Sediment (USGS 1999).

It may be necessary to separate the wash load (silt/clay fraction) from the total suspended sediment load for calculation and interpretation. For channel stability purposes, the silt/clay fraction is not energy limited or hydraulically controlled, and in some settings, it can be subtracted from the suspended sediment yield data for the prediction of potential aggradation. This would not be the case, however, if there were concerns over accelerated fine sediment deposition into extremely low-gradient streams, deltas, reservoirs, lakes, marshes, or estuaries. Colloidal sediments can present problems for impaired waters; thus, wash load may need to be retained in suspended sediment analysis. Enter these measurements in the FLOWSED worksheet (table 11–11).

Obtain or establish reference dimensionless suspended and bed-load rating curves—These curves should be developed for stable reference reach sites representing stable streams. A similar relation can be stratified for poor stability or unstable streams. These reference curves are used to establish sediment rating curves for the calculation of flow-related sediment increases and to establish an annual sediment yield estimate for proportioning contributing sediment sources. The equations for these curve relations are used in the FLOWSED worksheet (table 11–11).

Convert dimensionless suspended and bed-load sediment rating curves to actual (dimensioned) values—Convert dimensionless values by multiplying the field-measured bankfull discharge and sediment values by each of the ratios appropriate for the relation selected. Dimensionless ratio bed-load and suspended rating curves are used to convert data to dimensioned rating curves (fig. 11–20). Examples of dimensioned bed-load and suspended sediment rating curves are shown in figures 11–22 and 11–23 for the Weminuche Creek in Colorado. Tests of this relation are reported in the text in figures 11–13, 11–14, and 11–15, where reference dimensionless rating curves were used to establish sediment rating curves.

If it is not possible to obtain measured bankfull discharge, suspended sediment, and bed-load sediment data to convert dimensionless sediment rating curves to actual values, regional curves can be temporarily substituted. The user must obtain drainage area in square miles to calculate bankfull discharge from a similar hydro-physiographic province. The bankfull flow is used to convert the dimensionless flow-duration to dimensioned flow duration. The bankfull discharge is also used to convert the dimensionless discharge portion of the dimensionless bed-load and suspended rating curve to actual values. The sediment data obtained from the drainage area must be derived from existing measured bankfull suspended sediment and bed-load sediment data, then converted to unit area sediment values from the corresponding drainage area. These data need to represent the same lithology, stream type and stability condition of the stream being evaluated. These data are entered in the FLOWSED worksheet (table 11–11).

An example of unit area suspended sediment data from USGS sites throughout the United States is shown in Simon, Dickerson, and Heins (2004). These measured sediment values were separated by evolutionary stages. Additional stability or stream type data may help to identify appropriate relations for extrapolation. This drainage area extrapolation procedure represents only an interim procedure until measured bankfull values can be obtained.

FLOWSED—Continued

Convert dimensionless flow duration to dimensioned flow duration—The bankfull discharge is multiplied by each of the ratios to convert dimensionless data to actual discharge values representing mean daily discharge for each percentile. An example of a dimensioned flow-duration curve using bankfull discharge to convert from the dimensionless relation (fig. 11–21) is shown in figure 11–24.

Calculate annual sediment yield for both suspended and bed-load sediment—This is accomplished by taking the dimensioned flow-duration curve and multiplying flow increments for duration of time in days by the sediment yield associated with that flow. Enter these calculations in the FLOWSED worksheet (table 11–11).

Calculate flow-related sediment yield—This calculation is accomplished using the output of the flow-duration curves showing the increase in magnitude and duration of flow. The post-treatment flows are routed through the calculation in the FLOWSED worksheet (table 11–11). The excess water calculation output from the WRENSS snowmelt model (EPA 1980) or a similar model integrates the flow with flow-duration changes. Dimensionless flow-duration curves are also converted to dimensioned values by multiplication of the bankfull discharge value. Reference conditions for watersheds in relative hydrologic recovery are compared to watersheds where streamflow has been increased or decreased by change in vegetation or by reservoirs and/or diversions.

Stormflow models, such as TR–55, need to be used to compute new bankfull values, converting dimensionless values to new dimensioned flow durations. It is important to calibrate the bankfull discharge, as the precipitation probability for a given antecedent moisture content and runoff curve number that generates the bankfull discharge needs to be determined. Any greater flow will be distributed on flood plains or a flood-prone area if the stream is not entrenched. Thus, flow-related sediment changes are determined by the use of dimensionless sediment rating curves and dimensionless flow-duration curves. Other appropriate models can also be used for this step, based on the user's familiarity with the various models selected. The output required, regardless of the model, is bankfull discharge and pre- and post-treatment flow-duration curves.

Figure 11–19 General overview of the FLOWSED model

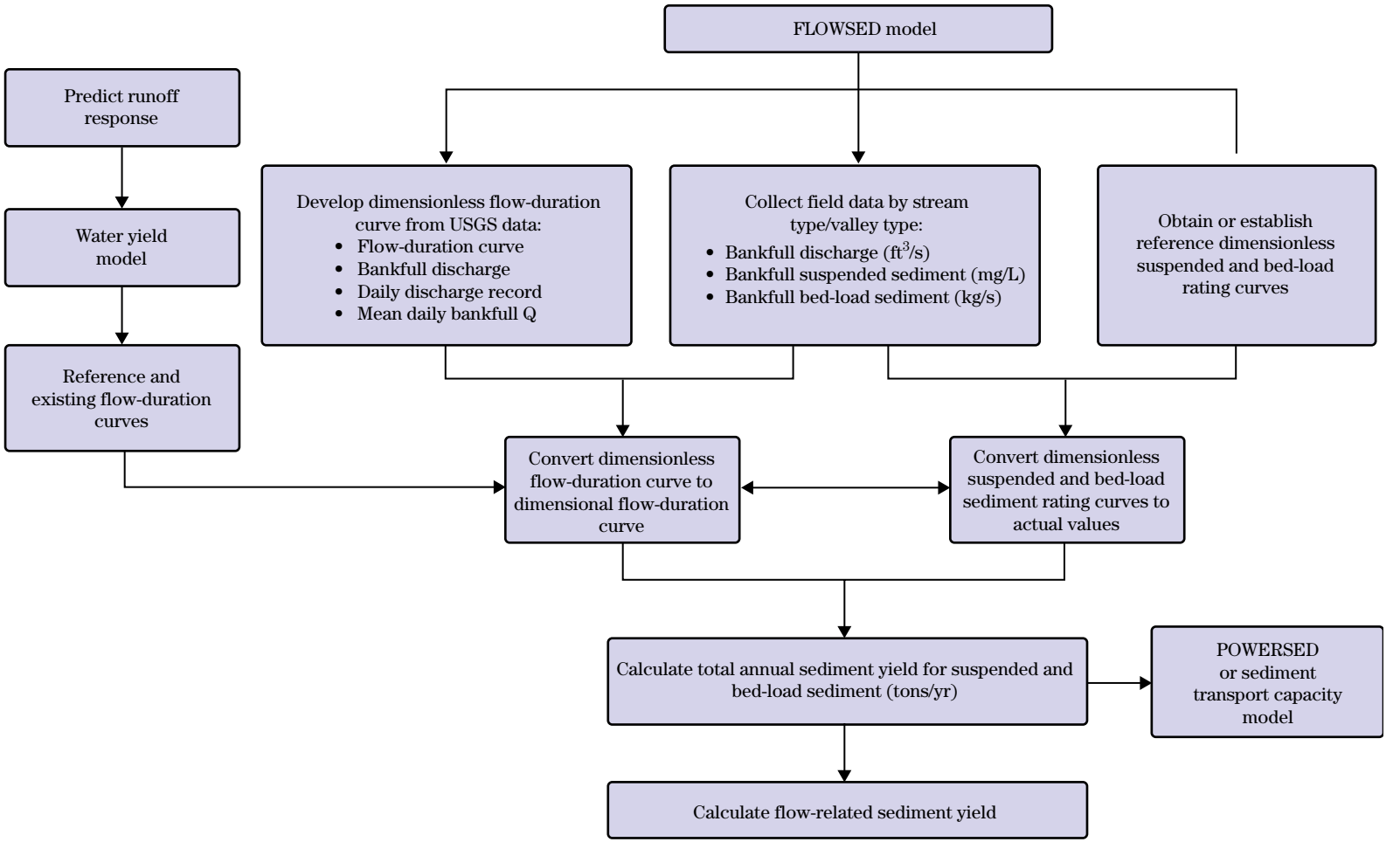


Figure 11–20 Graphical depiction of the FLOWSED model

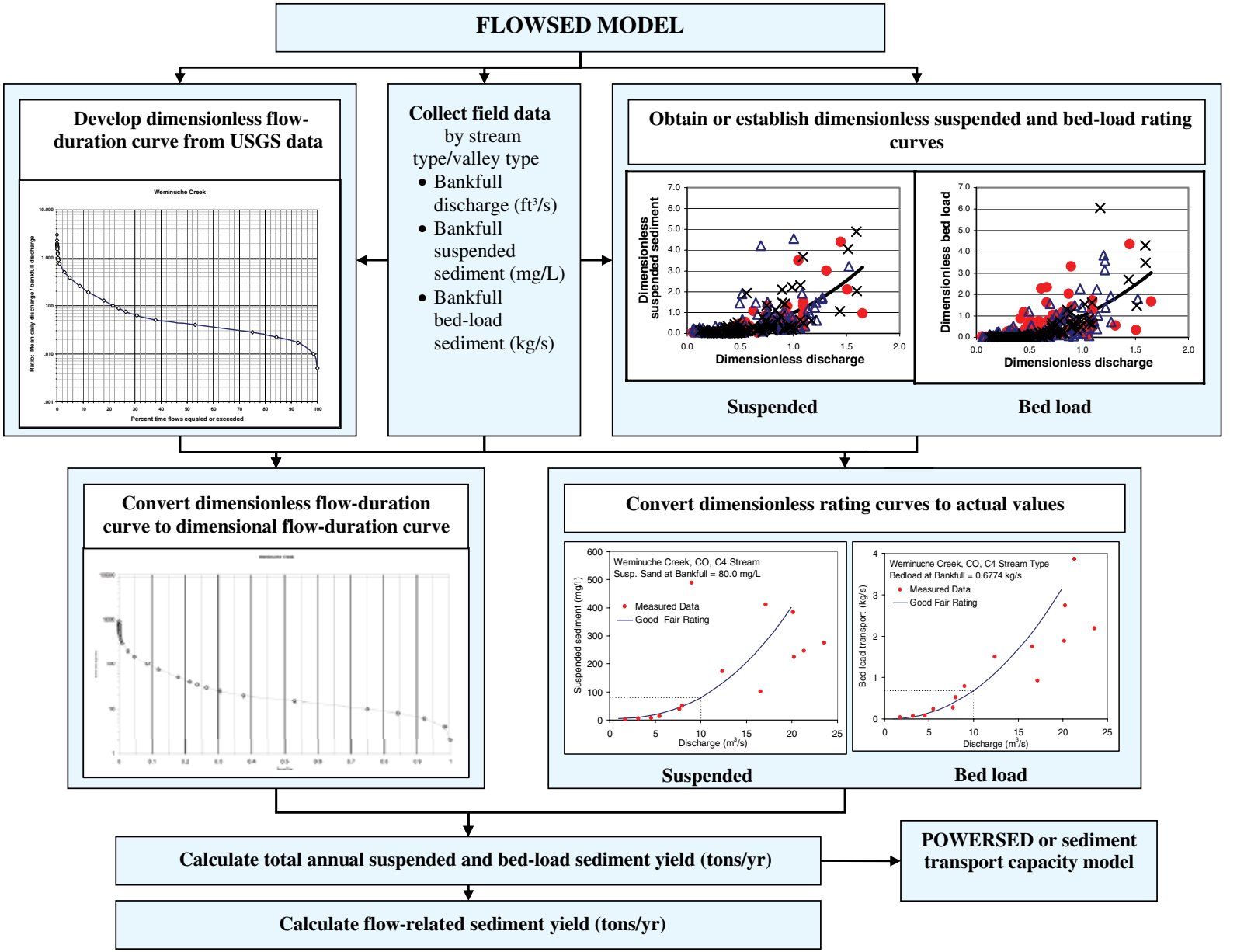


Table 11–10 FLOWSED model procedure to calculate annual bed-load and suspended sediment yield

FLOWSED procedure	
FS–1	Measure stream cross section (on riffle), profile, pattern, and materials.
FS–2	Measure bankfull width, mean depth, and velocity, and compute discharge.
FS–3	Measure suspended sediment at the bankfull stage; separate wash load in lab
FS–4	Measure bed-load sediment at the bankfull stage, sieve particle sizes, and measure largest size.
FS–5	Compute average water surface slope.
FS–6	Collect point bar sample, weigh by size fraction and record D_{50} and largest size (D_{max}).
FS–7	Collect pebble count on active riffle bed: obtain D_{50}, D_{84} sizes (mm).
FS–8	Determine stream type.
FS–9	Conduct channel stability assessment procedure, including Pfankuch channel stability rating.
FS–10	Obtain reference dimensionless bed-load sediment rating curve for appropriate stream type/stability rating.
FS–11	Obtain reference dimensionless suspended sediment rating curve for appropriate stream type/stability rating.
FS–12	Determine ratio of wash load/suspended sediment by Q/Q_{bkf} relation.
FS–13	Construct a bed-load rating curve (enter range of Q/Q_{bkf} ratios into the reference bed-load relation from step 10 and multiply by the measured bankfull bed load from step 4).
FS–14	Construct suspended sediment rating curve in the same manner as in step 13 using reference dimensionless sediment relations (step 11) and bankfull suspended sediment (step 3).
FS–15	Construct a suspended sediment rating curve less wash load (silt/clay) for potential settleable sediment by multiplying ratio of wash load/suspended sediment for appropriate Q/Q_{bkf}.
FS–16	Convert suspended sediment less wash load from mg/L to tons/day on rating curve: tons/d = $0.0027 \times ft^3/s \times mg/L$.
FS–17	Convert suspended sediment less wash load from mg/L to tons/d as in step 16.
FS–18	Convert bed load in lb/s to tons/d, where tons/d = (lb×86,400)/2000 (if metric, convert kg/s to lb/s by multiplying by 2.205).
FS–19	Obtain dimensionless flow-duration curve from either water yield model or regionalized relation.
FS–20	Develop the dimensionless flow-duration curves using the normalization parameter of mean daily bankfull discharge, rather than momentary maximum values from flood-frequency data. Divide the mean daily discharge (the day bankfull discharge occurs) by the momentary maximum value to determine the appropriate conversion ratio.
FS–21	Convert dimensionless flow-duration curve to actual flow by multiplying bankfull discharge (step 2) times the Q/Q_{bkf} ratios from dimensionless flow-duration curve (step 19).
FS–22	Calculate total annual sediment yield for suspended sediment, suspended sediment less wash load, and bed load from sediment rating curve/flow-duration curve procedure (table 11–11). Obtain flow from the water yield model for hydraulically recovered condition to compare departure from existing/proposed condition (step 22). This represents the pre-treatment flow duration/sediment relation.
FS–23	To determine flow-related increase in sediment, multiply post-treatment flow-duration curve times appropriate sediment rating curves for suspended, bed-load and total sediment rating curves to calculate total annual sediment yield using the same procedure as step 21 (table 11–11).

Table 11–11 FLOWSED calculation of total annual sediment yield

Stream: **Notes:**

From flow-duration curve						From sediment rating curves					Calculate	Calculate daily mean sediment yield				
Flow exceedance	Daily mean discharge	Mid-ordinate streamflow	Increment	Mid-ordinate streamflow	Dimensionless streamflow	Dimensionless suspended sediment discharge	Suspended sediment discharge	Suspended sediment minus wash load	Dimensionless bed-load discharge	Bed load	Time adjusted streamflow	Suspended sediment	Suspended sediment minus wash load	Bed load	Bed load plus suspended	Bed load plus suspended minus wash load
(%)	(ft³/s)	(%)	(%)	(ft³/s)	(Q/Q_{bkf})	(S/S_{bkf})	(tons/d)	(tons/d)	(b_s/b_{bkf})	(tons/d)	(ft³/s)	(tons/d)	(tons/d)	(tons/d)	(tons/d)	(tons/d)
									Annual totals:		(acre-ft)	tons/yr)	(tons/yr)	(tons/yr)	(tons/yr)	(tons/yr)

Bankfull discharge (ft³/s)	
Bankfull bed load (kg/s)	
Bankfull suspended (mg/L)	

Dimensionless sediment rating curve used

Type	Intercept	Coefficient	Exponent	X	Y	Form	Notes
Bed load							
Suspended							

Figure 11–21 Dimensionless flow-duration curve for Weminuche Creek, CO

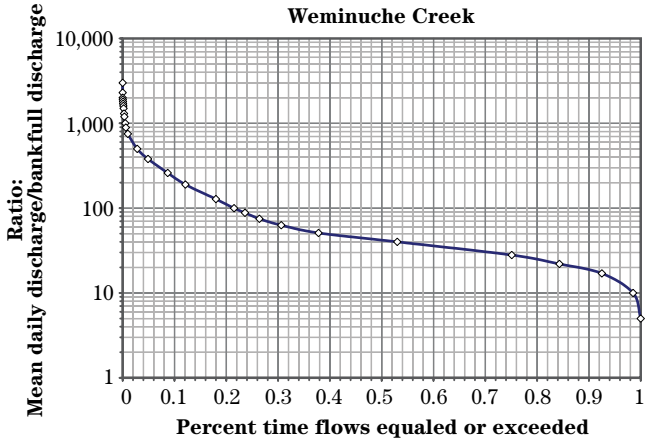


Figure 11–22 Bed-load sediment rating curve for Weminuche Creek, CO

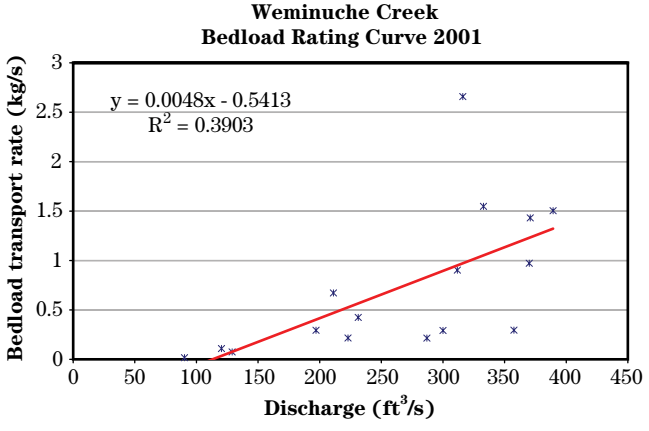


Figure 11–23 Suspended sediment rating curve for Weminuche Creek, CO

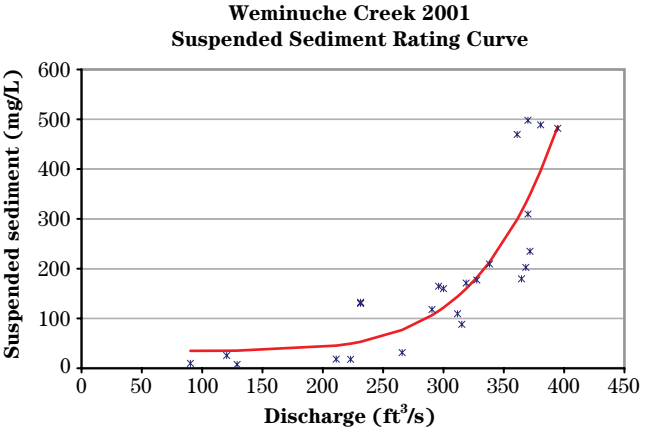
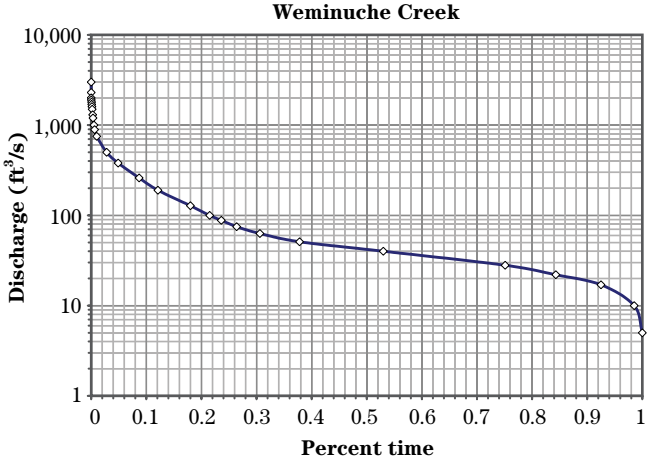


Figure 11–24 Dimensioned flow-duration curve for Weminuche Creek, CO



POWERSED

A generalized flowchart depicting the POWERSED model is shown in figure 11–25, and a graphical depiction of the model is shown in figure 11–26.

Evaluate channel characteristics that change hydraulic and morphological variables—Changes in the cross section and/or pattern (slope) for potentially impaired reaches are measured to determine width, depth, slope and calculated velocity. Comparisons are made between hydraulic characteristics of the reference versus the impaired reach. This analysis is used in the bed-load transport model (POWERSED) or in a comparable bed-load model selected by the user. Shear stress and unit stream power are calculated using equations 11–2 and 11–7:

$$\tau=\gamma dS \qquad \text{(eq. 11–2)}$$

where:
γ = specific weight of the fluid
d = mean depth
S = water surface slope

Unit stream power or power per unit of streambed area (ω_a) is defined as:

$$\omega_a=\tau u \qquad \text{(eq. 11–7)}$$

where:
τ = bankfull shear stress (lb/ft^2)
u = mean velocity

POWERSED can be used to simulate hydraulic geometry (width, depth, slope, velocity, and discharge) for a wide range of stages for reference and impaired reach hydraulic evaluations. POWERSED can also be used to compute changes in hydraulic character due to modified channel dimension, pattern, profile or materials. This information is used to determine changes in unit stream power for increased or decreased discharge. This model predicts channel stability response to imposed sediment load, change in flow, and/or change in distribution of energy due to channel change. The model determines sediment transport and predicts aggradation, stability, or degradation, depending on the nature and extent of the channel and/or flow change. The hydraulic/sediment departure is compared to the corresponding reference or stable condition. A recent comparison of predicted to observed values on an independent data set was shown in Rosgen (2006) where predicted annual sediment yield values were predicted within 3 percent of measured values for a C4 stream type and within 6 percent of measured values for a D4 stream type on Weminuche Creek, Colorado.

Calculate bed-load and suspended sand-bedmaterial load transport (stream power)—Bed load and suspended sand-bed material load transport calculations may use various equations, such as the Bagnold equation. The POWERSED model (figs. 11–25, 11–26 and tables 11–12 and 11–13) assists in the analysis of sediment transport and channel response. This model was developed to predict the effects of channel instability and sediment supply changes in sediment transport. Other bed-load and suspended sand-bed material load transport models can be employed by the user, based on familiarity with and calibration/validation of the model for application to the particular stream types being analyzed.

The POWERSED model applies the suspended sand-bed material and bed-load sediment rating curves/flow duration/revised unit stream power-transport curves or a comparable model selected by the user to predict sediment transport and channel stability. The prediction includes river stability and total annual bed-load sediment yield in tons/year. The equations or computer program generates a change in coarse bed-load transport that will be influenced by changes in channel cross section and/or slope. Changes in streamflow, velocity, unit stream power, critical dimensionless shear stress, and other variables due to land use changes predict changes

POWERSED—Continued

in river stability and total annual bed-load sediment yield. The sediment supply component is predicted using the FLOWSED model and is derived from dimensionless bed-load and suspended sediment rating curves for corresponding stream and stability types. These changes are compared to stable reference conditions for a departure comparison.

Procedural steps for computations of the POWERSED model are presented in table 11–12. Bed-load transport and suspended sand-bed material load is calculated using the POWERSED worksheet (table 11–13).

The POWERSED model is used to predict the transport rate and capacity for each reach independently. Reaches may be stable (sediment in versus sediment out), aggrading, or degrading. The model identifies reaches that may have serious instabilities due to changes in sediment supply and/or hydraulic characteristics. The analysis assists in pinpointing various river reaches for mitigation. The sediment transport changes reflect the sediment supply of the existing condition compared to the reference condition. Annual streambank erosion rates and other sources are compared to the total annual sediment yield.

Figure 11–25 POWERSED model to predict bed-load and suspended sand-bed-material load transport

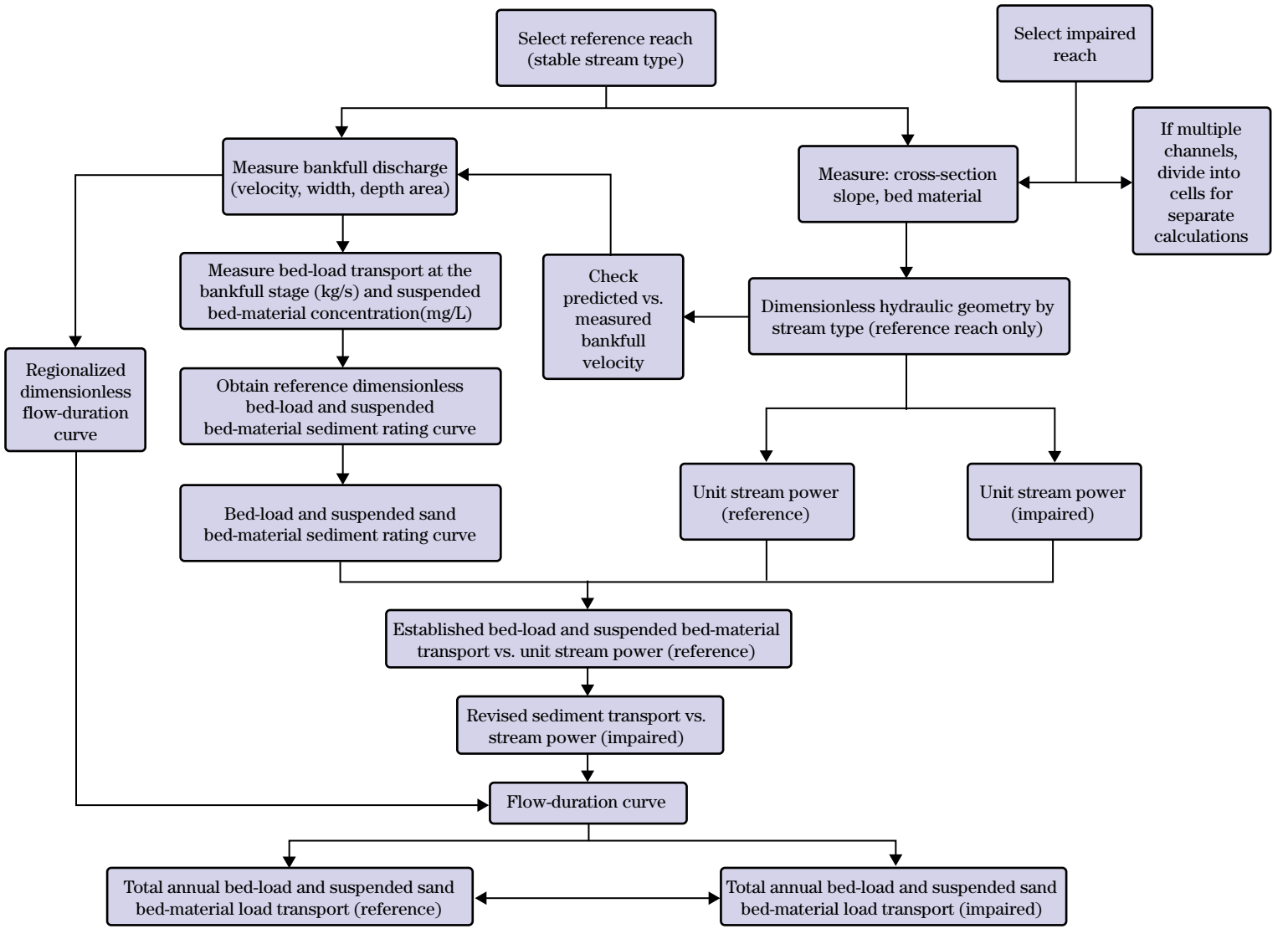


Figure 11–26 Graphical depiction of POWERSED model

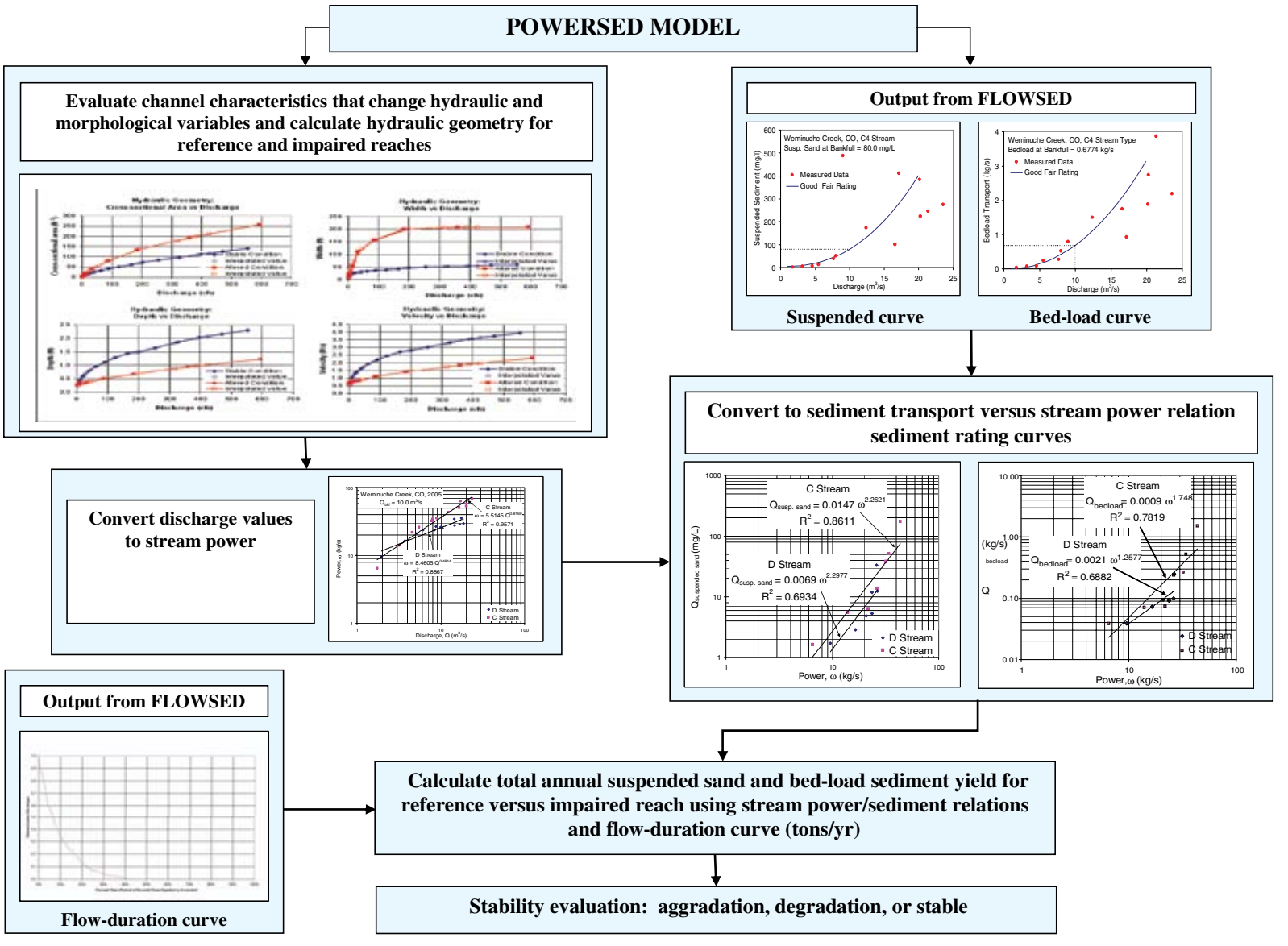


Table 11–12 POWERSED procedural steps of predicted bed-load and suspended sand-bed material transport changes due to alterations of channel dimension or slope (same stream with different bankfull discharges)

POWERSED procedure	
PS–1	Select a reference reach: a. Survey a stable cross section; measure the stream gradient and bed material. b. Measure bankfull discharge (ft^3/s). c. Measure bankfull bed load (kg/s).
PS–2	Obtain an appropriate dimensionless bed load and suspended sand sediment rating curve: a. Construct a dimensional bed load and suspended sand sediment rating curve for the defined range of flow using the measured bankfull discharge, bankfull bed load transport and suspended sand-bed material load.
PS–3	Obtain the drainage area of the reference reach: a. Predict bankfull discharge and cross-sectional dimensions using regional curves. b. Validate the regional curves using the measured bankfull discharge and cross-sectional dimensions.
PS–4	Use dimensionless hydraulic geometry by stream type to predict the hydraulic geometry of the stable cross section for a full range of discharge (baseflow to above bankfull): a. Construct hydraulic geometry curves. b. Check predicted versus measured bankfull velocity. c. Obtain hydraulic geometry for each discharge value within the defined range of flow. d. Calculate unit stream power for each discharge value within the defined range of flow.
PS–5	Select an impaired reach on the same stream: a. Obtain the drainage area. b. Predict bankfull discharge from the validated regional curve. c. Survey the cross section, and measure the stream gradient and bed material.
PS–6	Obtain the stable (potential) dimension, pattern, and profile for the impaired reach. If reference reach is not immediately upstream and/or is of different size or drainage area, complete the following procedure: a. Slope = valley slope/sinuosity. b. Obtain appropriate cross-sectional area from regional curve. c. Obtain width-to-depth ratio (W/d) from reference dimensionless ratios by stream type. d. Calculate appropriate width.
PS–7	Use the RIVERMorph® procedure or applicable spreadsheet calculations to predict the hydraulic geometry of the impaired and potential cross sections for a full range of discharge (baseflow to above bankfull). Follow the step below for the impaired and potential cross sections: a. Construct hydraulic geometry curves. b. Obtain hydraulic geometry for each discharge value within the defined range of flow. * If channel has multiple channels, divide the channels into thirds and treat as a separate channel c. Calculate unit stream power for each discharge value within the defined range of flow.
PS–8	Plot unit stream power vs. bed load and suspended sand-bed material transport for the stable cross section.
PS–9	Construct a unit stream power versus bed-load transport curve for the impaired and potential cross sections using the relationship constructed in step 8.
PS–10	Obtain a dimensionless flow-duration curve for the appropriate region: a. Create a dimensional flow-duration curve using the bankfull discharge for the stable reach. b. Create a dimensional flow-duration curve using the bankfull discharge for the impaired reach.

Table 11–12 POWERSED procedural steps of predicted bed-load and suspended sand-bed material transport changes due to alterations of channel dimension or slope (same stream with different bankfull discharges)—Continued

POWERSED procedure	
PS–11	Calculate total annual sediment yield (bed-load and suspended sand-bed-material load) in tons/yr for all three (stable, impaired, potential) cross sections using the appropriate flow-duration curve: a. Convert the predicted bed-load transport for each discharge value within the defined range of flow from kg/s to tons/d by multiplying kg/s by 95.24. Convert values of suspended sand-bed material load in mg/L to tons/d by multiplying (mg/L)(.0027)(ft^3/s). b. Multiply the predicted bed-load and suspended sand-bed material load transport (tons/d) by the percent time factor from flow-duration curve. c. Sum the time adjusted bed-load transport and multiply by 365 days to obtain annual bed load yield in tons/yr. d. Divide the annual yield for both bed-load and suspended sand-bed material load by the drainage area to obtain the annual unit area bed-load and suspended sand-bed material load yield (tons/yr/mi^2). e. Compare the annual unit area bed-load and suspended sand-bed material load yield predicted for all three conditions (stable, impaired and potential).
PS–12	Record data for impacted and reference condition (separately) in POWERSED worksheet (table 11–13).

Table 11–13 POWERSED model to predict bed-load and suspended sand and bed-material load transport*

Stream: **Gage station#:** **Date:**

Equation type	B0	B1	B2	Form	Equation name	Enter equation number (1 or 2)	Bankfull discharge (ft^3/s)	Bankfull bed load (lb/s)	Suspended bed-material load (mg/L)
1 Dimensional									
2 Dimensionless									
3 Bed load									
4 Suspended sand-bed concentration									

Flow-duration curve		Calculate	Hydraulic geometry				Measure	Calculate								
Exceedance probability	Daily mean discharge	Mid-ordinate streamflow	Area	Width	Depth	Velocity	Slope	Shear stress	Stream power	Unit power	Time increment	Daily mean bed-load transport	Time adjusted bed-load transport	Daily mean suspended transport	Time adjusted suspended transport	Time adjusted total transport
(%)	(ft^3/s)	(ft^3/s)	(ft^2)	(ft)	(ft)	(ft/s)	(ft/ft)	(lb/ft^2)	(lb/s)	(lb/ft/s)	(%)	(tons/d)	(tons)	(tons/d)	(tons)	(tons)
											Annual total sediment yield (tons/yr):					

*Use this model for both reference and impaired conditions separately. Calculate bed load separately from suspended bed-material load.

Step 28 Obtain maximum bankfull riffle depth (d_{max}) from ratio of maximum riffle depth divided by mean bankfull depth from dimensionless ratios of reference reach data (step 7) (table 11–3).

$$d_{mbkf} = \left[\left(\frac{d_{mbkf}}{d_{bkf}}\right)_{ref}\right] d_{bkf} \qquad \text{(eq. 11–18)}$$

Step 29 Determine entrenchment ratio of proposed channel by measuring the width of the flood-prone area at an elevation of twice the maximum bankfull depth ($d_{max\ bkf}$). Entrenchment ratio is calculated by:

$$ER = \frac{W_{fpa}}{W_{bkf}} \qquad \text{(eq. 11–19)}$$

Step 30 Calculate flood-prone area capacity. This involves estimating velocity associated with the cross-sectional area and slope of the stream channel and flood-prone area. Determine cross-sectional area of the flood-prone area. Plot the bankfull cross-section and flood-prone area elevation ($2 \times d_{max\ bkf}$) and width. Use valley slope for hydraulic calculations for the flood-prone area. Estimate roughness from Manning's equation based on vegetative cover and other roughness elements. HEC–2, HEC–RAS, or other models can be used to obtain the corresponding discharge of the flood-prone area. Calculate the 50- and 100-year flood levels based on the proposed design. Use the bankfull channel capacity from step 22.

Step 31 Calculate depth of pool (ratios from table 11–3):

$$d_{mbkfp} = \left[\left(\frac{d_{mbkfp}}{d_{bkf}}\right)_{ref}\right] d_{bkf} \qquad \text{(eq. 11–20)}$$

Step 32 Calculate depth of glide (ratios from table 11–3):

$$d_g = \left[\left(\frac{d_g}{d_{bkf}}\right)_{ref}\right] (d_{bkf}) \qquad \text{(eq. 11–21)}$$

Step 33 Calculate depth of run (ratios from table 11–3):

$$d_{run} = \left[\left(\frac{d_{run}}{d_{bkf}}\right)_{ref}\right] (d_{bkf}) \qquad \text{(eq. 11–22)}$$

Step 34 Calculate slope of pool (ratios from table 11–3):

$$S_p = \left[\left(\frac{S_p}{S}\right)_{ref}\right] S \qquad \text{(eq. 11–23)}$$

Step 35 Calculate slope of glide (ratios from table 11–3):

$$S_g = \left[\left(\frac{S_g}{S}\right)_{ref}\right] S \qquad \text{(eq. 11–24)}$$

Step 36 Calculate slope of run (ratios from table 11–3):

$$S_{run} = \left[\left(\frac{S_{run}}{S}\right)_{ref}\right] S \qquad \text{(eq. 11–25)}$$

Step 37 Calculate pool-pool spacing (from plan view and profile layout).

Step 38 Design stabilization/fish habitat enhancement measures (grade control, energy dissipation, bank stability, holding cover). See phase VI.

Step 39 Prepare revegetation plan compatible with native plants, soil, and site conditions. Make recommendations on vegetative maintenance and management for long-term solutions.

Step 40 Design a monitoring plan including effectiveness, validation, and implementation monitoring. Prepare maintenance plan to ensure long-term success.

The variables associated with existing, proposed, gage station, and reference reach data are summarized in the form as demonstrated in table 11–14 (Rosgen 1998). The variables used in table 11–14 and forms used in field data collection are in the Reference Reach Field Book (Rosgen, Leopold, and Silvey 1998; Rosgen and Silvey 2005).

Table 11–14 Morphological characteristics of the existing and proposed channel with gage station and reference reach data

Restoration site (name of stream and location):
Reference reach (name of stream and location):

Variables		Existing channel	Proposed reach	USGS station	Reference reach
1	Stream type				
2	Drainage area, mi^2				
3	Mean riffle depth, ft (d_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
4	Riffle width, ft (W_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
5	Width-to-depth ratio (W_{bkf}/d_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
6	Riffle cross-sectional area, ft^2 (A_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
7	Max riffle depth (d_{mbkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
8	Max riffle depth/mean riffle depth (d_{mbkf}/d_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
9	Mean pool depth, ft (d_{bkfp})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
10	Mean pool depth/mean riffle depth	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
11	Pool width, ft (W_{bkfp})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
12	Pool width/riffle width	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
13	Pool cross-sectional area, ft^2 (A_{bkfp})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
14	Pool area/riffle area	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
15	Max pool depth (d_{mbkfp})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
16	Max pool depth/mean riffle depth (d_{mbkfp}/d_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:

Table 11–14 Morphological characteristics of the existing and proposed channel with gage station and reference reach data—Continued

Variables		Existing channel	Proposed reach	USGS station	Reference reach
17	Low bank height (LBH)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
18	Low bank height to max riffle depth (LBH/d_{mbkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
19	Width of flood-prone area, ft (W_{fpa})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
20	Entrenchment ratio (W_{fpa}/W_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
21	Point bar slope	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
22	Bankfull mean velocity, ft/s (u_{bkf})				
23	Bankfull discharge, ft^3/s (Q_{bkf})				
24	Meander length, ft (Lm)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
25	Meander length ratio (Lm/W_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
26	Radius of curvature, ft (Rc)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
27	Ratio of radius of curvature to bankfull width (Rc/W_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
28	Belt width, ft (W_{blt})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
29	Meander width ratio ($W_{b/t}/W_{bkf}$)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
30	Individual pool length, ft	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
31	Pool length/riffle width	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
32	Pool to pool spacing (based on pattern), ft (p-p)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:

Table 11–14 Morphological characteristics of the existing and proposed channel with gage station and reference reach data—Continued

Variables		Existing channel	Proposed reach	USGS station	Reference reach
33	Ratio of p-p spacing to bankfull width (p-p/W_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
34	Stream length (SL)				
35	Valley length (VL)				
36	Valley slope (VS)				
37	Average water surface slope (S)		S = VS/k		
38	Sinuosity (k)	SL/VL:	SL/VL:	SL/VL:	SL/VL:
		VS/S:		VS/S:	VS/S:
39	Riffle slope (water surface facet slope) (S_{rif})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
40	Ratio riffle slope to average water surface slope (S_{rif}/S)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
41	Run slope (water surface facet slope) (S_{run})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
42	Ratio run slope/average water surface slope (S_{run}/S)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
43	Pool slope (water surface facet slope) (S_p)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
44	Ratio of pool slope/average water surface slope (S_p/S)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
45	Glide slope (water surface facet slope) (S_g)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
46	Ratio glide slope/average water surface slope (S_g/S)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
47	Max run depth, ft (d_{run})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
48	Ratio max run depth/ bankfull mean depth (d_{run}/d_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:
49	Max glide depth, ft (d_g)	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:

Table 11–14 Morphological characteristics of the existing and proposed channel with gage station and reference reach data—Continued

Variables		Existing channel	Proposed reach	USGS station	Reference reach
50	Ratio max glide depth/ bankfull mean depth (d_g/d_{bkf})	Mean:	Mean:	Mean:	Mean:
		Range:	Range:	Range:	Range:

Materials					
51	Particle size distribution of channel material (active bed)				
	D_{16} (mm)				
	D_{35} (mm)				
	D_{50} (mm)				
	D_{84} (mm)				
	D_{95} (mm)				
52	Particle size distribution of bar material				
	D_{16} (mm)				
	D_{35} (mm)				
	D_{50} (mm)				
	D_{84} (mm)				
	D_{95} (mm)				
	Largest size particle at the toe (lower third) of bar (mm)				

Sediment transport validation (Based on Bankfull Shear Stress)		Existing	Proposed
Calculated shear stress value (lb/ft^2) from curve			
Size from Shields diagram - Original data (mm)			
Size from Shields diagram - Colorado data (mm)			
Largest size (mm) to be moved (D_{max})			
Dimensionless shear stress (τ^*)			
Mean d_{bkf} (ft) calculated using dimensionless shear stress equations for given slope			

Remarks:

(f) Phase VI—Selection and design of stabilization and enhancement structures/methodologies

The objectives of river structures are often primarily designed to:

- buy time to protect the new channel from excess erosion until significant riparian vegetation can become established
- reduce accelerated streambank erosion
- provide grade control
- provide recreational boating
- obtain stable flow diversions
- enhance fish habitat including instream cover, holding cover, spawning habitat, and habitat diversity
- reintroduce and stabilize large wood for fishery, stability, and aesthetic purposes
- protect infrastructure adjacent to streams
- protect bridges, culverts, and drainageway crossings
- reduce flood levels
- transport sediment
- provide energy dissipation

River stabilization and enhancement structures are numerous and continue to be improved and developed. The effort here will not be to make a complete listing, but rather present methods used in the Rosgen geomorphic channel design methodology consistent with the objectives. The structures and methods primarily utilize native materials such as natural boulders, logs, rootwads, and vegetative transplants.

Design objectives will be presented to provide the user with alternatives to standard or traditional structures.

Grade control

Often cross-channel check dams are used for grade control. NRCS has successfully used many types of channel grade control structures, but streams with high sediment loads have experienced some adverse channel adjustment in some case. The adjustments are associated with aggradation, lateral erosion, flood stage increase, migration barriers for fish, increased recreational boating risk, land loss, channel incision through lateral migration and channel avulsion. To prevent these stability problems, the cross vane was developed (fig.11–27 (Rosgen 2001e)).

Application of this design is also very effective for bridge pier scour reduction (Johnson, Hey, et al. 2002). A photograph depicting the structure as constructed on the lower Blanco River, Colorado, is shown in figure 11–28. The structure also decreases near-bank shear stress, minimizing streambank erosion.

The photographs in figures 11–29 and 11–30 demonstrate the use of cross vanes in river restoration. In this example, a reconstructed river project on the East Fork Piedra River, Colorado, in a valley type V (glacial trough), converted a braided (D4) stream type to a meandering (C4) stream type. The use of the cross vane structure was effective at maintaining grade control, transporting excessive coarse bed load, reducing bank erosion, buying time for riparian vegetation colonization, and providing trout habitat. The structures located along 3 miles of this project withstood floods at twice the bankfull discharge magnitude in 2004. Logs and rootwads can also be utilized in this structure as designed in Rosgen (2001e) and as shown in figure 11–31. The use of large wood in this structure assists in the visual, as well as biological enhancement objectives. The step in the upper third of the structure dissipates energy, reduces footer scour, and minimizes risk for recreational boating and fish passage.

A structure designed for larger rivers for grade control and streambank protection is the W-weir. This structure can also be effectively used for irrigation diversions, protection of central piers and approach sections on bridges, bed-load transport, recreational boating, and fish habitat. Visually, it is improved over a line of rock often used in grade control. It resembles natural bedrock features in stream channels. Figure 11–32 depicts the design (Rosgen 2001e), and figure 11–33 shows a typical W-weir structure as installed on the Uncompahgre River in Colorado.

Streambank stabilization

Most stream restoration projects require some degree of streambank stabilization. Often the stabilization involves riparian vegetation reestablishment or change in management. Regardless, there is a time element that is needed to establish rooting depth, density, and

Figure 11–27 Cross section, profile, and plan view of a cross vane

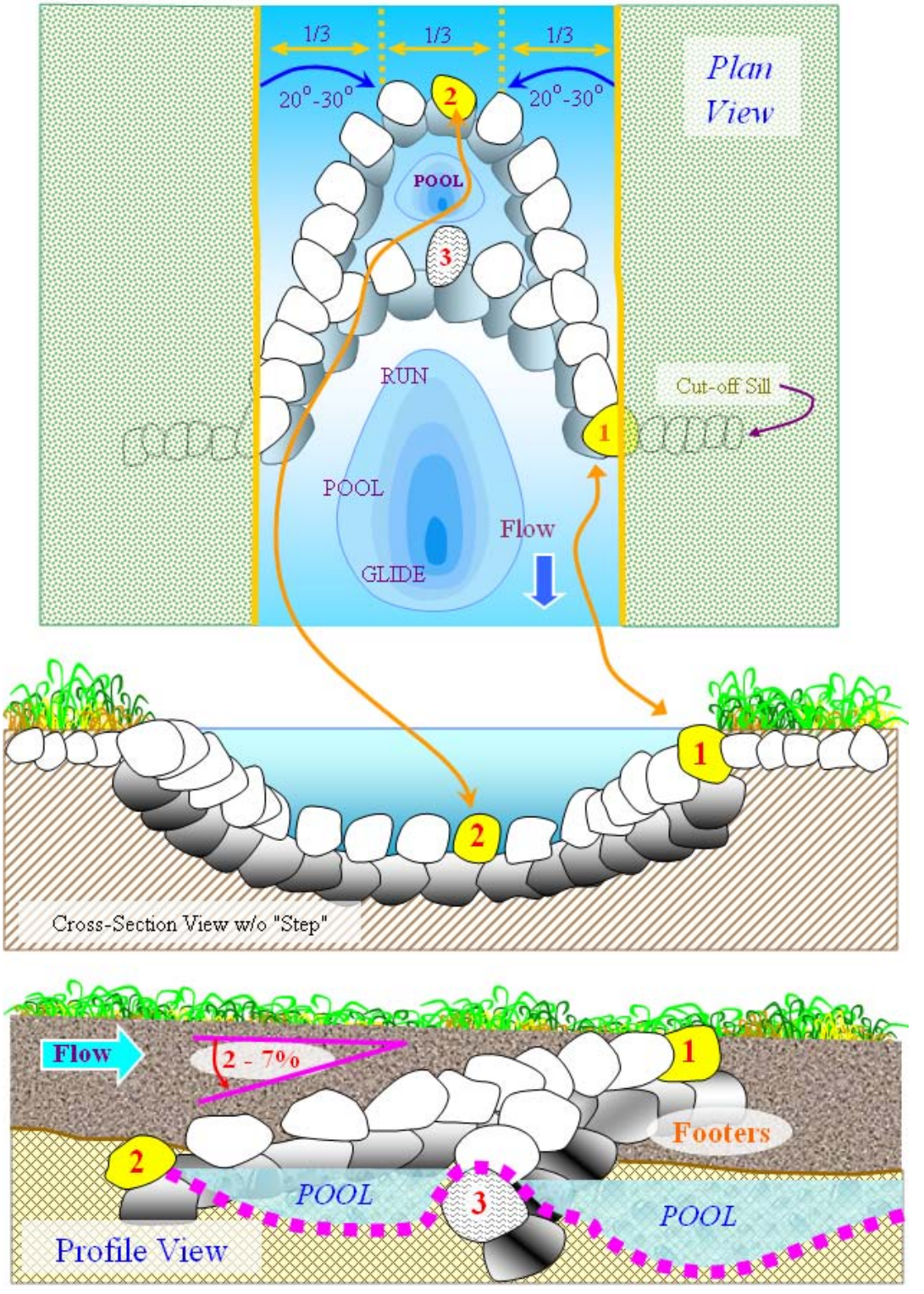


Figure 11–28 Cross vane installed on the lower Blanco River, CO

Figure 11–29 Cross vane structure with step on the East Fork Piedra River, CO

Figure 11–30 Cross vane/step-pool on the East Fork Piedra River, CO

Figure 11–31 Cross vane/rootwad/log vane step-pool, converting a braided D4→C4 stream type on the East Fork Piedra River, CO

Figure 11–32 Plan, cross section, and profile views of a W-weir structure

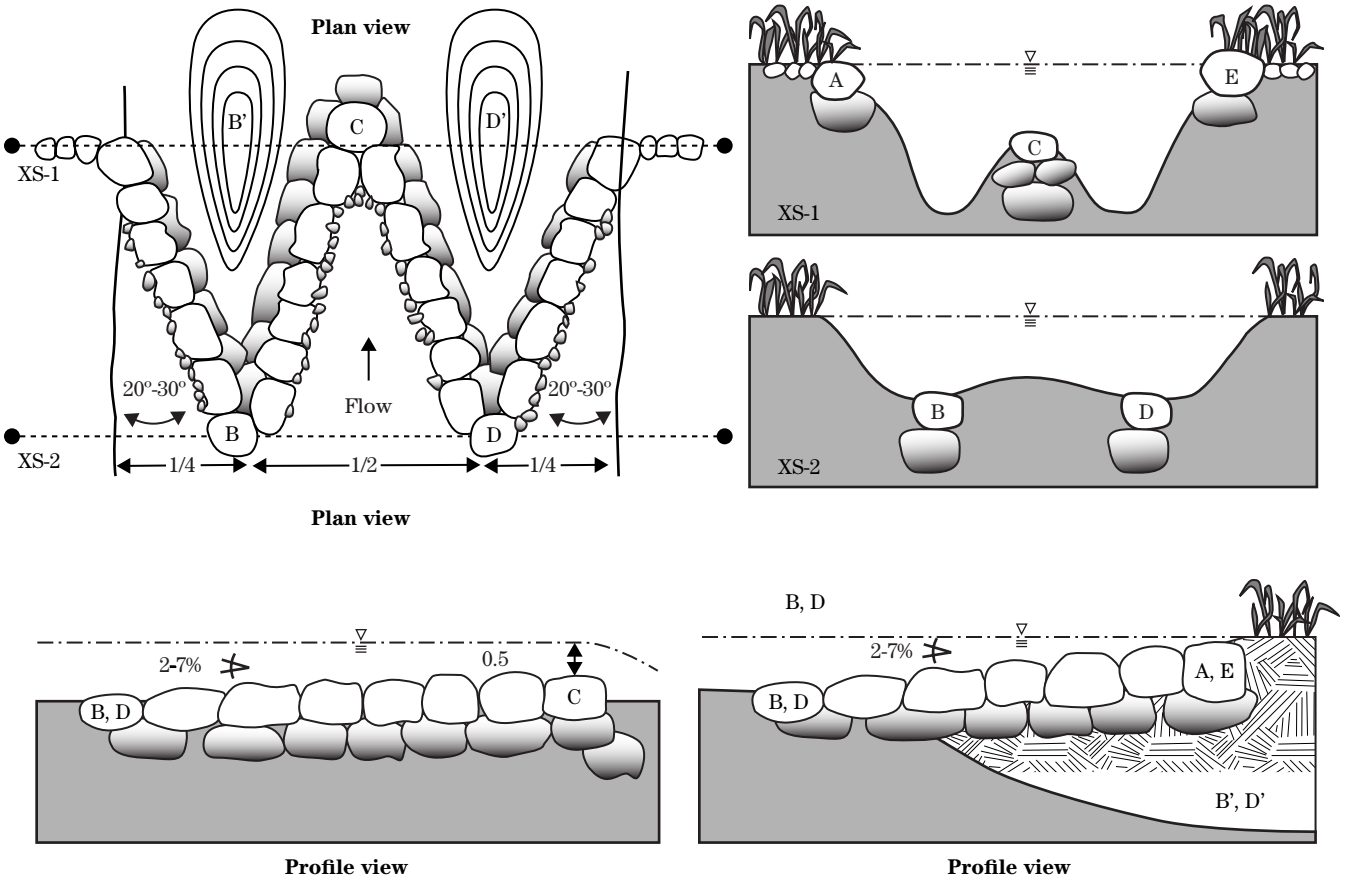


Figure 11–33 W-weir installed on the Uncompahgre River, CO

Figure 11–34 Plan, profile, and section views of the J-hook vane structure

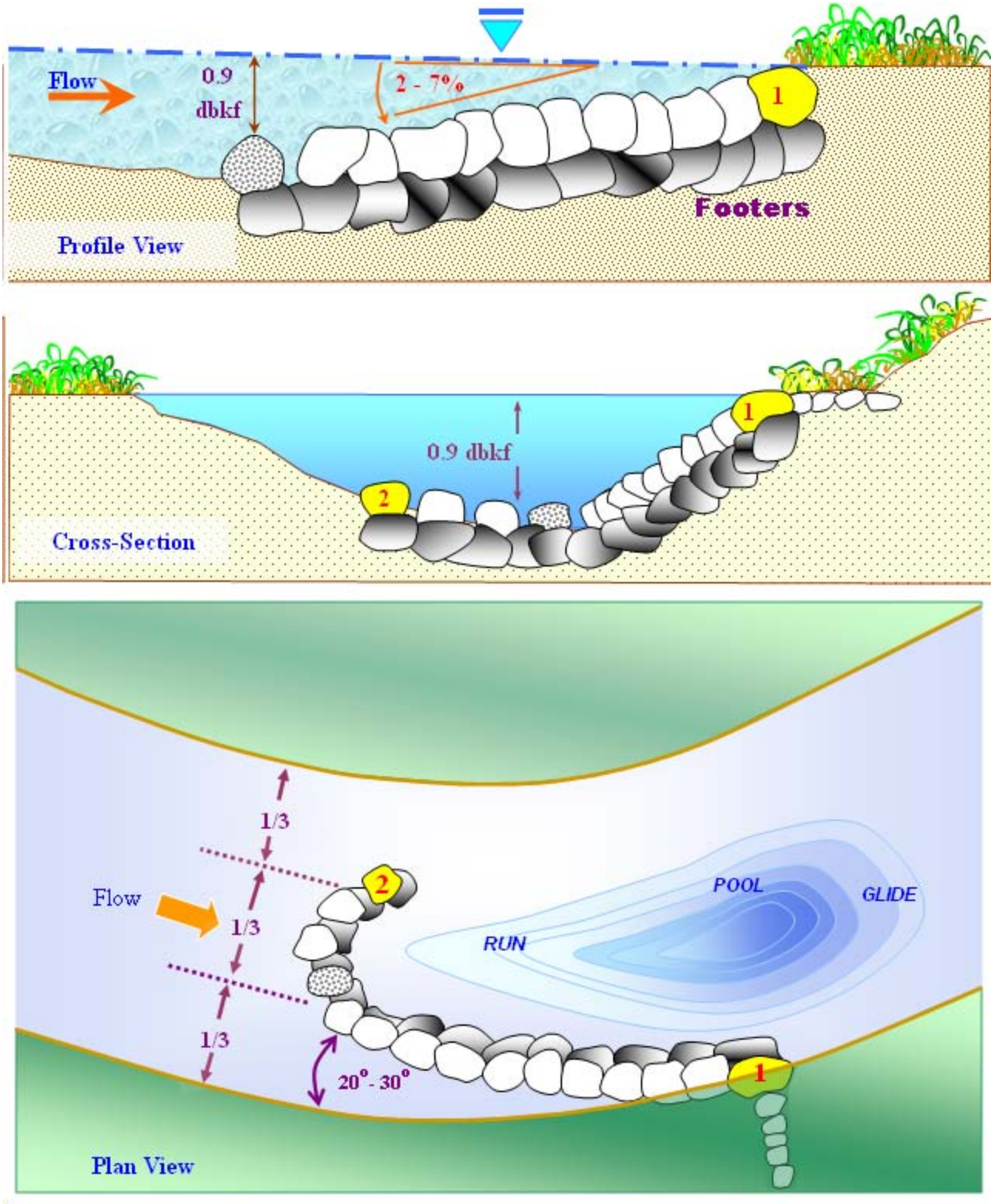


Figure 11–35 Log vane/J-hook combo with rootwad structure

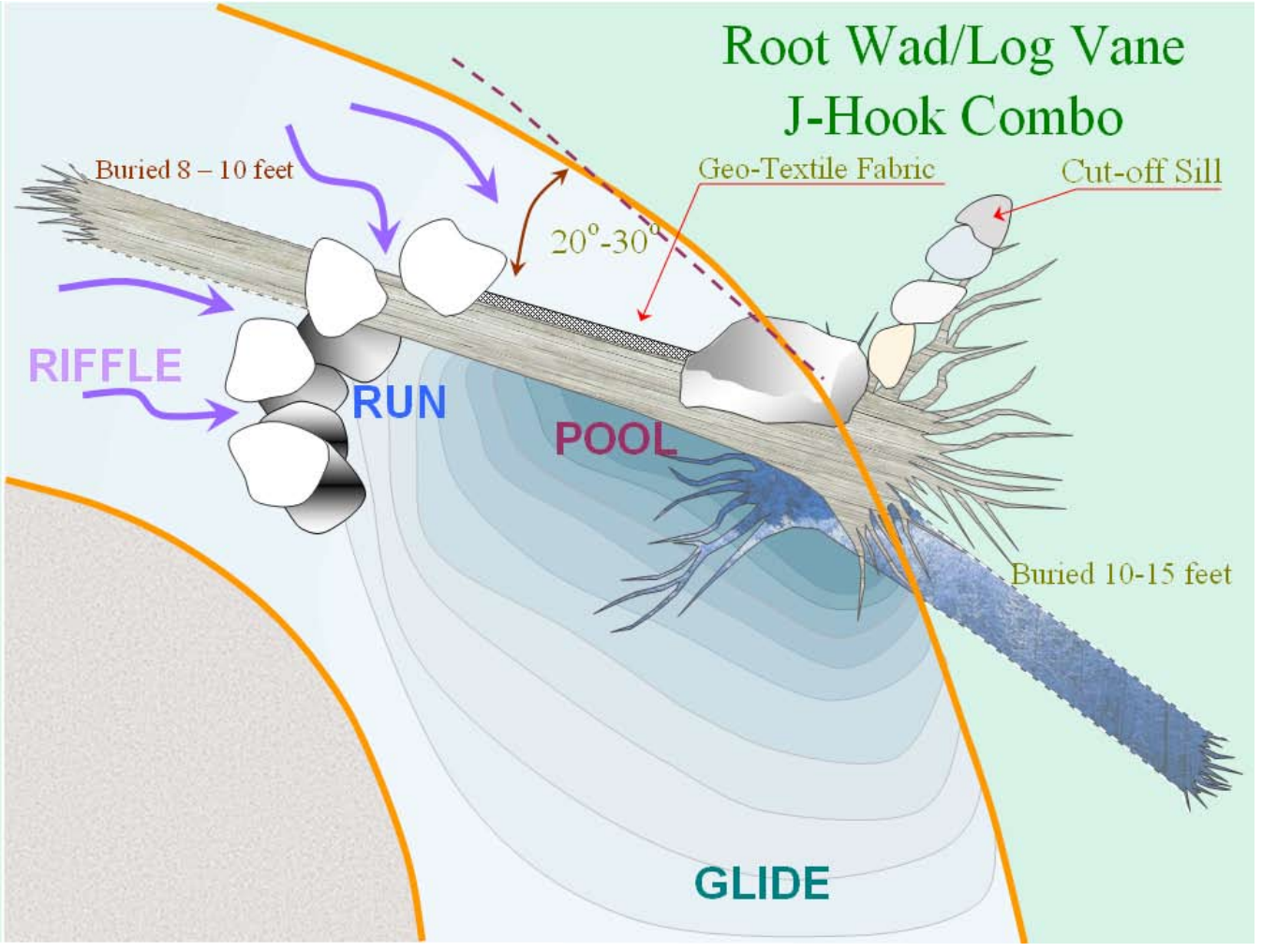


Figure 11–36 Rock vane/J-hook combo with rootwad and log vane footer

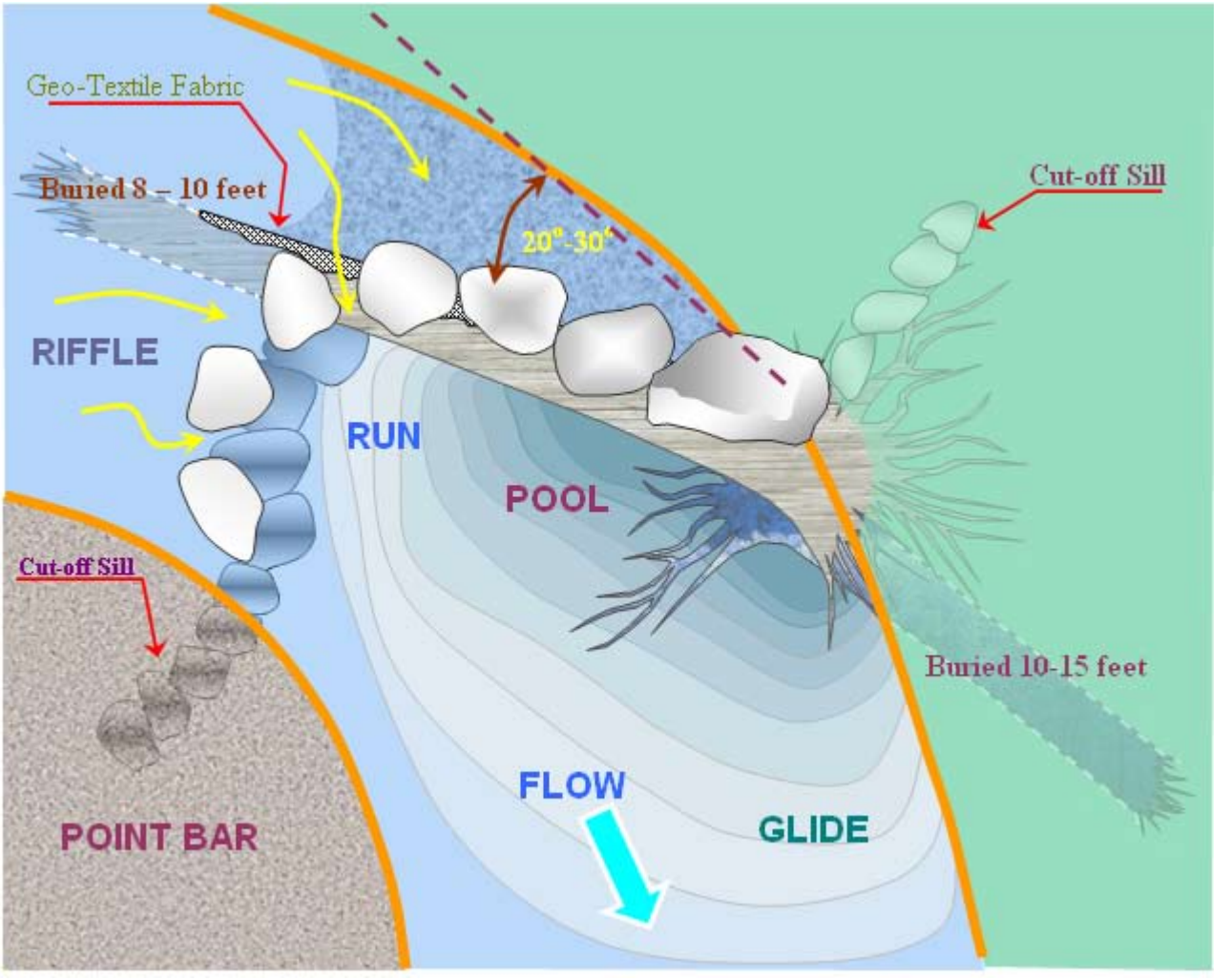
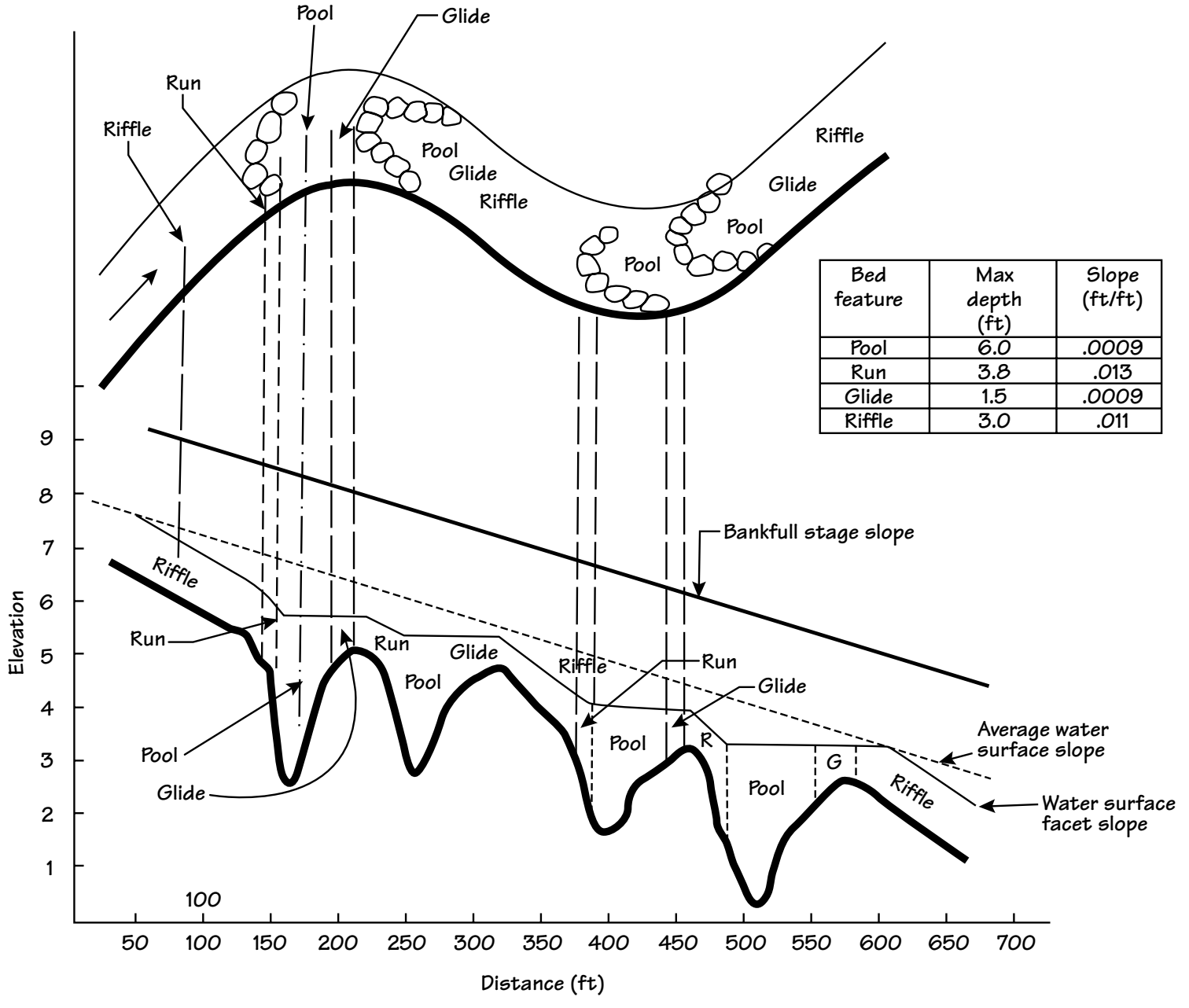


Figure 11–37 Native boulder J-hook with cut-off sill, East Fork Piedra River, CO

Figure 11–38 Rootwad/log vane/J-hook structure, East Fork Piedra River, CO

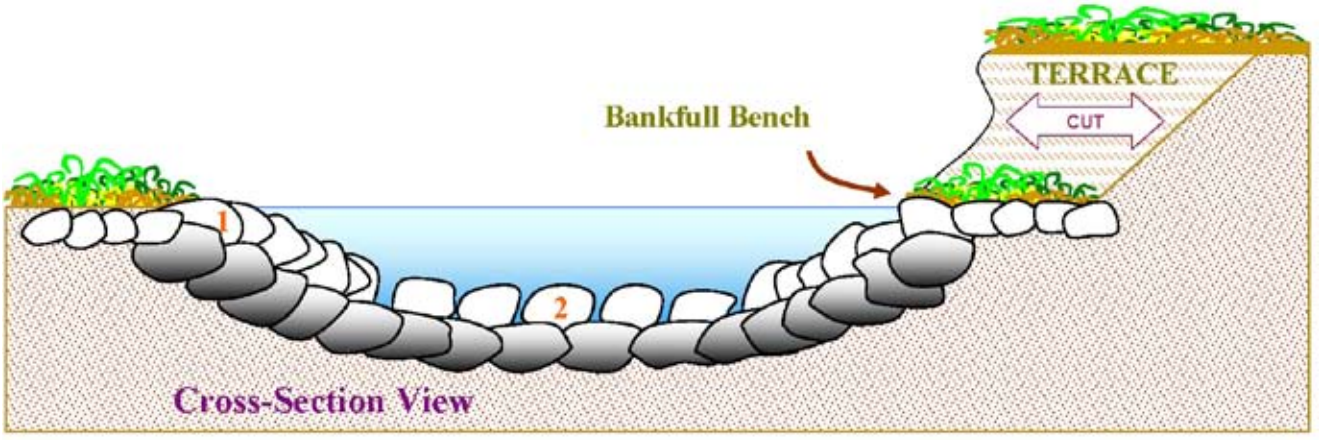
Figure 11–39 J-hook/log vane/log step with cut-off sill, East Fork Piedra River, CO

Figure 11–40 Longitudinal profile of proposed C4 stream type showing bed features in relation to structure location



Bed feature	Max depth (ft)	Slope (ft/ft)
Pool	6.0	.0009
Run	3.8	.013
Glide	1.5	.0009
Riffle	3.0	.011

Figure 11–41 Boulder cross vane and constructed bankfull bench



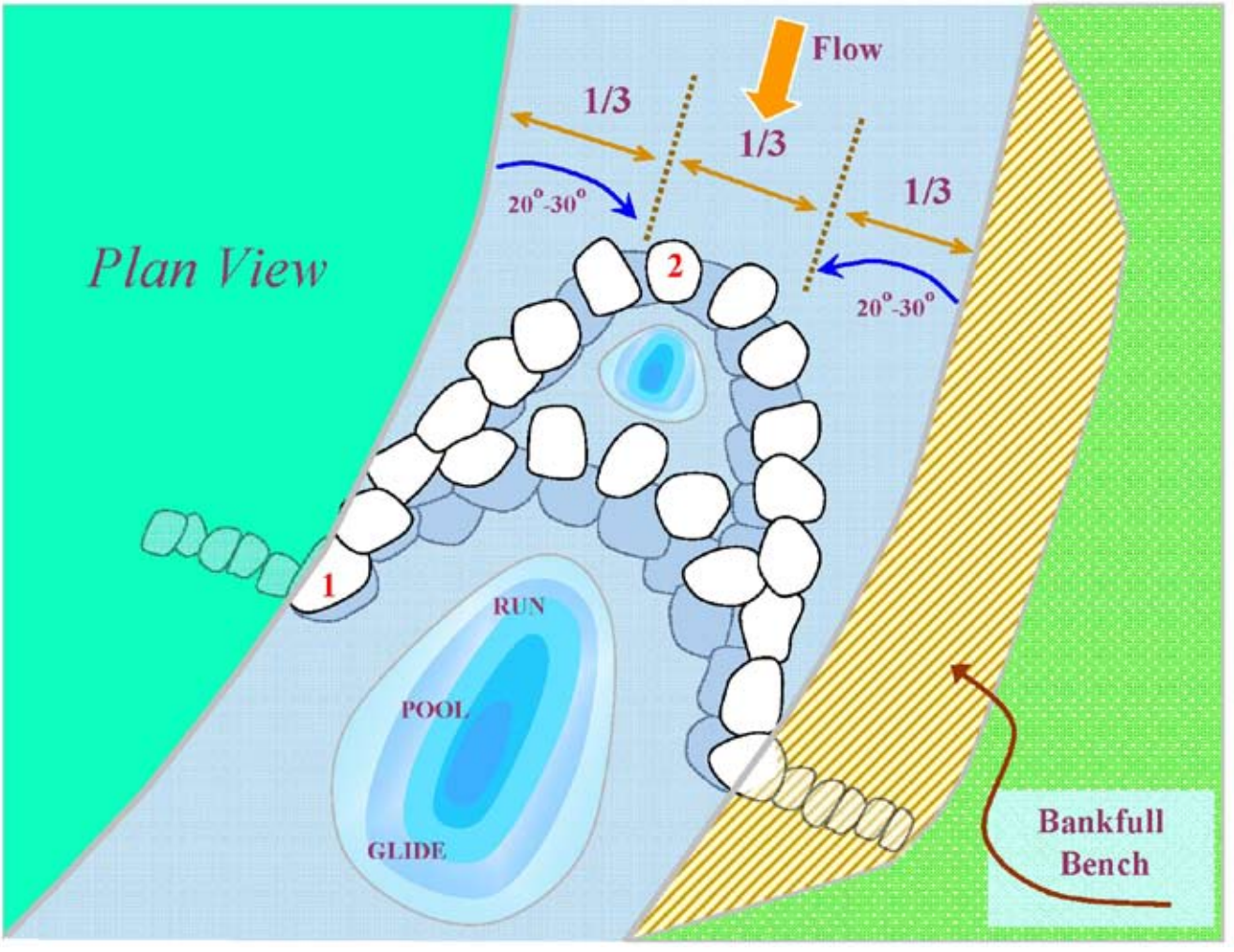
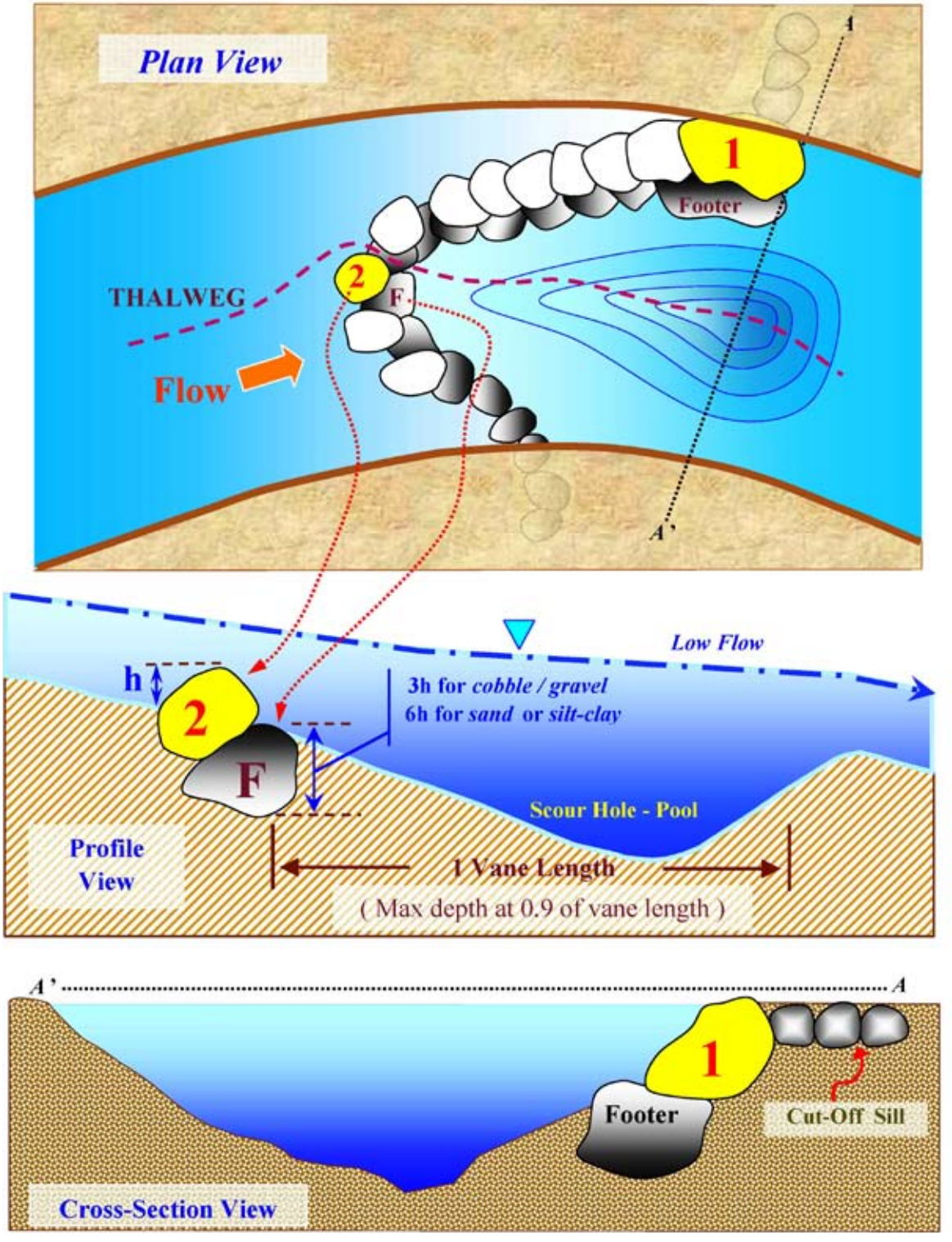


Figure 11–42 Locations/positions of rocks and footers in relation to channel shape and depths



strength to help maintain bank stability. The use of the J–hook (or fish hook) vane was developed to reduce near-bank stress to buy time for root development. The design is shown in figure 11–34 (Rosgen 2001e). Materials other than boulder are used in the J–hook vane. Logs and rootwads can be effectively used for multiple objectives (fig. 11–35 (Rosgen 2001e)). Variations in the use of materials are shown in figure 11–36 (Rosgen 2001e). An example of a J–hook vane is shown in figure 11–37, as constructed out of native boulders located in a reconstructed East Fork Piedra River. The structure also provides fish habitat, energy dissipation, bed-load transport, and provides protection of developments along streambanks. The use of a J–hook vane reduces the need for toe rock stabilization or a surfacing or hardening of the bank with riprap or other resistant structure. The length of bank protected is approximately two and a half to three times the length of the vane. The J–hook vane also is used to protect bridges and structures (Johnson, Hey, et al. 2001). Figures 11–38 and 11–39 provide examples of a J–hook vane using logs, rootwads, and log steps, as well as native boulders.

An example of the use of structure location forming compound pools consistent with meander curvature and bed features is shown in figure 11–40. The accompanying data indicate the slope and depth of the corresponding bed features. Regardless of structures, riparian vegetation establishment and management must be an active part of Rosgen geomorphic channel design.

Vane design specifications

The use of structures must be compatible with curvature and bed features of natural rivers. Figures 11–41 and 11–42 illustrate the use of rock for cross vanes, as well as for footers. Figure 11–43 provides guidance on rock sizing.

Vane slope—The slope of the vane extending from the bankfull stage bank should vary between 2 to 7 percent. Vane slope is defined by the ratio of bank height/vane length. For installation in meander bends, ratios of J-hook vane length/bankfull width is calculated as a function of the ratio of radius of curvature/bankfull width and departure angle (table 11–15). Equations for predicting ratios of J-hook vane spacing/bankfull width on meander bends based on ratio of radius of curvature/bankfull width and departure angle are shown in table 11–16. Vane length is the distance measured from the bankfull bank to the intercept with

Figure 11–43 Rock size

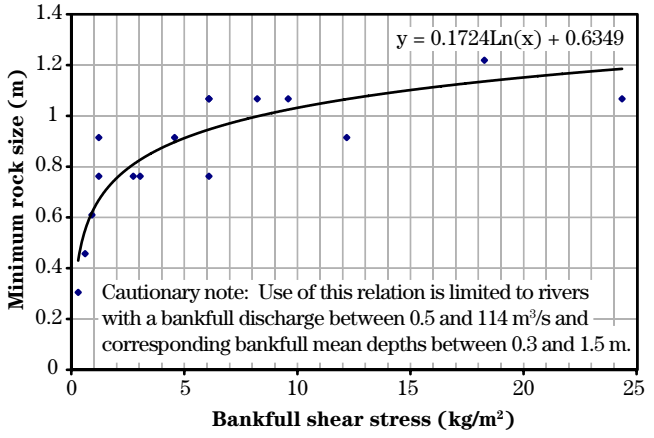


Table 11–15 Equations for predicting ratio of vane length/bankfull width (V_L) as a function of ratio of radius of curvature/width and departure angle, where W = bankfull width (SI units)

Rc/W	Departure angle (degrees)	Equation
3	20	$V_L = 0.0057\ W+0.9462$
3	30	$V_L = 0.0089\ W+0.5933$
5	20	$V_L = 0.0057\ W+1.0462$
5	30	$V_L = 0.0057\ W+0.8462$

Table 11–16 Equations for predicting ratio of vane spacing/width (V_s) as a function of ratio of radius of curvature/width and departure angle, where W = bankfull width (SI units)

Rc/W	Departure angle (degrees)	Equation
3	20	$V_s = -0.006\ W+2.4781$
3	30	$V_s = -0.0114\ W+1.9077$
5	20	$V_s = -0.0057\ W+2.5538$
5	30	$V_s = -0.0089\ W+2.2067$

the invert elevation of the streambed at a third of the bankfull channel width for either cross vanes or J-hook vanes. For very large rivers, where it is impractical to extend the vane length to a third of the bankfull width, vane slope is calculated based on the specified angle of departure and the ratio of bank height/vane length where the vane arm intercepts the proposed invert of the structure.

The spacing of J-hook vanes can be increased by 0.40W for a low BEHI of less than 30 (Rosgen 1996, 2001b).

Bank height—The structure should only extend to the bankfull stage elevation. If the bank is higher, a bankfull bench is constructed adjacent to the higher bank, and the structure is integrated into the bench. The use of a cross vane is shown in figure 11–41 where a bankfull bench is created adjacent to a terrace bank.

Footers—The minimum footer depth at the invert for cobble and gravel-bed streams is associated with a ratio of three times the protrusion height of the invert rock. This is applicable to all three structures and is shown in figure 11–41 for a J-hook vane. For sand-bed streams, the minimum depth is doubled due to the deeper scour depths that occur. All rocks for all three structures require footers. If spaces are left between the invert rocks for cross vane and W-weirs, the top of the footer rocks becomes the invert elevation for grade control. If no gaps are left, the top of the surface rock becomes the base level of the stream.

Rock size—The relationship of bankfull shear stress to minimum rock size used for all three structures is shown in figure 11–43. The application of this empirical relation is limited to size of rivers whose bankfull discharge varies from 0.56 cubic meters per second (20 ft^3/s) to 113.3 cubic meters per second (4,000

ft^3/s). For example, appropriate minimum rock sizes for values of bankfull shear stress less than 1.7 kilograms per square meter (0.35 lb/ft^2) are associated only with stream channel bankfull depths from 0.26 to 1.5 meters (2–5 ft). This relation would not be appropriate for applications outside the limits of the data for a river slope of 0.0003 and a mean depth of 6.1 meters, even though a similar shear stress results, as in the example presented.

(g) Phase VII—Design implementation

A key requirement at this phase is to correctly implement the proposed design. This involves the layout, construction supervision, and water quality controls during construction.

Layout

It is necessary to pre-stake the alignment of the channel and to provide for protection of existing vegetation outside of the construction alignment. The layout involves making necessary onsite adjustments to the design based on constraints that may have been previously overlooked. Terrain irregularities, vegetation, property boundaries, and channel changes since the field data were collected can all require local modifications to placement. Staging areas for materials must be located for both the collection and temporary storage of materials. Stockpile areas, vegetative donor sites, and boundary references/facilities requiring special identification must be flagged. Locations of structure placement and type must be flagged.

Construction supervision (oversight)

Without exception, it is critical to have daily onsite inspection and construction coordination. It is essential to check grades, dimensions, structure placement, slopes, angles, and footers as an on-going requirement. It is most effective to coordinate this work during construction, rather than wait and provide a postconstruction inspection and find problems after the work is completed. The daily field review and documentation at this phase is very helpful to properly implement the design.

Water quality controls

As part of the layout, sediment detention basins, diversions, silt fences, and pump sites must be located to prevent onsite and downstream sediment problems and as required by Federal, state, and local ordinances. Staging of construction should also be conducted in such a manner to minimize sedimentation problems. Monitoring of water quality during construction may be required; thus, preventative measures will reduce future potential problems.

(h) Phase VIII—Monitoring and maintenance

Monitoring

The key to a successful monitoring program is the focus on the question or the specific objectives of monitoring. Monitoring is generally recommended to:

- measure the response of a system from combined process interaction due to imposed change
- document or observe the response of a specific process and compare to predicted response for a prescribed treatment
- define short-term versus long-term changes
- document spatial variability of process and system response
- ease the anxiety of uncertainty of prediction
- provide confidence in specific management practice modifications or mitigation recommendations to offset adverse water resource impacts
- evaluate effectiveness of stabilization or restoration approaches
- reduce risk once predictions and/or practices are assessed
- build a data base to extrapolate for similar applications
- determine specific maintenance requirements

Watershed and river assessments leading to restoration involve complex process interactions, making accurate predictions somewhat precarious. Measured data reflecting specific processes will continually improve understanding and prediction of sedimentological, hydrological, morphological, and biological process relations. Another great benefit resulting from monitoring is the demonstration of the effectiveness of reduced sediment problems and improved river stability due to management/mitigation—the central purpose of watershed and sediment assessments and restoration.

The state of the science cannot be advanced, nor can the understanding of complex processes be improved without monitoring. This phase is divided into three major categories:

- implementation monitoring to ensure restoration designs were laid out and constructed correctly
- validation monitoring (matching predicted to observed response, including model calibration and model validation)
- effectiveness monitoring (response of a process or system to imposed change)

Field methods/procedures are also addressed.

Implementation monitoring—Often the best-laid design plans are not implemented correctly due to various reasons. Response of a process and/or system must first address the question or possible variable of potential problem in instituting the design and stabilization/enhancement structures correctly. Riparian vegetation response may be ineffective if heavy grazing of livestock occurred. Exclusion fence maintenance can also be a key in vegetative recovery. If restoration designs were correct, but the contractor installed structures at the wrong angle, slope, or position on the bank, then near-bank stress reduction or erosion rate would not be a correct design implementation related to the effectiveness of the mitigation structure.

As-built measurements of dimension, pattern, and profile are essential to compare to design plans. Documentation of exact locations and types of stabilization and/or enhancement structures is also required. Many failures observed in monitoring are due to poor structure placement locations, construction problems, as well as inability to implement correct design specifications.

Vegetation establishment problems are often traced to establishing the wrong plant associations (species), planting at the wrong time of year and at the wrong elevations on the bank (water table), using the wrong techniques in transplanting and/or cutting plantings, and lacking an irrigation plan, if needed. This monitoring leads the designer to be very thorough in the vegetative planning and implementation phase of restoration.

Validation monitoring—For every prediction methodology, there is a procedure to validate the model. Some methods are more difficult and time consuming to validate than others, while some results can be determined on a short-term, rather than a long-term basis.

The monitoring will improve predictive capability for the future and potentially reduce mitigation measures that would not be effective for continued implementation. Conversely, if management practices indicate that sediment and/or stability conditions create obvious impairment, revised practices or specific process-based mitigation such as restoration may be recommended. The restoration specialist will gain the most confidence in the procedure only by field measurements, which not only validate a prediction, but determine if the initial assessment objectives were met. The various categories of validation monitoring include calibration and validation.

- **Validation**—Model validation involves testing of a model with a data set representing local field data. This data set represents an independent source (different from the data used to develop the relation). Often these data are used to extend the range of conditions for which the model was developed. Due to the uncertainty of prediction, this step is very important prior to widespread application of model output. Models can be extremely helpful in comparative analysis, even if observed values depart from measured. It is important, however, to be aware of the variability in the prediction. Often this monitoring outcome develops tighter relations or subsets of the initial relation, improving the understanding of the processes being predicted. An example of this type of monitoring would be similar to the effectiveness monitoring of streambank erosion rates presented previously. However, beyond measuring bank erosion rate, the observer is additionally required to measure the same parameters used to predict streambank erosion. The streambank prediction involves calculating a bank erosion hazard index (BEHI) and near-bank stress (NBS) (Rosgen 1996, 2001b). The analysis involves plotting the observed values with the predicted values for the same prediction variables. In many cases (with sufficient numbers of observations), this monitoring can lead to improved local or regional models, adapted for unique soil types and vegetation. Validation modeling provides documentation not only on how well the mitigation performed but also on the performance of the model.

  Validation modeling is designed to answer specific questions at specific sites/reaches. Design must be matched with a strong understanding of the prediction model. Validation modeling for the dimensionless ratio sediment rating curves would involve sampling sediment over the full range of streamflows to compare predicted to observed values. The measurements would need to be stratified by the same stream type and stability rating used for the prediction.

- **Calibration**—Models are often used to predict potential impairment. Model calibration is the initial testing of a model and tuning it to a set of field data. Field data are necessary to guide the modeler in choosing the empirical coefficients used to predict the effect of management techniques. An example of this is the data set of measured suspended sediment and bed-load sediment by stream type and stability to establish dimensionless ratio sediment rating curves used for design. These data were not collected in all areas where the model would potentially be applied; thus, another type of monitoring (validation) is helpful to determine if the model is appropriate for extrapolation to a particular region.

Effectiveness monitoring—The specific restoration design and implementation needs to be monitored. Monitoring will determine the appropriateness or effectiveness of specific designs and is implemented to reduce potential adverse sediment and/or river stability effects. Since monitoring requires site-specific measurements, temporal, spatial, scale, streamflow variation, and site/reach, monitoring is required to properly represent such variability and extrapolate findings of a process and/or system response to imposed change. Such variability factors are summarized as:

- **Temporal**—To isolate the variability of season and/or annual change, designs of monitoring should include monitoring over time scales. For example, measuring annual lateral erosion rates should include measurements once per year at the same time of year. If the objectives are to identify seasons where disproportionate erosion occurs, measurements may be obtained during snowmelt runoff, later post stormflow runoff, ice-off, and other periods of time associated with a given erosional process. Annual replicate surveys of particle size gradation of bed material under a permanent glide cross section will provide valuable information of

magnitude, direction, and consequence of annual shifts. Temporal measurements must also cover a range of time during bed-load sampling as surges occur or slugs of bed load often appear as discontinuities of time. Sampling over recommended time periods for a given flow (generally 20 minutes) helps the probability of observing this variability (as opposed to an instantaneous point sample). Short-term versus long-term monitoring must also be considered based on the probability of change, the severity and consequence of effects, and the likelihood of variation. Sampling over many years, although costly, may be warranted to cover changes in wet/dry periods.

- **Spatial**—Variability of change/response involving spatial considerations can be identified by measurements of the same process at more than one site (cross section) or even more intense on the same site. For example, a longitudinal profile measured over a couple of meander wavelengths will indicate changes in the maximum depth and/or slope of pools, rather than just monitoring one pool at one location. Identifying more than one reach of the same morphological type can also be used to understand response trends. Sampling the spatial variability (both vertically and laterally) within a cross section of velocity and sediment helps identify or at least integrate such variability into a documented observation.
- **Scale**—Monitoring streams of various sizes and/or stream orders, but of the same morphological type and condition, will help identify variability in system response for proper extrapolation of results. For example, vertical stability measurements should be made on river reaches of the same condition and the same type, but at locations that reflect various stream widths (size) and stream order.
- **Streamflow variation**—Measurements of channel process relations need to be stratified over a range of seasonal and annual flows. For example, both suspended and bed-load sediment should be measured over a wide range of flows during the freshet, low-elevation snowmelt, high-elevation snowmelt, rising versus recession stages, stormflow runoff, and baseflow. This stratification for streamflow allows the field observer to plot a sediment rating curve that represents the widest range of seasonal flows where changes in sediment supply can vary.
- **Site or reach variation**—Monitoring a site for soil loss should include a soil type designation for potential extrapolation for similar conditions on similar soil types. The same is true for stream types. Sediment, hydraulic, and stability monitoring need to be stratified by stream type since such data will naturally vary for the reference (stable) reach between stream types. This information is helpful to be able to readily detect departure from a reference stream type, rather than differences between stream types.
- **Design concepts for effectiveness monitoring**—The key information summary from the assessments used to identify impairment and resultant restoration designs are as follows:
  - Summarize the causes of land use impacts responsible for the impairment.
  - Understand the processes affected.
  - Identify specific locations and reaches associated with adverse impacts.
  - Determine the time trends of impacts (potential recovery periods).
  - Identify the specific nature of impairment (direction, magnitude, and trend of change).
  - Evaluate the consequence of change.
  - Determine the nature, location, extent and quality of mitigation (implementation).

The information supplied in the following list leads the observer to identify the locations, nature of processes affected, the extent of the impact, and quality of the mitigation implementation. For example, if the dominant process impacted by a land use is causing disproportionate sediment supply, land loss and river instability, and is determined to be accelerated streambank erosion, then the lateral stability monitoring would emulate the following design:

- Locate reaches of the same stream type that represent an unstable bank.
- Locate reaches of the same stream type that represent a stable bank.

- Install permanent cross sections on each set of reaches.
- Install bank pins (if conditions warrant) and/or toe pins (see monitoring methods).
- Inventory vegetation, bank material, and slope for each site (see monitoring methods).
- Resurvey both streambanks at least once per year to measure soil loss (lateral erosion) and total volume (in cubic feet and tons/year).
- Compare annual lateral erosion rates over time to the stable reach and document rate of recovery based on the nature of the mitigation.

Vertical stability and enlargement rates and direction can also be monitored using permanent cross sections in a similar stratification procedure (comparison to reference reach, above versus below, before versus after).

Physical and biological monitoring—The sediment and river stability changes associated with assessment and design are primarily related to physical changes. However, the consequences of such physical changes are directly related to potential impairment of the biological function. Changes in river stability, such as aggradation, degradation, enlargement, and stream type changes, are also related to habitat and food chains. Limiting factor analyses assesses habitat loss due to river instability and/or excess sediment such as relations of holding cover, instream/overhead cover, water temperature, dissolved oxygen, and benthics. A range of information associated with stream condition can be stratified by stream type by stream stability including diversity index, population dynamics, age class distribution, spawning, rearing habitat, and many more attributes related to stream health. Biological monitoring should follow similar rules of inventory stratification based on the diverse nature of streams and their natural variability.

If a biologist is studying only the biological parameters within a specific ecoregion, the natural stable differences between reference reach stream types cannot be identified if the stratification of the inventory does not include stream types. In other words, a stable C4 stream type will not have the attributes of a stable E4 or B4 stream type, even though they are all gravel-bed streams. If the biological inventory is not stratified by stream type or stream stability, departure of habitat conditions between a stable C4 and an unstable C4 cannot be easily identified. Reference conditions that reflect biological potential must be stratified as a minimum by stream type and stream stability for adequate departure analysis to identify degree, direction, and magnitude of impairment. Companion biological inventories of assessment and monitoring can be very compatible with the monitoring methods of the physical system described.

Once this information is analyzed, the monitoring design can proceed. The next step is to identify a strategy of monitoring. Effectiveness monitoring should always be conducted near the activity responsible for the initial impairment. Four primary design strategies often utilized are as follows:

- Measurements obtained before versus after the initiation of a management change in the land use activity, mitigation, restoration, and enhancement. This can be very effective as it establishes a precalibration period that identifies premitigation variability of the measured parameters. Following mitigation, departure can be readily determined, assuming measurements take into consideration the aforementioned variability factors.
- Measurements or observations taken above versus below impact areas related to specific land uses and specific mitigation. For example, if two different grazing strategies are implemented, measurements of effectiveness can be observed above versus below fence line contrasts. This can also be implemented where a mitigation may only influence the lower reach of a river compared to the upper reach (assuming the same stream type).
- Measurements obtained determining departure from a paired watershed are often helpful as similar climatic events similarly impact both watersheds. The pairing would contrast a watershed that had extensive mitigation or land management change with one that had not been changed. This also assumes variability of scale, temporal, and spatial variability and comparisons of similar landscapes and stream types have been identified.

- Measurements obtained of a disturbed reach or site, receiving mitigation compared to a reference condition. This type of monitoring can occur at locations far removed from the reference reach. The reference condition, however, must be of the same soil type, stream type, valley type, lithology, and vegetative type.

Maintenance plan

To ensure that the implemented design is successful, it is key to have a maintenance plan. The maintenance plan must ensure the following:

- Survival of the riparian vegetation reestablishment—This could involve an irrigation supply or replanting/interplanting.
- Structure stability—Post-runoff inspections must be conducted of structures for grade control, bank stabilization and/or fish habitat enhancement. Maintenance needs are assessed and implemented to prevent future failures and to secure proper function.
- The dimension, pattern, and profile must stay within the natural variability or range as depicted in table 11–5 for each variable. Maintenance of these variables is recommended only if the values exceed the design channel ranges.
- The biological maintenance may involve reestablishment of described populations of various age classes and/or species of fish and/or food sources.

654.1103 Conclusion

The individual(s) responsible for the project should also become experienced by being involved in all phases of this methodology. If the same individual conducts the assessment and also completes the design, implementation, and monitoring, the desired objectives of restoration are the most likely to be accomplished. The complexity of this method requires great attention to detail, training, and an understanding of processes. The monitoring of the project, including the implementation, validation and effectiveness procedures, is the best approach to become experienced and knowledgeable about the Rosgen geomorphic channel design methodology.

Mathematical definitions

Variables	
Riffle cross-sectional area at bankfull	A_{bkf}
Pool cross-sectional area at bankfull	A_{bkfp}
Mean riffle depth at bankfull	d_{bkf}
Mean pool depth at bankfull	d_{bkfp}
Maximum glide depth at bankfull	d_g
Maximum riffle depth at bankfull	d_{mbkf}
Maximum pool depth at bankfull	d_{mbkfp}
Maximum run depth at bankfull	d_{run}
Diameter of riffle particle at 50% finer than size	D_{50}
Diameter of bar sample particle at 50% finer than size	$\hat{D}_{50}$
Diameter of riffle particle at 84% finer than size	D_{84}
Maximum size of particle on bar	D_{max}
Gravitational acceleration	g
Weight density of water	γ
Sinuosity	k
Low bank height	LBH
Meander length	Lm
Meander-length ratio	(Lm/W_{bkf})
Manning's *n*	*n*
Pool-to-pool spacing (based on pattern)	(p-p)
Bankfull discharge	Q_{bkf}
Hydraulic radius	R
Radius of curvature of meander	Rc
Average water surface slope (bankfull slope)	S
Slope of glide (water surface facet slope)	S_g
Stream length	SL
Slope of pool (water surface facet slope)	S_p
Slope of riffle (water surface facet slope)	S_{rif}
Slope of run (water surface facet slope)	S_{run}
Bankfull shear stress	τ
Dimensionless bankfull shear stress	τ^*
Bankfull mean velocity	u_{bkf}
Shear velocity	u^*

Variables	
Valley length	V_L
Valley slope	V_S
Riffle width at bankfull	W_{bkf}
Width-to-depth ratio at bankfull	(W_{bkf}/d_{bkf})
Width-to-depth ratio at bankfull of reference reach	$(W_{bkf}/d_{bkf})_{ref}$
Pool width at bankfull	W_{bkfp}
Belt width	W_{blt}
Meander-width ratio	(W_{blt}/W_{bkf})
Width of flood-prone area	W_{fpa}
Entrenchment ratio	(W_{fpa}/W_{bkf})
Stream power	ω

Subscripts	
Bankfull	bkf
Meander belt	blt
Flood-prone area	fpa
Glide	g
Maximum at bankfull	mbkf
Maximum at bankfull in pool	mbkfp
Pool	p
Reference reach	ref
Riffle	rif
Run	run

United States
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**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 12 Channel Alignment and Variability Design

Issued August 2007

Cover photo: Where alteration of a stream or channel is needed, planform may be modified to achieve the designed flow velocities by changing the channel length, as well as the gradient.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 12 Channel Alignment and Variability Design

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Chapter 12 Channel Alignment and Variability Design

654.1200 Purpose

Natural channel design includes establishment of a stable planform and often the incorporation of variability within the channel. The designer of a channel is also often asked to provide an assessment of natural bankline migration, as well. The purpose of this chapter is to provide systematic hydraulic design methodologies that can be used in the performance of these tasks. A wide variety of sources and techniques are available to the designer to make these assessments. This chapter provides overviews, descriptions, and examples illustrating some of the most common design techniques.

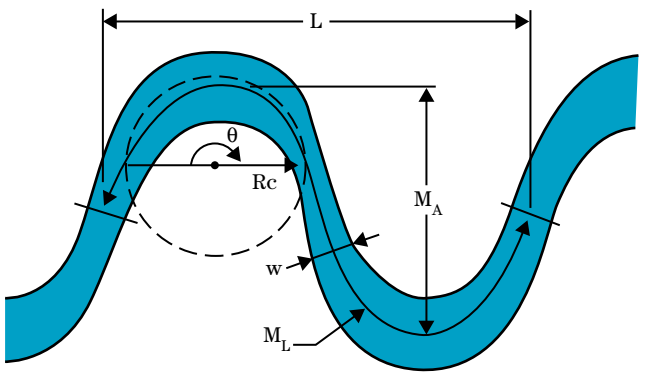
654.1201 Introduction

Natural channels are rarely perfectly linear and straight. While there are exceptions, and while boundary constraints may require a straight constructed channel, most natural channels exhibit at least some degree of sinuosity in their planform. Therefore, the assessment and design of a stable channel planform is an important part of any open channel design.

Planform design parameters include the meander wavelength, radius of curvature, sinuosity, and general alignment. Several of these variables are illustrated in figure 12–1 (Federal Interagency Stream Restoration Working Group (FISRWG) 1998). Several techniques that can be used to approximate natural channel alignments are presented in this chapter.

Natural channels rarely exhibit a uniform cross section. In fact, the variability often provides important ecological benefits. Since variability in a channel section is expected in natural channels, the designer of a natural channel restoration project is often asked to incorporate similar variability into the design. Generalized morphologic relationships for channel variability in streams and rivers are described in this chapter. Material from regionally specific studies is presented for illustration.

Figure 12–1 Variables used to describe channel alignment and planform



L Meander wavelength
M_L Meander arc length
w Average width at bankfull discharge
M_A Meander amplitude
Rc Radius of curvature
θ Arc angle

654.1202 Planform

This step in the design process involves laying out a planform after determining a meander wavelength and an appropriate channel length for one meander wavelength. Channel sinuosity is defined as the channel centerline length divided by the length of the valley centerline. It is determined from the calculated channel slope and valley slope. Analogy, hydraulic geometry, and analytical methods are employed to determine both the meander wavelength and a planform.

To apply the analogy method, a reference or control reach is located on either the study stream or another suitable stream. From this reach, a template for the meander planform is developed. This may be problematic due to the nonavailability of a suitable reference reach or subtle, but important fluvial, sedimentary, or morphological differences between a reference reach and the study reach.

Alternatively, meander wavelength can be determined using hydraulic geometry techniques. The most reliable hydraulic geometry relationship is wavelength versus width. As with the determination of channel width, preference is given to wavelength predictors from stable reaches of the existing stream either in the project reach or in reference reaches. The channel trace may also be determined analytically using the sine-generated curve. Finally, a string cut to the appropriate length can be laid on a map and fit to existing constraints and to the proper wavelength to form a meandering planform.

When uncertain about the appropriate technique, many practitioners use both analogy and hydraulic geometry and look for points of convergence in the recommendations. It is also important to note that planform flexibility may be limited by riparian features, infrastructure, land use, or other restrictions on the right-of-way. These factors may preclude the use of meanders with the amplitudes suggested from the described analogy or hydraulic geometry methods.

Braided channel systems are an important exception to much of the material presented in this chapter. Braided stream systems can exist naturally in estuarine, lacustrine, and glacial landscapes and valleys. These systems have depositional requirements and physical characteristics that are very different from single-thread channels. For braided streams, a single or dual thread channel reconstruction may be inappropriate and carry a potentially high risk of failure.

(a) Hydraulic geometry for meander wavelength

A composite relationship has been developed by Thorne and Soar (2001), combining 9 data sets and 438 sites. Their mean linear regression predictor for wavelength is:

$$\lambda = 10.23W \qquad \text{(eq. 12–1)}$$

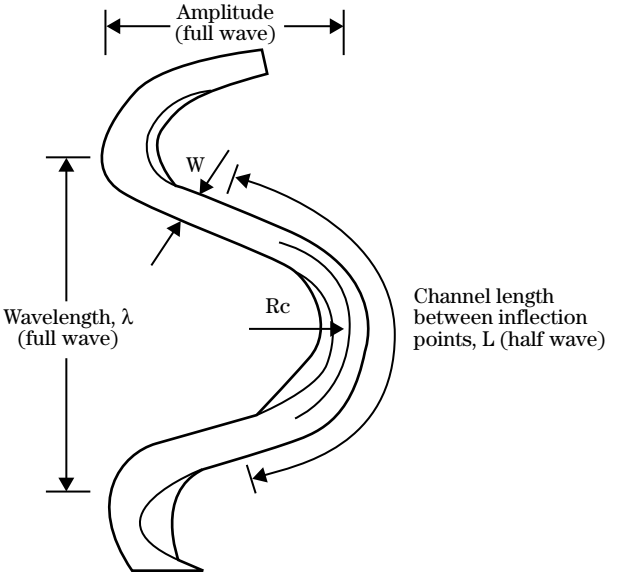
where:

λ = meander wavelength

W = channel width in any consistent units of measurement

Definitions of planform descriptive variables are shown in figure 12–2. Confidence bands about this equation are shown in figure 12–3. The r^2 for the

Figure 12–2 Planform descriptive variables



wavelength equation was 0.88 for a linear regression equation, with a variable exponent on W. This exponent was found not to be significantly different from 1.0, so the exponent was fixed at 1.0 for convenience. Only sites with sinuosities of at least 1.2 and bankfull widths between 1 meter and 1,000 meters were used in development of this regression equation. Within these constraints, meander wavelengths range between 10.4 meters and 19,368 meters, and sinuosity values range between 1.2 and 5.3. The equation, corrected for bias, is:

$$\lambda = 11.85W \qquad \text{(eq. 12–2)}$$

An unbiased hydrologic equation for meander wavelength suitable for engineering design, within 95 percent confidence limits on the mean response is:

$$\lambda = (11.26 \text{ to } 12.47)W \qquad \text{(eq. 12–3)}$$

According to Hey (1976) and Thorne (1997), twice the distance between successive riffles (or pools) in a straight channel should equal 4πW, or 12.57W. This is based on the assumption that the average size of the largest macroturbulent eddies (or helical flow cell) is half the channel width. Equation 12–3 shows that the upper range of stable meander wavelengths is numerically very close to this value and similar to the coefficient of 12.34 given by Richards (1982). This corroborates the assertion by Leopold and Wolman (1957, 1960) that the matching of waveforms in bed topography and planform is related to the mechanics of the flow and, in particular, to the turbulent flow structures responsible for shaping the forms and features of meandering channels.

Table 12–1 shows the data sources (438 sites) used in the development of these equations.

Figure 12–3 Hydraulic geometry relationship for meander wavelength with confidence intervals, λ = 10.23W, based on a composite data set of 438 sites in a variety of areas

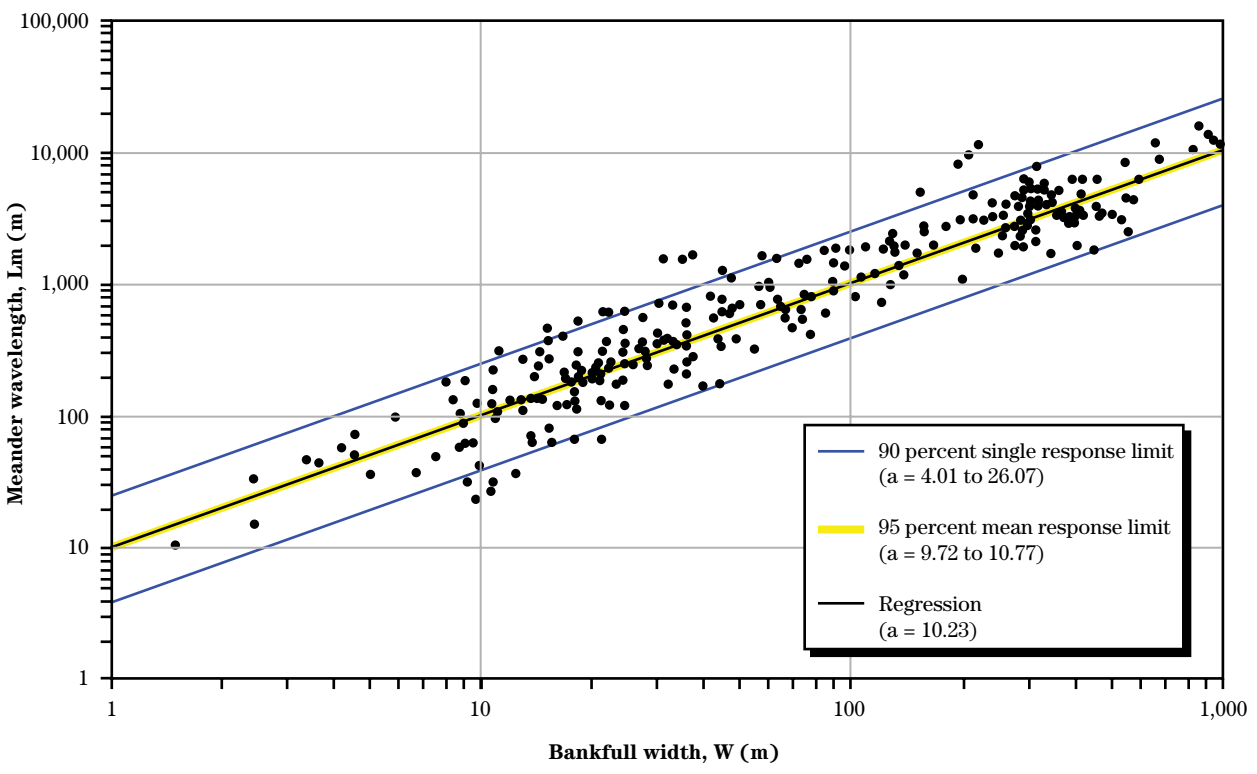


Table 12–1 Research and data sources for meander wavelengths

Researchers	Locations		No. of sites
Leopold and Wolman (1957)	United States rivers		21
Leopold and Wolman (1960)	Various sources	France	1
		United States	34
		Model river	1
		Total	36
Carlston (1965)	United States rivers		29
Schumm (1968)	Midwestern United States rivers		25
Chitale (1970)	Large alluvial rivers	Africa	1
		Canada	1
		India	16
		Pakistan	2
		United States	1
		Total	21
Williams (1986)	Various sources	Australia	2
		Canada	7
		Sweden	17
		Russia	1
		United States	16
		Model river	1
		Total	44
Thorne and Abt (1993)	Red River	1966	35
		1981	39
	Hydrographic surveys between Index, AR, and Shreveport, LA		1
		India	12
		Netherlands	1
		United Kingdom	48
		United States	18
		Total	154
Annable (1996)	Alberta, Canada		30
Cherry, Wilcock, and Wolman (1996)	United States rivers, predominantly sand bed		79

Leopold (1994) provided a hydraulic geometry relationship for meander wavelength as a function of both channel width and mean radius of curvature (fig 12–4 (FISRWG 1998)). His data include measurements from rivers, flumes, the Gulf Stream, and glaciers. He suggested that the relationships could be used to indicate stream instability if meander wavelength for a given stream did not plot closely to the predicted relationship.

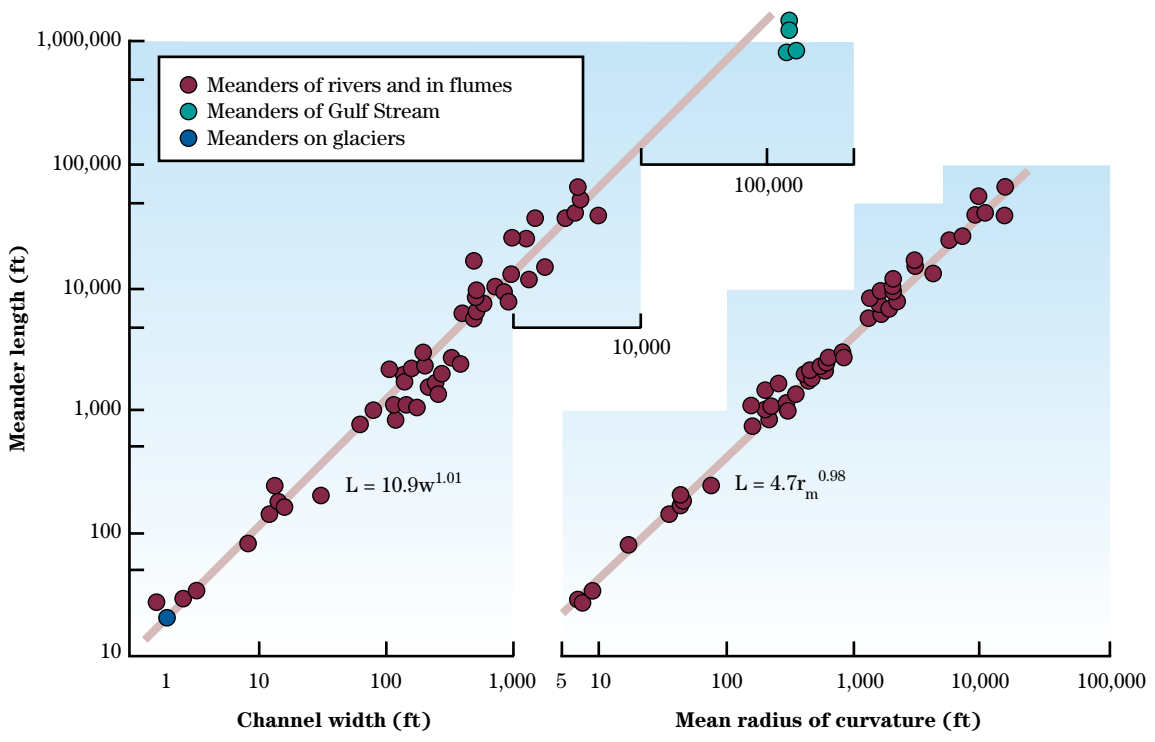
Other hydraulic geometry relationships for meander wavelength from the literature are given in table 12–2.

Additional guidance for determining meander geometry, including wavelength, along channel bend length, meander belt width, radius of curvature, and sinuosity are provided in Leopold (1994).

The channel meander length is simply the meander wavelength times the valley slope divided by the channel slope.

$$\text{channel meander length} = \frac{\text{wavelength} \times \text{valley slope}}{\text{channel slope}} \qquad \text{(eq. 12–4)}$$

Figure 12–4 Planform geometry relationships



From Leopold (1994)

Table 12–2 Hydraulic geometry relationships for meander wavelength

Author	Equation	Units
Leopold and Wolman (1960)	$\lambda = 10.9\ W^{1.01}$	ft
Inglis (1941)	$\lambda = 6.06\ W^{0.99}$	ft
Yalin (1992)	$\lambda = 6\ W$	length
Dury (1965)	$\lambda = 30\ Q_{bf}^{0.5}$	ft, ft^3/s
Carlston (1965)	$\lambda = 8.2\ Q_{bf}^{0.62}$	ft, ft^3/s
Carlston (1965)	$\lambda = 106.1\ Q_{ma}^{0.46}$	ft^3/s
Schumm (1967)	$\lambda = 1890\ Q_{ma}^{0.34}\ M^{0.74}$	ft, ft^3/s

Notes: λ = meander wavelength
W = width
Q_{bf} = bankfull discharge
Q_{ma} = mean annual discharge
M = silt-clay factor

(b) Layout and sine-generated curve

Once meander wavelength is determined, planform can be determined using an analogy method or by using the sine-generated curve. Using a reference reach as a guide, planform can be laid on a map with a string cut to the appropriate channel length. Assuming that the planform can be approximated by the sine-generated curve is a more analytical approach and was suggested by Langbein and Leopold (1966). Their theory of minimum variance is based on the hypothesis that the river will seek the most probable path between two fixed points (the path that provides the minimum variance of bed shear stress and friction). The sine-generated curve is defined in figure 12–5 and by the following dimensionless equation:

$$\phi = \omega \cos \frac{2\pi s}{M} \qquad \text{(eq. 12–5)}$$

where:

ϕ = angle of meander path with the mean longitudinal axis (degrees or radians)

ω = maximum angle a path makes with the mean longitudinal axis (degrees or radians)

s = curvilinear coordinate along the meander path (ft or m)

M = meander arc length (ft or m)

The shape parameter, ω, is a function of the channel sinuosity, P, which can be solved by numerical integration, or may be approximated by the following equation (Langbein and Leopold 1966), in which ω is in radians:

$$\omega = 2.2\sqrt{\frac{P-1}{P}} \qquad \text{(eq. 12–6)}$$

The shape parameter of a sine-generated curve defines the shape of the stream as shown in figure 12–6 (Langbein and Leopold 1966).

Calculation of the points on a sine-generated curve is a rather tedious numeric integration for ϕ. However, the integration can be accomplished using a computer program such as the one in the U.S. Army Corps of Engineers (USACE) Hydraulic Design Package: SAM (Thomas, Copeland, and McComas 2003). The sine-generated curve produces a very uniform meander pattern. The alignments of natural channels are rarely perfect sinusoids. Channels that are constructed as such, therefore, appear strange. A combination of the string layout method and the analytical approach would produce a more natural looking planform.

Figure 12–5 Definition of sine-generated curve

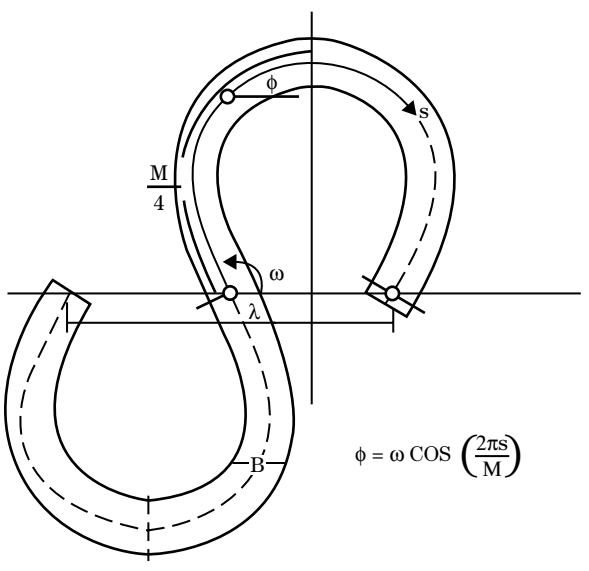
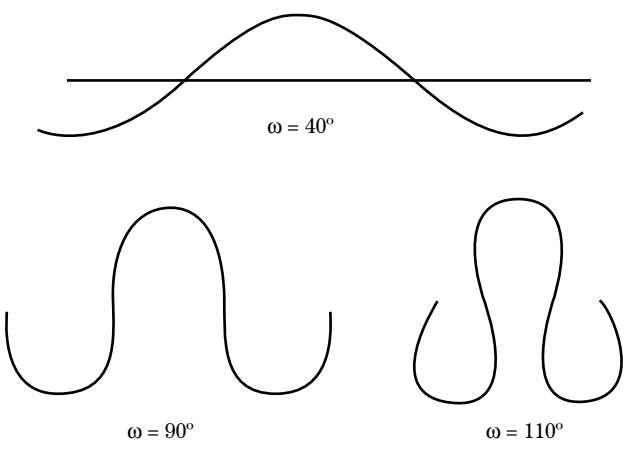


Figure 12–6 Effect of the shape factor on channel sinuosity with the sine-generated curve



Example problem: Channel alignment

Objective: Determine the planform layout for the channel designed in the previous problem. Use the sine-generated curve.

Given:

Base width	=	80 ft
Depth	=	2.9 ft (at channel-forming discharge)
Side slopes	=	1V:2.5H
Channel slope	=	0.0050
Valley slope	=	0.0055

Solution:

Step 1 Calculate the top width of the channel for the channel-forming discharge.

$$TW = 2zy + BW$$

$$TW = (2)(2.5)(2.9) + 80 = 94.5 \text{ say } 95 \text{ ft}$$

Step 2 Determine the meander wavelength directly from figure 12–3 or from the mean regression equation for the meander wavelength (fig. 12–4). If figure 12–3 is used directly, the top width must be converted to meters (95 divided by 3.281 which equals 29 m). If the equations from figure 12–4 are used, the unit conversion is not needed because the regression equation is dimensionless.

$$\lambda = 10.23TW$$

$$\lambda = 10.23(95) = 972 \text{ ft}$$

Step 3 Determine the distance along the channel for one wavelength.

$$M = \frac{\lambda(\text{Valley slope})}{\text{Channel slope}}$$

$$M = \frac{972(0.0055)}{(0.0050)} = 1{,}069 \text{ ft}$$

Step 4 Calculate xy coordinates for the channel using a spreadsheet or the USACE Hydraulic Design Package, SAM (Thomas, Copeland, and McComas 2003). Input is wavelength, λ, and channel length, M. Output is shown in table 12–3. The calculated shape factor, ω, is 34.9 degrees. The calculated planform amplitude is 199 feet. A planform plot developed from the SAM output is shown in figure 12–7. Note that this planform is very regular and does not replicate natural meanders. The designer should use the sine-generated curve layout as a guide and manipulate the actual centerline layout based on site constraints.

Table 12–3 Output for hydraulic computations using the SAM model

```
*****************************************************************
* SAMwin Software Registered to the US Army Corps of Engineers  *
*****************************************************************
*                              HYDRAULIC CALCULATIONS           *
*                                   Version 1.0
* A Product of the Flood Control Channels Research Program      *
* Coastal & Hydraulics Laboratory, USAE Engineer R & D Center   *
*                                in cooperation with
*            Owen Ayres & Associates, Inc., Ft. Collins, CO     *
*****************************************************************
TABLE 1. LIST INPUT DATA.

T1 Gravel bed River Example
T1 Base width 80 ft
T1 Depth 2.9 ft
T1 Side Slopes 1V : 2.5H
T1 Top Width 95 ft
MG  972  1069
$$END

INPUT IS COMPLETE.
   ***********************************************************
   *
   *  PLANFORM GEOMETRY FOR A MEANDERING Sand bed STREAM
   *
   ***********************************************************

                                          MAXIMUM
    WAVE      MEANDER                   DEFLECTION
   LENGTH     LENGTH      SINUOSITY    ANGLE-DEG      AMPLITUDE
   972.00    1069.00        1.10        34.933         198.97

  COORDINATES ALONG ONE MEANDER WAVELENGTH

  ALONG THE       DEFLECTION       PERPENDICULAR TO      ALONG THE
   CHANNEL          ANGLE           VALLEY SLOPE       VALLEY SLOPE
                   DEGREES

      S             THETA                Y                  X

      0.00           34.93              0.00               0.00
     10.69           34.86              6.12               8.77
     21.38           34.66             12.21              17.55
     32.07           34.31             18.26              26.36
     42.76           33.84             24.25              35.22
     53.45           33.22             30.16              44.13
     64.14           32.48             35.96              53.11
```

Table 12–3 Output for hydraulic computations using the SAM model—Continued

S	THETA	Y	X
74.83	31.61	41.63	62.17
85.52	30.61	47.15	71.32
96.21	29.49	52.51	80.58
106.90	28.26	57.67	89.94
117.59	26.92	62.62	99.41
128.28	25.47	67.34	109.00
138.97	23.91	71.80	118.72
149.66	22.27	76.00	128.55
160.35	20.53	79.90	138.50
171.04	18.72	83.49	148.57
181.73	16.83	86.75	158.75
192.42	14.87	89.67	169.03
203.11	12.86	92.23	179.41
213.80	10.79	94.42	189.87
224.49	8.69	96.23	200.41
235.18	6.55	97.65	211.00
245.87	4.38	98.67	221.64
256.56	2.19	99.28	232.31
267.25	0.00	99.48	243.00
277.94	-2.19	99.28	253.69
288.63	-4.38	98.67	264.36
299.32	-6.55	97.65	275.00
310.01	-8.69	96.23	285.59
320.70	-10.79	94.42	296.13
331.39	-12.86	92.23	306.59
342.08	-14.87	89.67	316.97
352.77	-16.83	86.75	327.25
363.46	-18.72	83.49	337.43
374.15	-20.53	79.90	347.50
384.84	-22.27	76.00	357.45
395.53	-23.91	71.80	367.28
406.22	-25.47	67.34	377.00
416.91	-26.92	62.62	386.59
427.60	-28.26	57.67	396.06
438.29	-29.49	52.51	405.42
448.98	-30.61	47.15	414.68
459.67	-31.61	41.63	423.83
470.36	-32.48	35.96	432.89
481.05	-33.22	30.16	441.87
491.74	-33.84	24.25	450.78
502.43	-34.31	18.26	459.64
513.12	-34.66	12.21	468.45
523.81	-34.86	6.12	477.23

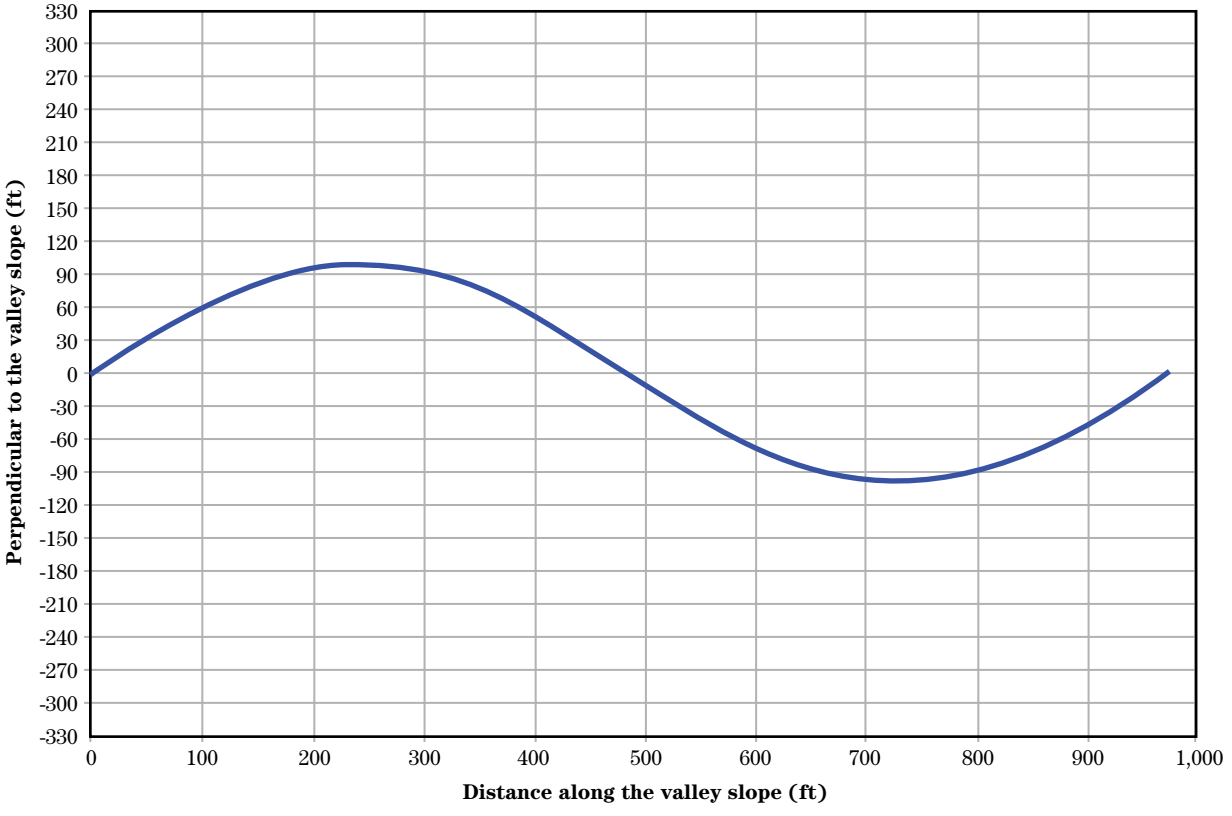
Table 12–3 Output for hydraulic computations using the SAM model—Continued

S	THETA	Y	X
534.50	-34.93	0.00	486.00
545.19	-34.86	-6.12	494.77
555.88	-34.66	-12.21	503.55
566.57	-34.31	-18.26	512.36
577.26	-33.84	-24.25	521.22
587.95	-33.22	-30.16	530.13
598.64	-32.48	-35.96	539.11
609.33	-31.61	-41.63	548.17
620.02	-30.61	-47.15	557.32
630.71	-29.49	-52.51	566.58
641.40	-28.26	-57.67	575.94
652.09	-26.92	-62.62	585.41
662.78	-25.47	-67.34	595.00
673.47	-23.91	-71.80	604.72
684.16	-22.27	-76.00	614.55
694.85	-20.53	-79.90	624.50
705.54	-18.72	-83.49	634.57
716.23	-16.83	-86.75	644.75
726.92	-14.87	-89.67	655.03
737.61	-12.86	-92.23	665.41
748.30	-10.79	-94.42	675.87
758.99	-8.69	-96.23	686.41
769.68	-6.55	-97.65	697.00
780.37	-4.38	-98.67	707.64
791.06	-2.19	-99.28	718.31
801.75	0.00	-99.48	729.00
812.44	2.19	-99.28	739.69
823.13	4.38	-98.67	750.36
833.82	6.55	-97.65	761.00
844.51	8.69	-96.23	771.59
855.20	10.79	-94.42	782.13
865.89	12.86	-92.23	792.59
876.58	14.87	-89.67	802.97
887.27	16.83	-86.75	813.25
897.96	18.72	-83.49	823.43
908.65	20.53	-79.90	833.50
919.34	22.27	-76.00	843.45
930.03	23.91	-71.80	853.28
940.72	25.47	-67.34	863.00
951.41	26.92	-62.62	872.59
962.10	28.26	-57.67	882.06
972.79	29.49	-52.51	891.42
983.48	30.61	-47.15	900.68

Table 12–3 Output for hydraulic computations using the SAM model—Continued

S	THETA	Y	X
994.17	31.61	-41.63	909.83
1004.86	32.48	-35.96	918.89
1015.55	33.22	-30.16	927.87
1026.24	33.84	-24.25	936.78
1036.93	34.31	-18.27	945.64
1047.62	34.66	-12.21	954.45
1058.31	34.86	-6.12	963.23
1069.00	34.93	0.00	972.00

Figure 12–7 Planform layout for one meander wavelength from sine-generated curve for example problem: wavelength = 972 ft; amplitude = 199 ft; sinuosity = 1.1



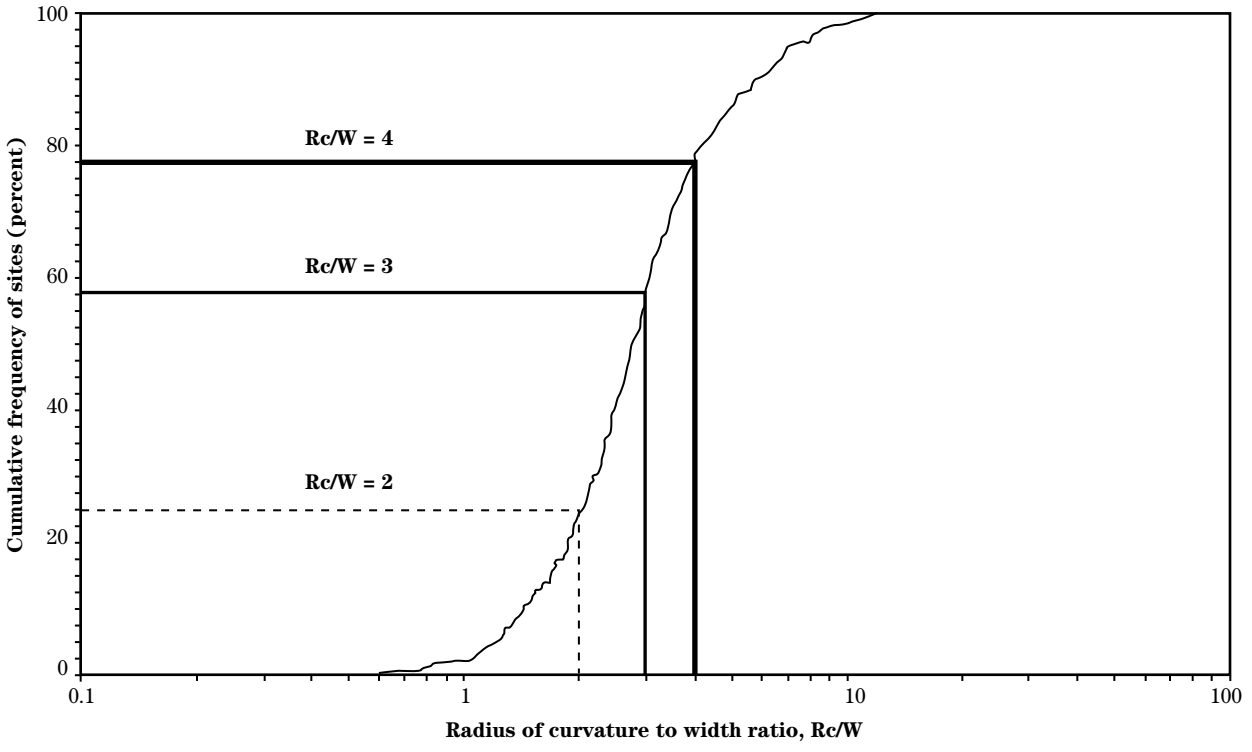
(c) Radius of curvature

The radius of planform curvature is not constant in the sine-generated curve, but ranges from a maximum value at the inflection point to a minimum curvature around the bend apex. The average radius of curvature is centered at the bend apex for a distance of approximately a sixth of the channel meander length.

Most reaches of stable meandering rivers have radius of curvature-to-width ratios between 1.5 and 4.5. Of the 438 sites used to derive the wavelength-width relationship in figure 12–3, radius of curvature is recorded for 263 of the sites. This subset was used to develop a cumulative distribution curve of radius of curvature-to width ratios (fig. 12–8). This figure shows that 33.5 percent, 52.9 percent, and 71.2 percent of the sites have radius of curvature-to-width ratios between 2 and 3, 2 and 4, and 1.5 and 4.5, respectively. The final planform layout should have ratios within the normal range.

If the calculated meander length is too large or if the required meander belt width is unavailable, grade control structures may be required to reduce the channel slope and stabilize the bed elevations.

Figure 12–8 Cumulative distribution of radius of curvature-to-width ratio derived from a composite data set of 263 sites



654.1203 Natural variability

Natural streams and rivers are rarely of uniform depth and width. Variability in channel width and depth can either be allowed to develop naturally or can be part of the project design. Sand-bed streams have the ability to create natural variability in channel form rather quickly because they are characterized by significant bed-material sediment transport. Gravel-bed streams typically adjust much more slowly. Streams with very little bed-material movement may not adjust at all. If variability is to be included in the project design, dimensions for cross sections in riffles and pools can be obtained from stable reaches of the existing stream or from reference reaches.

(a) Natural variability in width for gravel-bed rivers

Gravel-bed rivers are typically characterized by riffles and pools which correspond to bends and crossings in sand-bed rivers. In stable gravel-bed rivers, riffles are wider and shallower than the average channel width, and the pools tend to be deeper and somewhat narrower. In meandering gravel-bed rivers, the pools tend to be in the bends and the riffles at the inflection points. In the design of gravel-bed channels, the natural variability in cross-sectional width can be estimated using hydraulic geometry relationships or reference reaches. As with all hydraulic geometry and analogy methods, the design tools should be developed from physiographically similar watersheds.

Hey and Thorne (1986) developed hydraulic geometry relationships for meandering gravel-bed rivers in the United Kingdom. Their regression equations were based on surveys from 62 stable self-formed channels in erodible material with well-defined flood plains. At most sites, the banks were either cohesive or composite. Composite banks are defined as noncohesive sand and gravel layers overlain by a cohesive layer. These sites include the range of data used in developing the regression equations that are described in more detail in NEH654.09. A typical example of one of the Hey and Thorne rivers is shown in figure 12–9.

Riffle spacing and riffle width are determined from regression equations as a function of the mean channel width. The mean channel width is determined from one of three hydraulic geometry relationships described in NEH654.09. The Hey and Thorne hydraulic geometry relations for mean width consider the density of vegetation along the channel banks. The equations for riffle spacing are applicable for all bank conditions. The riffle spacing is given as a function of mean width:

$$Z = 6.31W \qquad \text{(eq. 12–7)}$$

where:
Z = the riffle spacing
W = the mean channel top width

Most of the data fell between the equations as shown in figure 12–10.

$$Z = 10W \qquad \text{(eq. 12–8)}$$

$$Z = 4W \qquad \text{(eq. 12–9)}$$

Riffle spacing tends to be nearer 4 channel widths on steeper gradients, increasing to 10 channel widths with more gradual slopes.

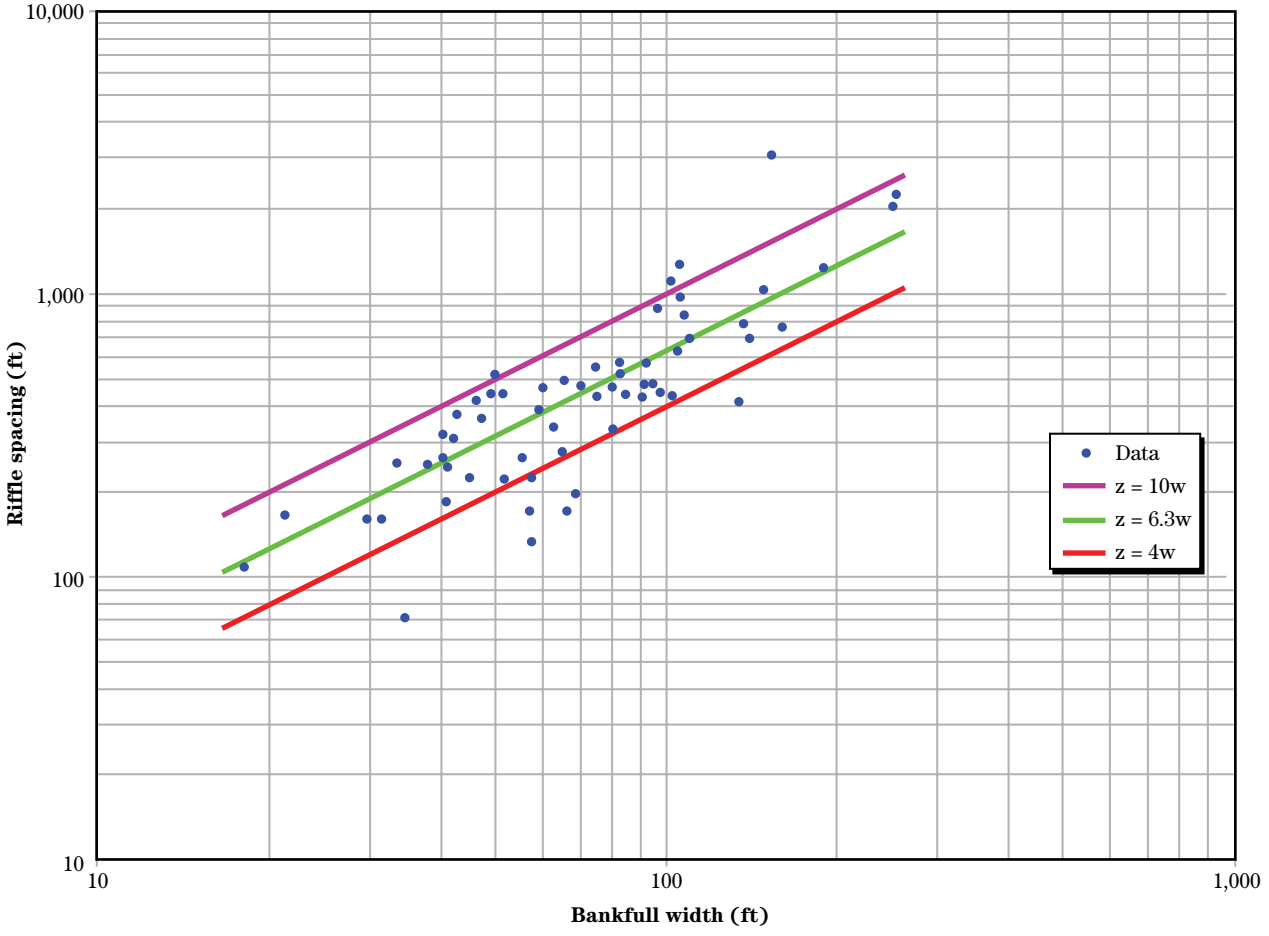
The riffle mean width, RW, was given as:

$$RW = 1.034W \qquad \text{(eq. 12–10)}$$

Hey and Thorne's riffle widths varied between 1.5W and 0.75 W.

Figure 12–9 Typical British gravel-bed river used in Hey and Thorne (1986) study

Figure 12–10 Relation between riffle spacing, Z, and bankfull channel width, W



(b) Riffle and pool spacing in nearly straight rivers

In rivers that are nearly straight (sinuosity less than 1.2), riffle and pool spacing may be set as a function of channel width. The empirical guide of 5 to 7 channel widths applies here (Knighton 1984). Two times this riffle spacing gives the total channel length through one meander pattern.

(c) Natural variability around meander bendways

Thorne (1988) and Thorne and Soar (2001) compiled empirical data sets of cross-sectional and planform dimensions from meander bends in the Red River between Index, Arkansas, and Shreveport, Louisiana. The Red River in this reach is typical of relatively large meandering rivers, with a wide variety of both bend geometries and bank materials. These studies provided a useful baseline database for examining the variability of width around meander bends, location of pools, and maximum pool depths. Equations were developed to define natural variability around the meander bends. Of course, if applied elsewhere, these equations should be used with caution.

The bends in the Red River data set were classified as one of three types based on the Brice (1975) classification system: equiwidth meanders, denoted as Type e (T_e) meanders (fig. 12–11); meanders with point bars, denoted as Type b (T_b) meanders (fig. 12–12); and meanders with point bars and chute channels, denoted as Type c (T_c) meanders (fig. 12–13). The Red River meander bend geometry data set is shown in table 12–4.

- **Equiwidth meandering**—Equiwidth indicates that there is only minor variability in channel width around meander bends. These channels are generally characterized by low width-to-depth ratios, erosion resistant banks, fine-grain bed material (sand or silt), low bed-material load, low velocities, and low stream power. Channel migration rates are relatively low because the banks are naturally stable.
- **Meandering with point bars**—Meandering with point bars refers to channels that are significantly wider at bendways than crossings, with well-developed point bars, but few chute channels. These channels are generally

Figure 12–11 Equiwidth meandering river, Type e (T_e)

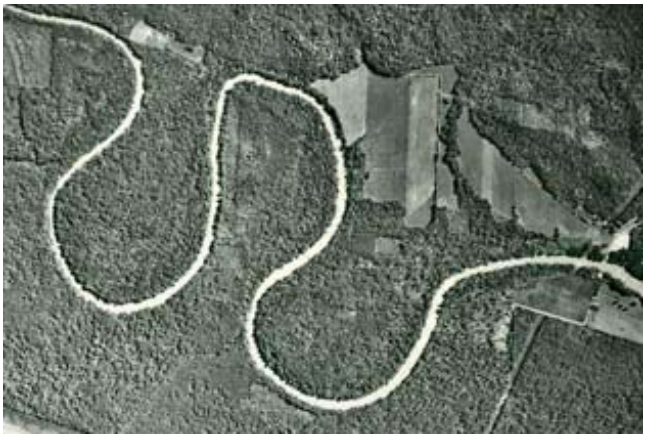


Figure 12–12 Meandering with point bars, Type b (T_b)

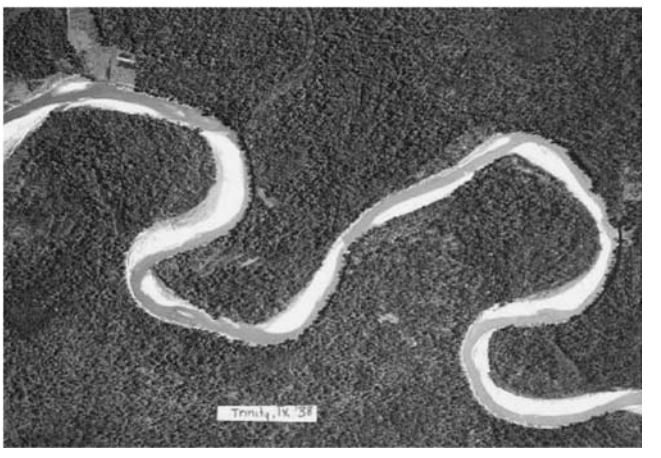


Figure 12–13 Meandering with point bars and chute channels, Type c (T_c)

characterized by intermediate width-to-depth ratios, moderately erosion-resistant banks, medium-grained bed material (sand or gravel), medium bed-material load, medium velocities, and medium stream power. Channel migration rates are likely to be moderate unless banks are stabilized.

- **Meandering with point bars and chute channels**—Meandering with point bars and chute channels refers to channels that are much wider at bendways than crossings, with well-developed point bars and frequent chute channels. These channels are generally characterized by moderate to high width-to-depth ratios, highly erodible banks, medium to coarse-grained bed material (sand, gravel and/or cobbles), heavy bed-material load, moderate to high velocities, and moderate to high stream power. Channel migration rates are likely to be moderate to high unless banks are stabilized.

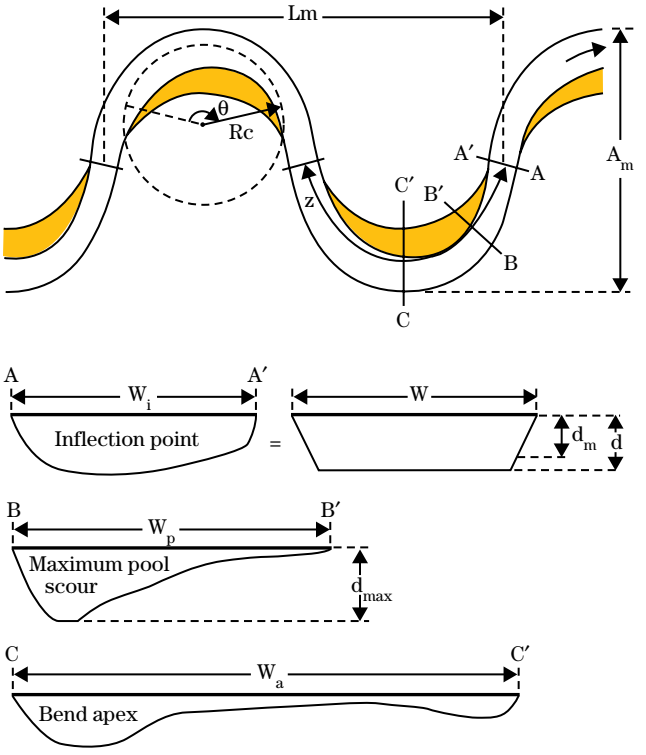
Ranges of physical characteristics pertaining to each of the meander bend types are addressed in more detail in NEH654.09. Figure 12–14 provides a definition sketch for channel cross-sectional geometries and dimensions through a meander.

Table 12–4 Ranges of physical characteristics found in different meander bend types identified from the 1981 Red River hydrographic survey between Index, AR, and Shreveport, LA

	n	S (10^6)	P	W_i / d_m	d_{max} / d_i	Rc / W_i
Type e	20 (8)	65 to 268 (133 to 268)	1.0 to 2.1 (1.2 to 2.1)	34.2 to 74.1 (38.3 to 74.1)	1.6 to 2.4 (1.7 to 2.4)	0.9 to 9.3 (0.9 to 5.2)
Type b	34 (19)	76 to 294 (105 to 294)	1.0 to 2.0 (1.1 to 2.0)	36.8 to 121.0 (36.8 to 102.4)	1.5 to 2.6 (1.7 to 2.6)	1.5 to 9.1 (1.5 to 6.1)
Type c	13 (10)	91 to 201 (91 to 201)	1.1 to 2.3 (1.2 to 2.3)	33.5 to 88.2 (33.5 to 88.2)	1.6 to 2.4 (1.6 to 2.4)	2.2 to 6.8 (2.2 to 5.2)

Note:
n = number of meander bends studied
S = water surface slope
P = sinuosity
W_i / d_m = inflection point width-to-mean depth ratio
d_{max} / d_i = maximum scour depth in pool-to-mean depth at inflection point
Rc / W_i = radius of curvature-to-inflection point width ratio
Values in parentheses refer to meander bends with sinuosity 1.2 or greater.

Figure 12–14 Meander cross-sectional dimensions for variability design



Note: Point bars defined by shaded regions

Lm	=	meander wavelength	Z	=	meander arc length (riffle spacing)
A_m	=	meander belt width	Rc	=	radius of curvature
θ	=	meander arc angle	W	=	reach average bankfull width
d	=	depth of trapezoidal cross section	d_m	=	mean depth (cross-sectional area / W)
d_{max}	=	maximum scour depth in bendway pool	W_i	=	width at meander inflection point
W_p	=	width at maximum scour location	W_a	=	width at meander bend apex

(d) Width variability around meander bends

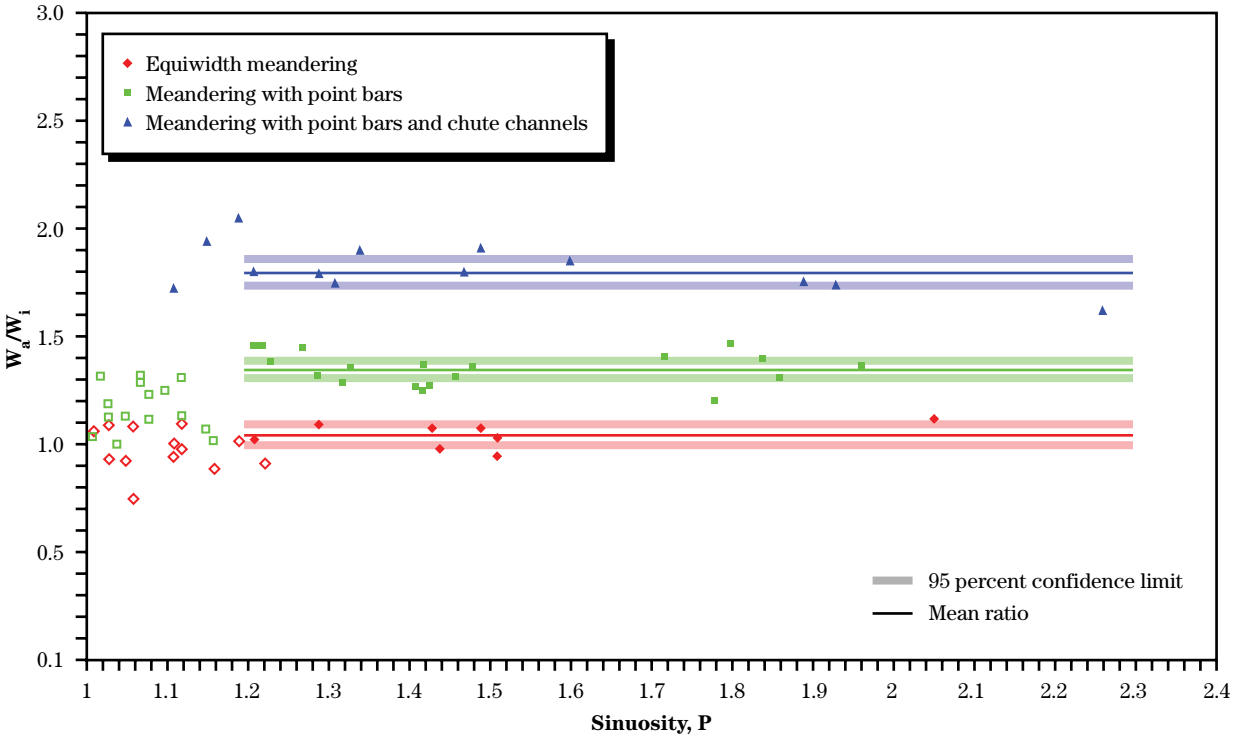
Two dimensionless parameters can be used to describe the width variability around meander bends, based on the enhanced Red River data set. These are the ratio of bend apex width to inflection point width, W_a/W_i, and the ratio of width at the location of maximum bend pool scour to inflection point width, W_p/W_i. Theoretically, these parameters adjust according to the degree of curvature and the type of meander bend. To derive new morphological relationships, sinuosity, P, was preferred as the independent variable, rather than the radius of curvature-to-width ratio. The latter would have resulted in width appearing on both sides of the regression equations.

Morphologic relationships for the width ratios as a function of meander type were developed for channels with sinuosities greater than 1.2. This is a commonly accepted threshold between nearly straight channels with only slight sinuosity and meandering channels with moderate to high sinuosity. The bed apex width to the inflection point width ratio, W_a/W_i, was found to be independent of sinuosity. Data are plotted with confidence limits in figure 12–15. Values for the ratios for each type of meander bend can be determined from table 12–5 and the following equation, where p denotes the level of significance and corresponds to the 100(1–p) percent confidence level.

Morphologic relationships for the width ratios as a function of meander type were developed for the ratio of pool width at the location of maximum scour to inflection point width (W_p/W_i) for channels with sinuosities greater than 1.2. This ratio was also found to be independent of sinuosity. Data and confidence limits are plotted in figure 12–16 (source data: 1981 Red River hydrographic survey). Values for the ratios for each type of meandering river can be determined from the following equation and table 12–6.

$$\left(\frac{W_a}{W_i}\right)_p = a + u_p \qquad \text{(eq. 12–11)}$$

Figure 12–15 Ratio of bend apex width to inflection point width, W_a/W_i as a function of meander bend type only, for sinuosities of at least 1.2. Confidence limits of a mean response are shown at the 95 percent level. (Source data: 1981 Red River hydrographic survey)



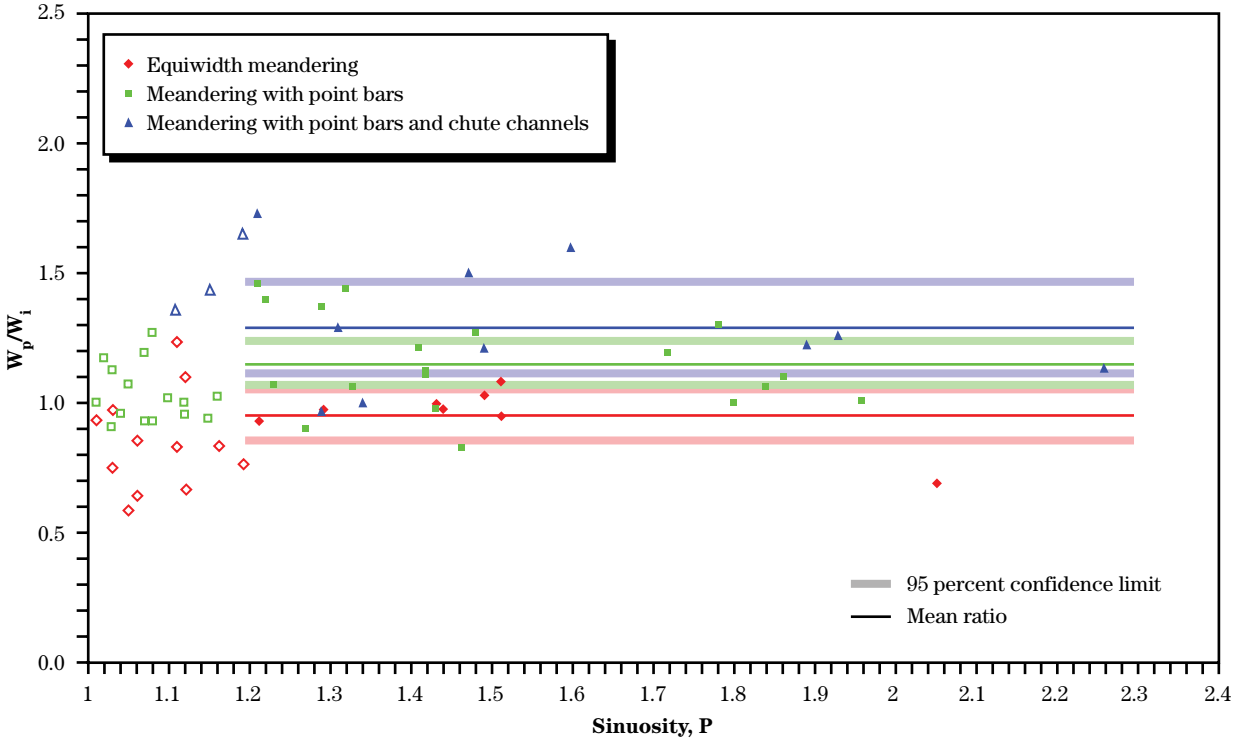
Note: Filled symbols = sinuosity of at least 1.2; empty symbols = sinuosity less than 1.2

Table 12–5 Constant values used to estimate the mean ratio of bend apex width to inflection point width, W_a/W_i, within confidence bands for different types of meander bends and for sites with sinuosity of at least 1.2. Coefficients pertaining to the 99, 95, and 90 percent confidence limits are given.

	a	$u_{0.01}$	$u_{0.05}$	$u_{0.1}$
Type e	1.05	0.08	0.05	0.04
		(0.29)	(0.20)	(0.16)
Type b	1.35	0.05	0.04	0.03
		(0.27)	(0.20)	(0.16)
Type c	1.79	0.09	0.06	0.05
		(0.36)	(0.25)	(0.20)

Note: Values given refer to mean response confidence limits. Values in parentheses are used to calculate single response confidence limits.

Figure 12–16 Ratio of pool width (at maximum scour location) to inflection point width, W_p/W_i as a function of meander bend type only, for sinuosities of at least 1.2. Confidence limits of a mean response are shown at the 95 percent level.



Note: Filled symbols = sinuosity of at least 1.2 empty symbols = sinuosity less than 1.2

Table 12–6 Constant values used to estimate the mean ratio of pool width (at maximum scour location) to inflection point width, W_p/W_i, within confidence bands for different types of meander bends and for sites with sinuosity of at least 1.2. Coefficients pertaining to the 99, 95, and 90 percent confidence limits are given.

	a	$u_{0.01}$	$u_{0.05}$	$u_{0.1}$
Type e	0.95	0.15	0.10	0.08
		(0.56)	(0.38)	(0.30)
Type b	1.15	0.12	0.09	0.07
		(0.64)	(0.47)	(0.39)
Type c	1.29	0.26	0.18	0.14
		(1.07)	(0.74)	(0.60)

Note: Values given refer to mean response confidence limits. Values in parentheses is used to calculate single response confidence limits.

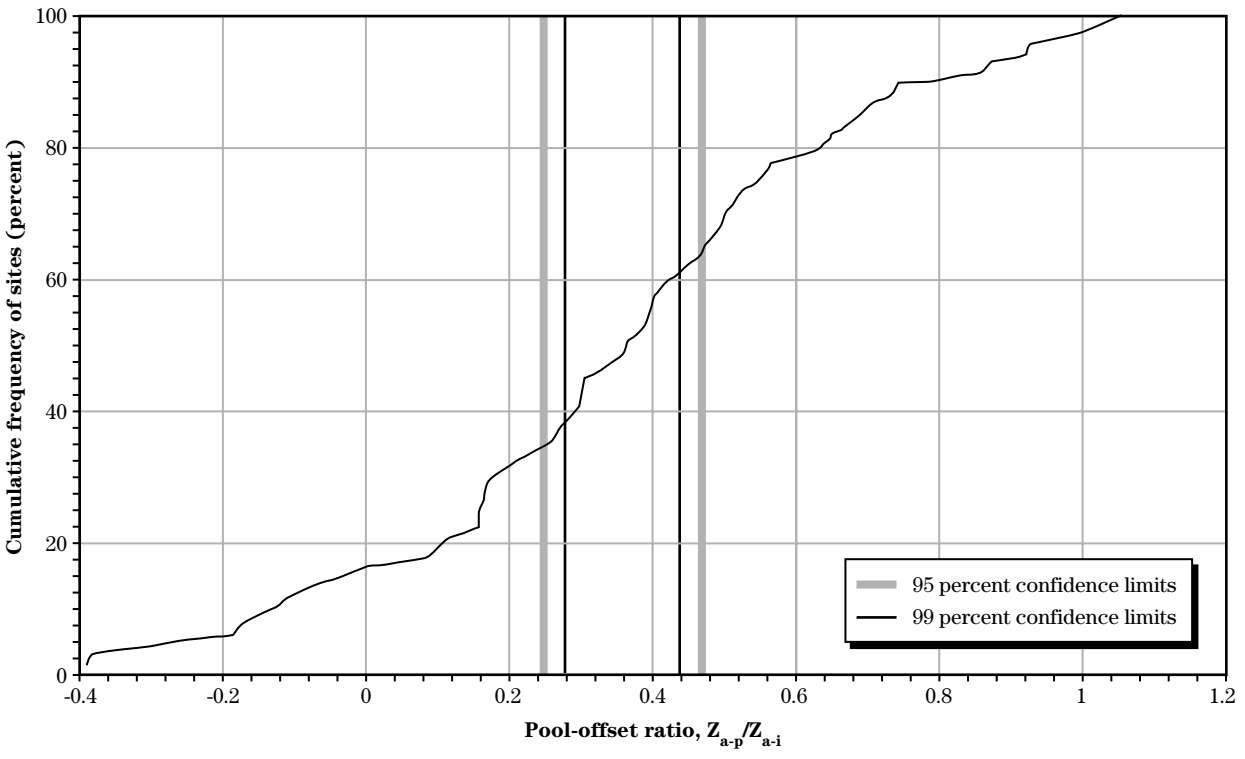
(e) Location of the pool in a meander bend

While the location of meander inflection points and bend apexes are geometrically defined, the location of pools, defined by the position of maximum bend scour, is variable. Pool location is controlled by the meander configuration, complex velocity distribution, and large-scale coherent flow structures which pulse sediment along the channel to form alternate zones of scour and fill. In natural meanders, the deepest pool is usually located downstream from the bend apex. The pool location in a meander bend can be represented empirically by a pool-offset ratio, defined as the ratio of the channel distance between bend apex and maximum scour location to the channel distance between bend apex and downstream inflection point, Z_{a-p} / Z_{a-i}. The pool–offset ratio was found to be independent of sinuosity. Neither was a distinct relationship found for the different meander types. The range and cumulative distribution function for the pool-offset ratio is shown in figure 12–17 (source data: 1981 Red River hydrographic survey). The mean value for the ratio was 0.36 and the range was –0.4 to 1.08.

(f) Maximum scour in bendways

Maximum scour depth is calculated to incorporate deep pools in constructed channels and to estimate required toe depths for bank protection. Data from a wide range of rivers (Thorne and Abt 1993; Maynord 1996) were used to develop morphological equations for the maximum scour depth in pools. These maximum scour depths are based on the surveyed maximum local depth at the bend. The data were divided into two subsets using a width-to-depth threshold value of 60, which is an approximate modal value.

Figure 12–17 Cumulative distribution of the pool-offset ratio, Z_{a-p}/Z_{a-i}, for all types of meander bends studied. Confidence limits on the mean response are shown.



The best-fit morphological relationships are given by Thorne and Soar (2001) as:

$$\frac{W_i}{d_m} < 60 \qquad \text{(eq. 12–12)}$$

$$\frac{d_{max}}{d_m} = 2.14 - 0.19 \ln\left(\frac{Rc}{W_i}\right) \qquad \text{(eq. 12–13)}$$

$$\frac{W_i}{d_m} \geq 60 \qquad \text{(eq. 12–14)}$$

$$\frac{Rc}{W_i} < 10 \qquad \text{(eq. 12–15)}$$

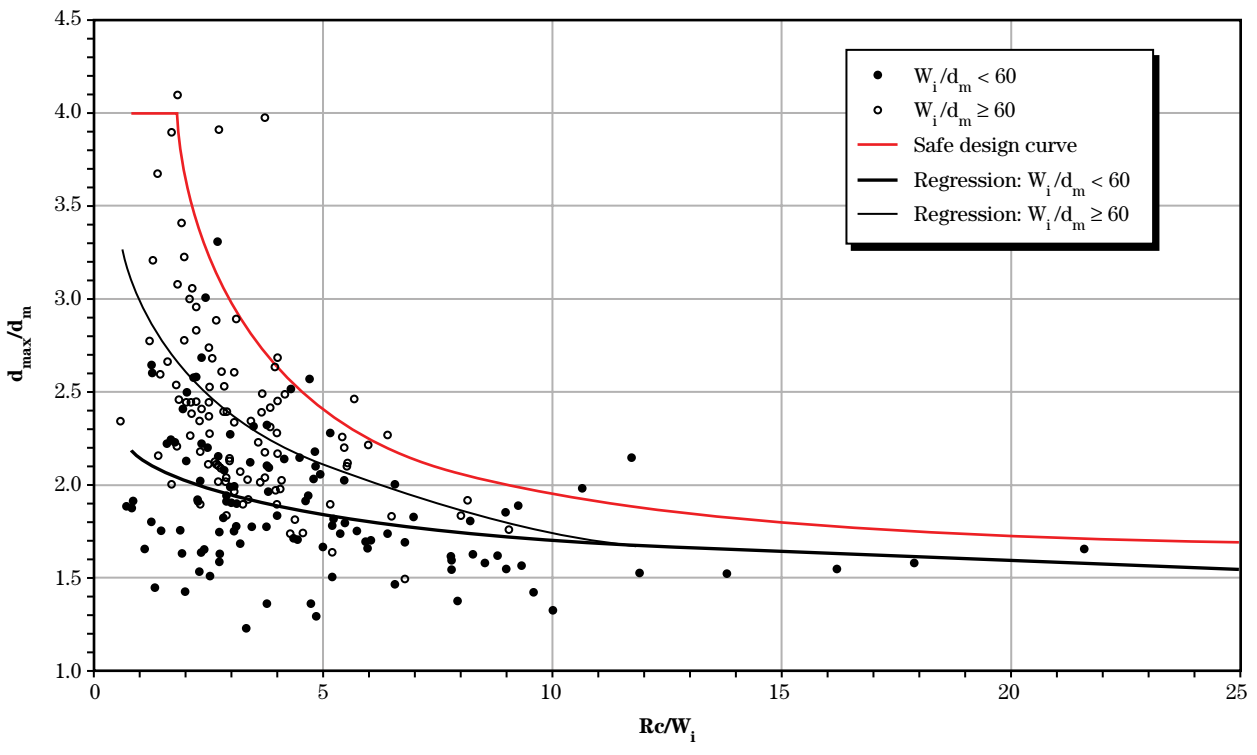
$$\frac{d_{max}}{d_m} = 2.98 - 0.54 \ln\left(\frac{Rc}{W_i}\right) \qquad \text{(eq. 12–16)}$$

A practical, safe design curve may then be defined by considering both equations as:

$$\frac{d_{max}}{d_m} = 1.5 + 4.5\left(\frac{Rc}{W_i}\right)^{-1} \qquad \text{(eq. 12–17)}$$

This equation is an asymptotic relationship with a theoretical minimum d_{max}/d_m of 1.5, representing pool scour depths expected in a straight channel with a pool-riffle bed topography. From this upper-bound relationship, d_{max}/d_m ranges from 4 to 3 for Rc/W_i between 1.8 and 3. For channels with an Rc/W_i less than 1.8, pool depth is independent of bend curvature. The recommended dimensionless scour depth should be 4. All three relationships are portrayed in figure 12–18 (Thorne and Abt 1993; Maynord 1996), which show that this equation is a safe curve for both classes of W_i/d_m. More information on scour and how it relates to specific project features is provided in NEH654.14.

Figure 12–18 Dimensionless maximum scour depth in meander pools as a function of radius of curvature-to-width ratio



654.1204 Practical channel design equations for meander bend geometry

It is possible to derive a mean band of uncertainty, u, suitable for all three types of meander bends and to provide a set of practical design equations. The cumulative effects of Type e, Type b, and Type c bends are represented by the binary parameters, T_e, T_b and T_c, respectively. The value of T_e has a value of 1 for all three types of bend and represents the smallest planform width ratio. If point bars are present, but chute channels are rare, T_b is assigned a value of 1, and T_c is assigned a value of 0. If point bars are present and chute channels are common, both T_b and T_c are assigned values of 1. Obviously T_c can only be given a value of 1 when T_b has a value of 1.

Bend apex ($P \geq 1.2$)

$$\frac{W_a}{W_i} = 1.05T_e + 0.30T_b + 0.44T_c \pm u \qquad \text{(eq. 12–18)}$$

Pool width ($P \geq 1.2$)

$$\frac{W_p}{W_i} = 0.95T_e + 0.20T_b + 0.14T_c \pm u \qquad \text{(eq. 12–19)}$$

For all three bend types and sinuosities greater than 1, the pool offset ratio is given by:

Pool-offset ($P>1.0$)

$$\frac{Z_{a-p}}{Z_{a-i}} = 0.36 \pm u \qquad \text{(eq. 12–20)}$$

Values of u refer to confidence limits on the mean response as given in table 12–7.

A practical design equation for predicting or constructing maximum scour depths at bends is the upper-bound curve in figure 12–17, given by the following equation:

$$\frac{d_{max}}{d_m} = 1.5 + 4.5\left(\frac{R_c}{W_i}\right)^{-1} \qquad \text{(eq. 12–21)}$$

For sites where active meandering is not permitted, bank protection will be required along the outer bank to prevent erosion. In addition, this equation should be used together with bank stability charts to establish whether bank stabilization against mass failure is also necessary.

Table 12–7 Uncertainty, u, in estimates of width variability around meander bends and location of pools. Values refer to confidence limits on the mean response.

Confidence limits %	W_a / W_i	W_p / W_i	Z_{a-p} / Z_{a-i}
99	0.07	0.17	0.11
95	0.05	0.12	0.08
90	0.04	0.10	0.07

654.1205 Bankline migration

Bankline migration is a natural process associated with natural meandering channels. Meander loops tend to move downstream as river processes erode the outside of bends and deposit sediment on point bars. The ability to forecast adjustments in planform is important to the planning and design of any project where highways or structures could be damaged. The rate of bank migration at a given site is a function of erosional forces and resisting forces. The variables affecting erosional forces include discharge, cross-sectional geometry, sediment load, bed roughness, bedforms and bars, and the geometry of the bend itself. The variables affecting resistance forces include bank geometry, the composition of the bank, bank vegetation, pore water pressure, freezing and thawing, and wetting and drying. Due to the wide variability in significant variables, it is difficult to develop an algorithm that can reliably predict bankline migration rates.

Nanson and Hickin (1986) compiled data for 18 gravel-bed rivers in western Canada and reported maximum bankline migration rates at the bend apex (fig. 12–19). Cherry, Wilcock, and Wolman (1996) used 133 data sets from meandering sand-bed rivers collected by James Brice of the USGS to develop an empirical relationship to estimate bankline migration. They related mean annual flow and channel width to the mean erosion rate along the entire length of channel (fig. 12–20). They concluded that the simple correlation was inadequate. However, it does provide an order of magnitude estimate for the mean erosion rate and can be used to estimate a range of probable erosion rates.

Figure 12–19 Bankline migration rates in gravel-bed rivers

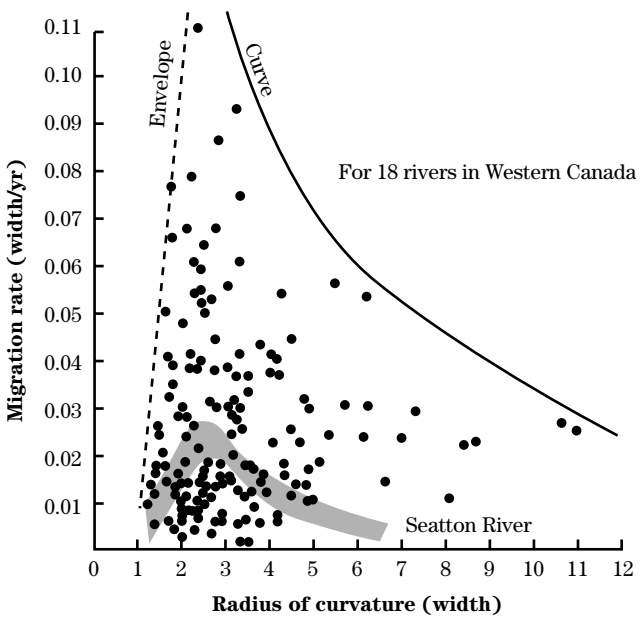


Figure 12–20 Average bank erosion rate

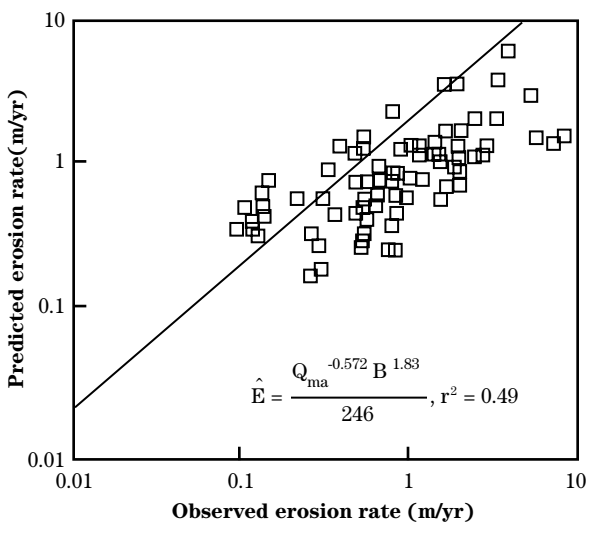
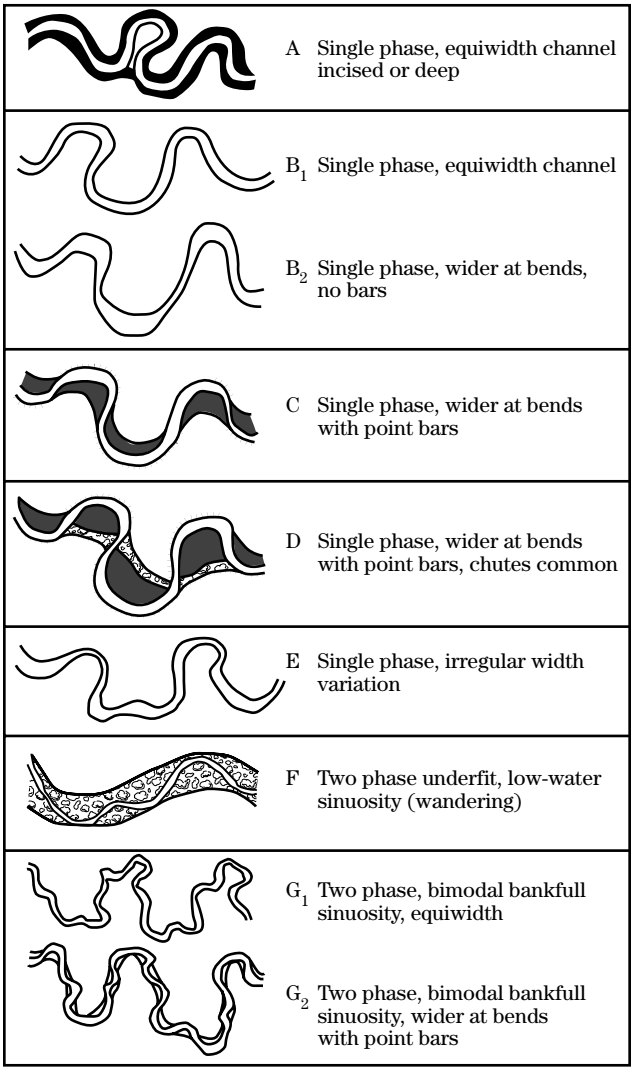


Figure 12–21 Modified Brice classification system for estimating bankline migration



Ayres Associates (2004) used the same data set as Cherry, Wilcock, and Wolman (1996), relating the maximum rate of apex movement to the channel width at the apex. They segregated the data for four channel types: B1–single phase equiwidth, B2–single phase wider at bends, C–single phase with point bars, and A–single phase incised or deep. Their classification is a modification of the Brice (1975) classification system shown in figure 12–21 (Ayers Associates 2004). The scatter in the Ayres data (fig. 12–22 is about the same. Note that the Cherry, Wilcock, and Wolman (1996) data and bank migration rates can only be estimated in an approximate sense. Ayres Associates plotted a cumulative percentage of apex movement curve (fig. 12–23) which provides a useful tool for predicting bankline migration in terms of risk and uncertainty.

Several researchers have developed two-dimensional, depth-averaged numerical models to predict bankline migration. These models are data intensive and should be considered research tools. Garcia, Bittner, and Nino (1994) related the local erosion rate to the difference in the average velocity and the near-bank velocity. Odgaard (1986) related erosion to the difference in average depth and near-bank depth. The models produce relatively accurate velocity distributions in the meandering channel; however, bank resistance coefficients must be empirically determined or calibrated to existing conditions at specific sites. The high degree of variability in bank composition in meandering alluvial systems makes application of these models difficult.

The most reliable method for predicting bankline migration rate is to estimate historical rates from aerial photos of the project river. It must be recognized that rates at a specific site will change as the planform changes. In addition, erosion rates change with cyclic climate changes and changes in the watershed.

Figure 12–22 Bankline migration—apex movement versus channel width

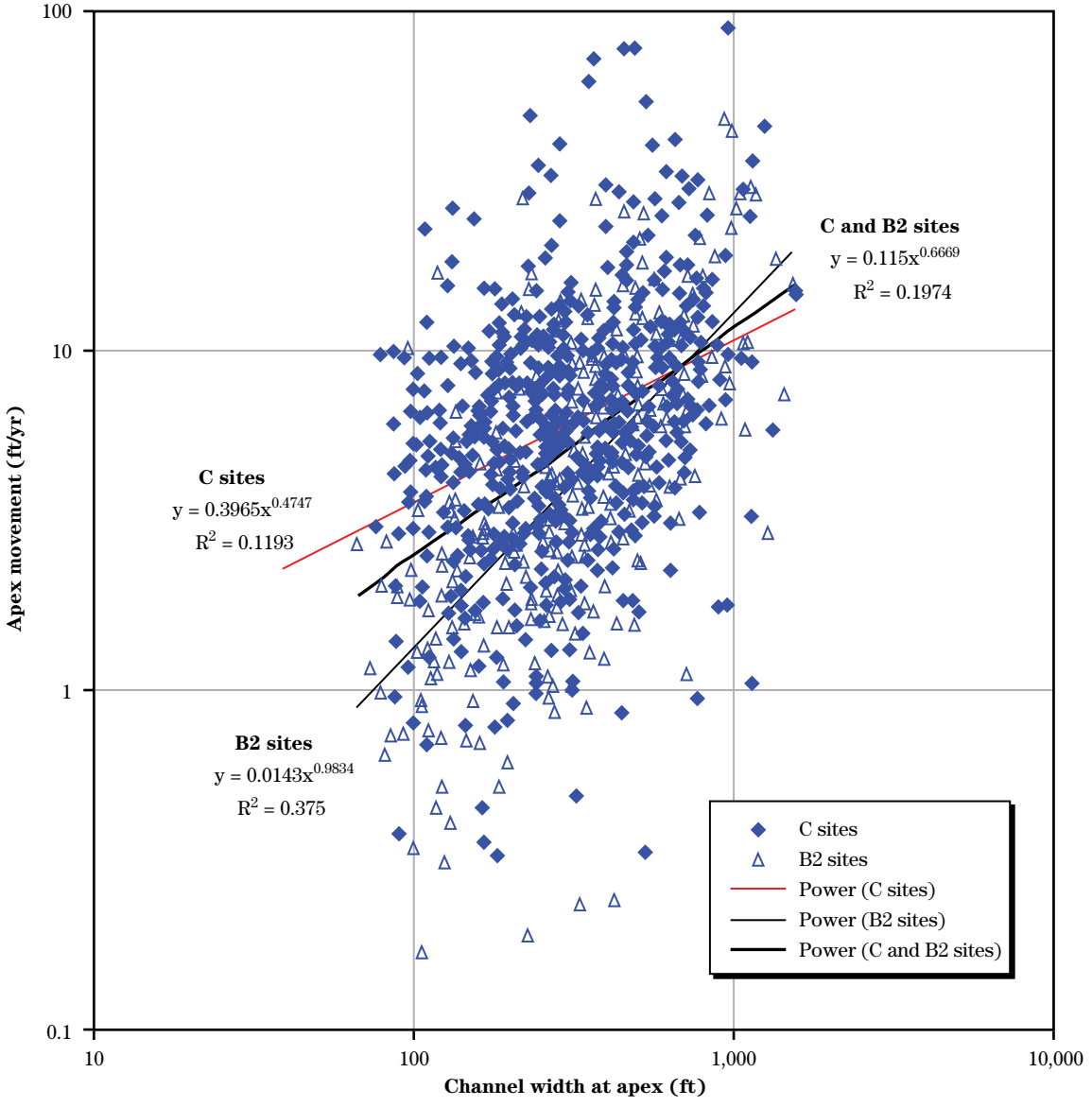
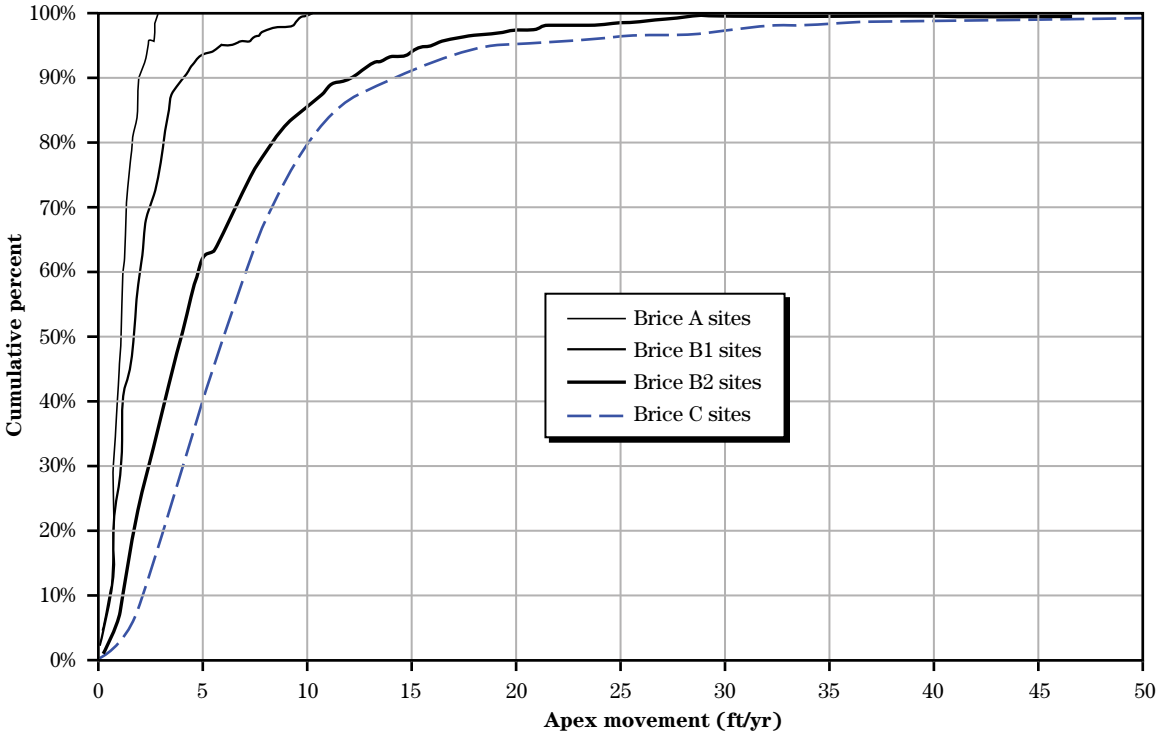


Figure 12–23 Cumulative percentage of apex bend movement



Example problem: Variability in channel planform geometry

Objective: Average channel dimensions for a new meandering sand-bed channel with point bars have been determined. Provide for channel geometry variability so that the new channel will not have excessive adjustments to make as it seeks its new equilibrium condition. Determine the channel width at the bend apex and at the location of the maximum scour. Also, estimate the most probable maximum scour depth and its most probable location. Determine the design depth of the bank protection if it is needed. Estimate the bankline migration rate that might occur if the bank is not protected.

Given: Average channel dimensions at the crossing and the general planform alignment are:

Width = 450 ft
Depth = 25 ft
Channel slope = 0.00030
Valley slope = 0.00049
Meander wavelength = 5,000 ft
Radius of curvature = 2,000 ft
Channel-forming discharge = 50,000 ft^3/s
Mean annual flow = 8,000 ft^3/s

Step 1 Calculate sinuosity

$$P = \frac{\text{Valley slope}}{\text{Channel slope}}$$

$$P = \frac{0.00049}{0.00030} = 1.63$$

Step 2 Calculate the channel distance through one meander wavelength

$$M = P \times \text{ meander wave length}$$

$$M = 1.63 \times 5,000 = 8,150 \text{ ft}$$

Step 3 Calculate the channel width at the bend apex and at the pool using tables 12–5 and 12–6.

This is a type b channel (meandering with point bars)

Width at apex = 1.35 × average channel width

Width at apex = 1.35 × 450 ft

Width at apex = 608 ft

With 90 percent confidence that the width should be between

$$\text{Width at apex} = (1.35 + 0.16)$$

$$\text{and } (1.35 - 0.16) \times 450 = 680 \text{ and } 536 \text{ ft}$$

Width at pool = 1.15 × average channel width

Width at pool = 1.15 × 450 ft

Width at pool = 518 ft

With 90 percent confidence that the width should be between

$$\text{Width at pool} = (1.15 + 0.39)$$

$$\text{and } (1.15 - 0.39) \times 450 = 693 \text{ and } 356 \text{ ft}$$

Step 4 Determine the most probable location of the pool in bend using figure 12–17.

At a cumulative frequency of 50 percent (most probable) the pool-offset ratio is 0.36

$$\frac{Z_{a-p}}{Z_{a-i}} = 0.36$$

The distance to the bend apex from the crossing (inflection point) is one half the channel distance through a meander wavelength.

$$Z_{a-i} = \frac{M}{2} = \frac{8,150}{2} = 4,075 \text{ ft}$$

$$Z_{a-p} = 0.36 \times 4,075 = 1,467 \text{ ft}$$

The location of the pool is then 5,542 feet downstream from the inflection point.

$$4,075 + 1,467 = 5,542 \text{ ft}$$

Step 5 Determine the most probable scour depth and the safe design depth for bank protection using figure 12–18.

$$\frac{\text{Radius of curvature}}{\text{Average width}} = \frac{2,000 \text{ ft}}{450 \text{ ft}} = 4.44$$

$$\frac{\text{Average width}}{\text{Mean depth}} = \frac{450 \text{ ft}}{25 \text{ ft}} = 18$$

Calculate the average depth of scour at the pool.

$$\frac{d_{max}}{d_m} = 2.14 - 0.19 \ln\left(\frac{Rc}{W_i}\right)$$

$$d_{max} = 25\left[2.14 - 0.19 \ln\left(\frac{2{,}000 \text{ ft}}{460 \text{ ft}}\right)\right]$$

$$d_{max} = 46 \text{ ft}$$

Calculate the safe design depth for bank protection.

The mean depth should represent the depth at channel-forming discharge to set an average pool depth for the initial channel geometry, but should represent depth at a design frequency flood peak to set a design depth for bank protection. In this example, a wide flood plain is assumed so that channel-forming and flood depths are similar.

$$\frac{d_{max}}{d_m} = 1.5 + 4.5\left(\frac{Rc}{W_i}\right)^{-1}$$

$$d_{max} = 25 \text{ ft}\left[.5 + 4.5\left(\frac{450 \text{ ft}}{2{,}000 \text{ ft}}\right)\right]$$

$$d_{max} = 63 \text{ ft}$$

Step 6 Estimate bankline migration rate using the Cherry, Wilcock, and Wolman equation (fig. 12–20) and the Ayres Associates graph for a type C channel (fig. 12–21).

$$E = \frac{Q_{ma}^{-0.572} B^{1.83}}{246}, \; r^2 = 0.49$$

$$Q_{ma} = \frac{8{,}000}{35.3} = 227 \text{ m}^3/\text{s}$$

$$B = \frac{450 \text{ ft}}{3.28 \text{ ft/m}} = 137 \text{ m}$$

$$E \text{ m/yr} = \frac{\left(227^{-0.572} \times 137^{1.83}\right)}{246} = 1.5 \text{ m/yr} = 4.9 \text{ ft/yr}$$

$$\text{Apex movement (ft/yr)} = 0.3965 W^{0.4747} \text{ ft}$$

$$\text{Apex movement} = 0.3965(450)^{0.4747}$$

$$\text{Apex movement} = 7.2 \text{ ft/yr}$$

It is not surprising that this analysis indicates such a difference in these estimates, considering the large number of variables that have been ignored. However, this analysis provides an idea of the probable magnitude of the meander migration. It can also be used with additional analysis to assess if bank protection is necessary.

654.1206 Conclusion

In the natural system, there are rarely perfectly linear or straight systems. Natural systems that appear linear typically have some slight sinuosity to them.

Natural channel work requires that the proposed design fits into the natural system within the physical constraints imposed by other project objectives and riparian conditions. Design requires not only channel planform design but also an assessment of natural variability, as well as potential channel movement.

Planform design parameters include sinuosity, meander wavelength, an appropriate channel length for one meander wavelength, and trace of the channel. The approach used to perform this design should be appropriate to the stream conditions. In a threshold channel, planform relates to establishing a maximum slope based on critical stability of the boundary material. In an alluvial channel, planform is a separate dependent variable that must be determined within natural geomorphic limits.

Channel sinuosity is determined from the calculated channel slope and valley slope. To determine other parameters, analogy, hydraulic geometry, and/or analytical methods are employed. To apply the analogy method, a reference reach is located on either the study stream or another suitable stream. From the reference reach a template for the meander planform is developed for the project reach. This may often be problematic due to the nonavailability of a reference reach or subtle, but important fluvial, sedimentary, or morphological differences between it and the study reach.

Alternatively, meander wavelength can be determined using hydraulic geometry techniques. The most reliable hydraulic geometry relationship is wavelength versus width. As with the determination of channel width, preference is given to wavelength predictors from stable reaches of the existing stream either in the project reach or in reference reaches. The channel trace may also be determined analytically using the sine-generated curve. Finally, a string cut to the appropriate length can be laid on a map and fit to existing constraints and to the proper wavelength to form a meandering planform.

The methods used to estimate variability in cross section, as well as potential bank migration, are dependent on site-specific conditions. Some guidance developed for regionally specific studies has been presented. While this material provides a guideline, it should only be used with caution if applied elsewhere.

United States
Department of
Agriculture

**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 13 Sediment Impact Assessments

Issued August 2007

Cover photo: Sediment moved in suspension and as bed load may have impacts on the restoration design. Watershed conditions that produce sediment (either naturally or in excess) should be well understood.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 13 Sediment Impact Assessments

654. 1300 Purpose

Sedimentation analysis is a key aspect of design since many projects fail due to excessive erosion or sediment deposition. A sediment impact assessment is conducted to assess the effect that a full range of natural flows will have on possible significant aggradation or degradation within a project area. This chapter provides a brief overview of several types of sediment impact assessments, along with their rigor and level of uncertainty. The focus of this chapter is primarily on techniques appropriate for the analysis of alluvial channels. However, sediment assessments for threshold channels are also described. There are variants in each of the presented techniques, and more information may be needed to perform the assessments. It is the intent of this chapter to provide an introduction to sediment impact assessments sufficient to select the approach that is most appropriate for most projects. Note that although sediment impact assessment is presented following channel design chapters of this handbook, much of this analysis described should also be done in the sediment assessment phase of the design process that precedes and supports channel design. However, a sediment impact assessment is an important closure loop on any proposed design.

654.1301 Introduction

The success of any restoration that includes channel reconstruction is based on the designed channel's ability to transport the inflowing water and sediment load without excessive sediment deposition or scouring on the channel bed. Therefore, a critical step in any channel design project is a sediment impact assessment. Also, since any bank protection measures may fail if the bed is unstable, an assessment of bed stability is also critical for any bank stabilization project.

Sediment impact assessments can range widely in effort and output. These assessments can be accomplished using visual or qualitative techniques for relatively simple projects or by using a numerical model that incorporates solution of the sediment continuity equation for more complex projects. Several types of sediment impact assessments are described in this chapter. While the focus of this document is primarily on techniques appropriate for the analysis of alluvial channels, threshold channels are also described.

The first step in understanding and implementing a sediment impact assessment is to define the anticipated channel bed response. This is an assessment of bed stability to determine if the channel bed is aggrading, degrading, or is relatively stable. Other aspects of a stability assessment may include bank stability or planform stability. The sediment impact assessment is primarily concerned with the stability of the channel bed.

654.1302 Bed stability

Aggradation and degradation are potential major adjustments of an individual channel or a fluvial system. Since a sediment impact assessment is concerned with predicting these responses, it is important to define what these adjustments are and how they can affect a channel.

(a) Aggrading channel

A channel is considered to be aggrading when long-term sediment deposition occurs on the bed. The channel cross section is filling up or becoming shallower. Channel widening, avulsions, and a reduction in flood capacity are characteristic of an aggrading channel. A channel may experience aggradation due to localized watershed processes such as landsliding or construction activities, or it may be due to natural processes, watershed characteristics, and geology. A constructed channel may aggrade if it is deepened and widened for flood conveyance and does not maintain flows and depths sufficient to transport inflowing sediments under more frequent lower discharges.

(b) Degrading channel

A channel is considered to be degrading when long-term sediment removal occurs from the channel bed. The channel cross section becomes deeper. Bank failure, lowering of water tables, and restriction of a stream's connection to its flood plain can occur in a degrading channel. A channel may experience degradation due to a reduction in sediment supply (as may occur in the stream reach below a dam), an increase in flow (as may occur with development in the watershed), or as a result of a lowering of the base level at the mouth of the reach, triggering headcutting, nickpoints, and degradation. A constructed channel may degrade if bed shear stresses are increased in excess of what the channel boundary was designed to withstand. This can occur due to channel straightening or elimination of flood plain access at high flow.

(c) Stable channel

For the purposes of this chapter, a channel is considered stable (or in dynamic equilibrium) when the prevailing flow and sediment regimes do not lead to long-term aggradation or degradation. A stable channel does not experience changes in its cross-sectional geometry over the medium to long term. Short-term changes in sediment storage, channel shape, and planform are both inevitable and acceptable in natural channels. For example, aggradation or degradation may occur on a streambed over the course of a storm hydrograph, but does not necessarily indicate overall instability. While short-term adjustment may damage bank stabilization or bank habitat structures, these assessments are usually performed in a scour analysis as described in NEH654.14. The focus of the analysis described in this chapter is on long-term, progressive changes.

654.1303 Threshold versus alluvial channels

The choice of the appropriate type of sediment impact assessment depends, in part, on whether the channel at the project location is an alluvial channel or a threshold channel. Therefore, it is important for the practitioner to be able to distinguish between these channel types. In general, the geomorphology of a threshold channel is a product of a process that is no longer at work or not regularly at work. Sediment passes through a threshold channel with very little impact on the channel boundary. In an alluvial channel, there is an exchange of sediment between the channel boundary and the flow. An alluvial channel is more active, and its geomorphology is a product of more frequent events. It is important to note that there is not always a sharp demarcation between threshold and alluvial channels. A channel may behave as a threshold channel under low to moderate flow events, yet behave as an alluvial channel under larger flow events. More information about threshold and alluvial channels is provided in NEH654.09.

A sediment impact assessment is particularly important in alluvial channel design. As described in NEH654.09, stability design for alluvial channels begins by determining the channel dimensions for the channel-forming discharge, using analogy, hydraulic geometry, and/or analytical methods. While a single flow and associated sediment load may have a strong effect on the geomorphology of the stream over the long term, other flows and sediment loads may adversely impact the project. Therefore, once these preliminary dimensions are determined, the next step is to assess how well that channel will maintain sediment continuity for the full range of natural flows. This becomes even more important in cases where the desired channel dimensions cannot be achieved due to project constraints or conflicting project objectives. Alluvial channels typically require more in-depth analyses to assess the potential impacts of sediment, but qualitative techniques can be used in low risk situations.

While the focus of this chapter is primarily on alluvial channels, sediment impact assessment should also be considered for threshold channels. Where the design channel is threshold in nature, the sediment impact assessment may be more qualitative, or it may be integral to the design process itself. For example, the identification of the flow condition that would mobilize the boundary of a threshold channel can be sufficient as a check for potential degradation. In this case, the sediment impact assessment is often referred to as a stability assessment. Many of the approaches for stability assessment of threshold channels are presented in NEH654.08. However, it may also be appropriate to perform a check to assure that any suspended sediment will remain in suspension and not be deposited in the design threshold channel. This analysis can be accomplished by comparing the channel shear velocity to the settling velocity of the sediment, under a variety of expected flow conditions. Finally, the designer should consider possible impacts that may occur if the threshold channel were to transition to an alluvial channel.

654.1304 Types of sediment impact assessments

A variety of techniques may be used to assess the impact of sediment on a project area. The approaches described here are not exhaustive, nor are they applicable in all situations. However, a final sediment impact assessment should be viewed as a closure loop at the end of the design process to:

- validate the efficacy of the design channel geometry
- identify flows which may cause aggradation or degradation over the short term (these changes are inevitable and acceptable in a dynamic channel)
- recommend minor adjustments to the channel design to ensure dynamic stability over the medium to long term

The type of sediment impact assessment used will determine the certainty of the result, as well as the precision of a conclusion that the channel will aggrade, degrade, or remain stable. The selection of the appropriate methodology should be done with a firm understanding of the assumptions, accuracy, data requirements, and limitations of the approach. This chapter outlines some of the most common techniques and offers general guidelines regarding selection criteria. For more details regarding the assumptions and limitations of these methodologies or approaches, the original documentation associated with each should be reviewed. Final decisions regarding the suitability of a particular approach must be determined using engineering judgment on a case-by-case basis.

Most of the following approaches were developed for application with the analysis and design of alluvial channels. However, they can also be used with threshold channels, as well. The following approaches are listed in general increasing level of difficulty.

654.1305 Visual geomorphic assessment

A visual geomorphic assessment is primarily a qualitative check that should be done for both threshold and alluvial streams. This may be the only assessment needed for a potential project, or it may be the first step of a more detailed sediment impact assessment, if required. Visual geomorphic assessments of sediment impacts are generally sufficient where:

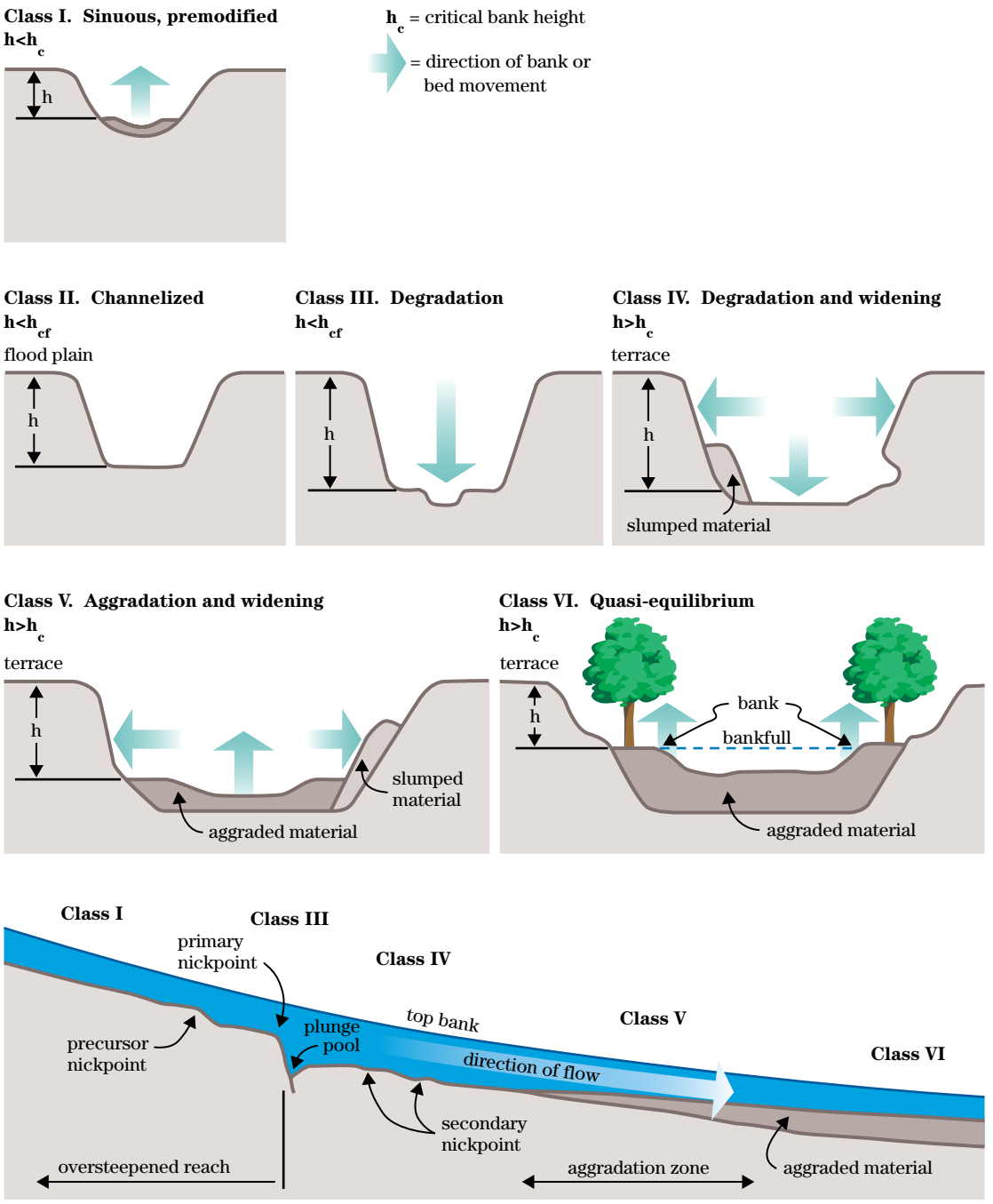
- project failure will have minimal adverse effects
- minimal change to the channel shape is proposed
- the watershed land use and cover and erosion processes are relatively stable

The visual geomorphic assessment includes judgment of current conditions, expected future conditions, and the river's anticipated response to the designed project. It includes the identification of potentially destabilizing processes of erosion, sediment storage, and deposition. A visual assessment can involve the use of channel evolution stage, the use of Lane's stream balance relationship (described in NEH654.1305(c)), and assessments of dominant channel processes. It is critical that experienced personnel conduct this effort. In all cases, the reasoning, judgment, and estimates that support the assessment should be clearly documented and discussed by the stakeholders.

(a) Assessments of channel processes and evolution

The existing shape or morphology of a stream is an indication of ongoing channel evolution processes and has long been recognized as a diagnostic tool in evaluating fluvial landforms. The appropriate channel evolution model can be applied to identify current stream condition, subsequent stages and direction of evolution, and the ultimate expected stable channel form that will evolve, as well as qualitatively estimate the time scale of channel recovery. An assessment of the existing channel evolution stage, as well as the stage that will exist with the proposed project, can be an aid in assessing channel responses. The channel evolution model (CEM) (fig. 13–1 (modified from Simon and Hupp 1986; Simon

Figure 13–1 Six-stage model of channel evolution



1989)) was developed by Schumn, Harvey and Watson (1981, 1984), and modified by Simon and Hupp (1986) and Simon (1989, 1994).

Using space-for-time substitution, the authors developed a conceptual model with reach types that are divided into the following six stages. In a space-for-time substitution, downstream conditions are interpreted as preceding (in time) the immediate location of interest, and upstream conditions are interpreted as following (in time) the immediate location of interest. A reach in the middle of the watershed that previously looked like the channel upstream will, therefore, evolve to look like the channel downstream.

- Stage 1 is a U-shaped channel. It has no sediment storage in the channel as would occur in a newly constructed channel. This stage has also been used to represent a sinuous, premodified, nonincised channel. One of the key features of this stage is the frequent access of the channel flows to the flood plain.
- Stage 2 is a modified or channelized stage. This has also been used to represent the relatively instantaneous change which initiates the following sequence of changes:
  - newly straightened or a steepened slope stage 1
  - reduction in sediment supply
  - increase in discharge
  - lowering of the tailwater
  - advancing headcut
- Stage 3 is a downcutting stage. Rapid degradation is occurring as the channel slope flattens in response to the perturbation imposed on the system in stage 2. A lowering of ground water and undermining of bridge piers may occur in this stage. Stage 3 evolves into stage 4 when the channel bank height exceeds the critical bank height and the banks begin to fail.
- A stage 4 channel is evidenced by a widening channel. The toes of the bank slopes are subject to lateral erosion and undercutting. Usually, both sides of the channel show erosion, not just the outer banks. Stage 4 evolves into stage 5 when the channel widens to a point where it is no longer able to transport the incoming supply of sediment and deposition begins to occur.
- Stage 5 is an aggrading channel. The overwidened channel cannot maintain the velocities necessary to move the sediment that is being supplied from the upper watershed.
- Stage 6 is the quasi-equilibrium stage. The toes of the banks are stabilized with accumulated sediment and vegetation. Alternate bars with perennial vegetation may be evident. Simon (1994) observed that the deposition will likely not be sufficient to return the channel to its preimpacted stage.

These evolutionary stages are linked to rates of sediment transport (Simon 1989), bank stability, sediment accretion, and ecologic recovery (Hupp 1992; Simon and Hupp 1992). The model has been widely used to rapidly identify dominant, systemwide channel processes in watersheds impacted by various human and natural disturbances. Identification of channel process and forms is often accomplished concurrent with the geomorphic assessment and site investigations conducted at the beginning of a project. The CEM was developed from streams responding to straightening and base-level lowering. Specific assessment techniques, including this model, are addressed further in NEH654.03.

While this channel evolution model has been applied in a variety of watersheds throughout the United States, it is most applicable in the Southeast, with its abundant precipitation and deep soils. The use of a channel evolution model may be supported by a study of the watershed and channel history, future land use and development patterns, and appropriate classification of the existing and proposed stream.

(b) Regional hydraulic geometry relationships

Regional hydraulic geometry relationships may also be useful in performing a visual geomorphic assessment. Morphological measurements of width, cross-sectional area, and depth at the project site can be compared to regionally developed relationships or equations and their associated bands of uncertainty. This comparison can provide semiquantitative information on channel stability and sensitivity to change. However, this method only provides an indication of stability, because data points that lie far from the best-fit regression line could be influenced by other factors that are

not common to the rest of the data set such as reach history, land use, or vegetation. Be aware that while observations and hydraulic geometry relations may be used to identify possible stability problems, analytical methods are often required to determine the magnitude of an identified stability problem. More information on the use and limitations of regional hydraulic geometry is provided in NEH654.03 and NEH654.09.

(c) Lane's alluvial channel balance relationship

Lane's balance or Lane's relationship is a qualitative conceptual model that can be used as an aid to visually assess stream responses to changes in flow, slope, and sediment. The model is based on the general theory that if force applied by the flowing water on an alluvial channel boundary is balanced with strength of the channel boundary and the delivered sediment load, the channel will be stable and neither aggrade nor degrade. This equilibrium condition in the channel can be expressed as a balance of four basic factors (Lane 1955b):

- sediment discharge, Q_s
- median grain size of bed material, D_{50}
- dominant discharge or streamflow, Q_w
- thalweg slope or energy slope, S

This balance can be expressed in the proportional relationship (eq. 13–1) or figuratively (fig. 13–2).

$$(Q_s)(D_{50})\alpha(Q_w)(S) \qquad \text{(eq. 13–1)}$$

Lane's relationship suggests that a stream will remain in equilibrium as along as these four variables are kept in balance. If one variable changes significantly, the stream will respond by aggrading or degrading, and another variable must adjust to restore balance. For example, a decrease in discharge could result in aggradation (as may occur downstream of a flood control dam or due to flow diversion). In contrast, a straightening of a stable channel (which would increase slope) may result in degradation. The increased slope of a straightened channel creates a disequilibrium condition where an increased sediment supply or a larger particle size is needed. Therefore, erosion of the streambed and streambanks will return the reach to an equilibrium condition. Since sediment yield varies over a long time in establishing the equilibrium condition, Lane's (1955a) conceptual relationship fits the concept of dynamic equilibrium established by Schumm (1977) and is, therefore, applicable to most streams and rivers.

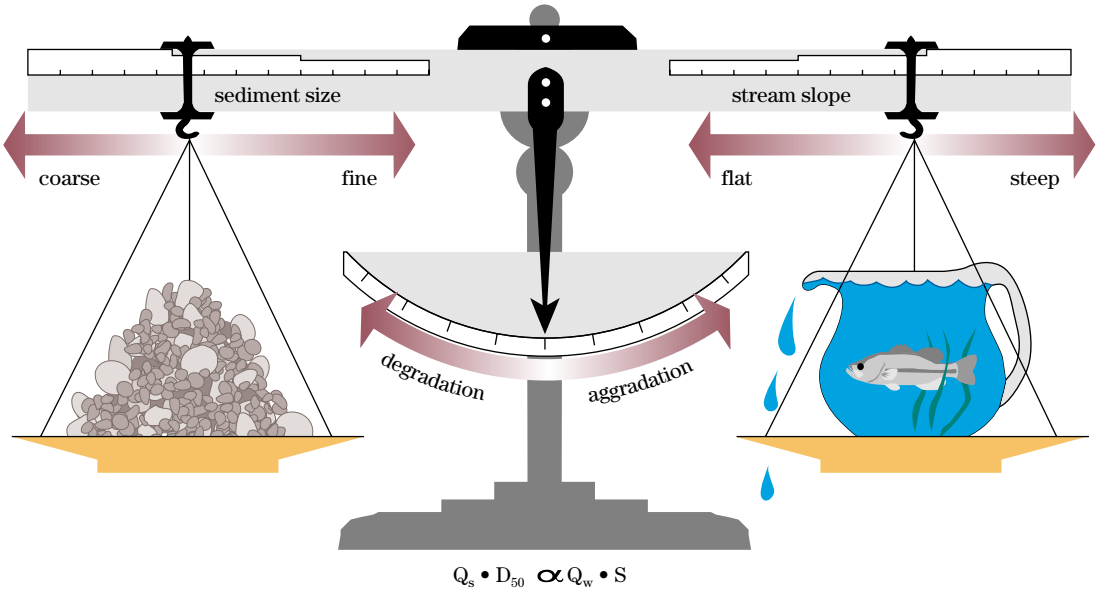
A limitation of this conceptual model is that it does not indicate which variable will adjust, the magnitude of the adjustment, or the timeframe that will be involved. While it may be used to identify possible stability problems, analytical methods are often required to predict, in quantifiable terms, their magnitude. In addition, even while in balance, the stream is free to migrate laterally, maintaining its cross-sectional area. This lateral movement may be unacceptable due to land use or boundary constraints. More detail on Lane's relationship, as well as other qualitative relationships is provided in NEH654.03 and also available in Stream Corridor Restoration: Principles, Processes and Practices (FISRWG 1998).

(d) Assessments of dominant channel processes

Dominant channel processes are the forces at work in the watershed that cause and limit channel change. They are the causal factors, direct and indirect, and controls likely to be present in the study watershed and at a study site. An understanding of these dominant channel forces or processes can assist the designer with the prediction of the proposed project's impact on channel morphology, ecology, and stability. The assessment and evaluation of dominant channel and watershed processes is often accomplished early in the planning and design stages as part of data collection. NEH654.03 addresses this in detail. However, the assessment of dominant processes should be revisited as the project is finalized, to ensure that the design fits the context of the watershed and is consistent with the sediment impact assessment.

Of particular interest should be the characterization of sediment sources based on their relative contribution to the project reach's bed load, suspended load, and wash load. The with-project conditions should be assessed within the context of this overall sediment balance. The designer should focus on significant sediment sources and sinks within the study reach and how they may be affected by the proposed project. The broad elements that should be examined are:

Figure 13–2 Lane's balance as represented in Federal Interagency Stream Restoration Working Group (FISRWG) (1998)



From Rosgen (1996), from Lane, Proceedings, 1955.
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- spatial and temporal patterns of watershed sediment production
- sediment storage within the channel and in adjacent tributaries
- patterns and behavior of sediment movement through the system
- rates of sediment transport
- sediment deposition rates on the flood plain
- changes in sediment load due to changes in watershed land use

Much of the assessment of dominant processes can be accomplished by an examination and review of geological information, local historical accounts, historical thalweg and cross-sectional information, gage data, Federal Emergency Management Agency (FEMA) maps, biological monitoring, hydrologic modeling, and watershed development and land use patterns. Aerial photographs, maps, and old reports can also be useful in this assessment. Recent gage data can be analyzed and reviewed to determine if current conditions might be the result of a recent extreme event, rather than long-term and systemic instabilities.

Historical analysis can provide meaningful information. Well-documented stream history may provide a reasonably accurate assessment of future stream trends: will it aggrade or degrade? Historical data can be used to identify trends, provide information on rates of landform change in the watershed, and help determine land use impacts on current conditions. These effects can be due to watershed development that has altered streamflows, stream morphology, and sediment yields. Effects could have occurred gradually over a long period of time, such as changes in land use, population, or agricultural crops and farming practices. Streams in these watersheds may be adjusting naturally to an aggraded condition by slowly downcutting. Landslides and gravel nourishment, as well as gravel mining activities, can also have short-term, but profound impacts on reach dynamics and project performance. Finally, geologic aspects of the watershed should be considered. For example, as streams and rivers migrate laterally within their valleys in glaciated regions, they can encounter glacial till and coarse-grained glacial outwash, altering sediment loads and sediment particle sizes. A slug of sediment that enters the stream and moves downstream in pulses during high runoff is also common along streams where sediment load is dominated by landslides and debris flow torrents.

Onsite field assessments are needed to augment analysis and existing information sources. Observe conditions in tributaries and abandoned channels in the project reach, and identify indications of channel behavior and geomorphic conditions. Anthropogenic features, such as bridge abutments and piers, grade control structures, low-flow crossings, and bank protection can also provide an indication of possible channel responses to the project. Finally, determine whether the channel bed is aggrading, degrading, or stable.

Evidence of degradation will be different, depending on the project's location within a watershed, whether it is in the upland, middle, or lowland zone. Some field indicators of river stability/instability are given in table 13–1 (modified after Sear and Newson 1994) for each of these zones in a watershed. These are not absolutes, and exceptions and additions will be encountered.

While an assessment of the dominant processes and the application of engineering judgment are valuable and necessary for any design, the limitations of what is essentially a qualitative approach must be recognized. Issues that should be considered in weighing the impact of these assessments include observer experience and bias, temporal limitations, and spatial limitations. Issues related to observer bias can be partially overcome with the consistent use of trained personnel and consistent inventory procedures. This will minimize relative differences between observations. Temporal bias can be minimized with an examination of historical records, but these may be incomplete. While an assessment of the dominant processes may be used to identify possible stability problems, analytical methods are often required to determine the magnitude and direction of change in the instability.

Table 13–1 Field indicators of river stability/instability

	Location within watershed		
Condition	**Upland**	**Middle**	**Lowland**

Degradation	• Perched boulder berms
	• Terraces
	• Old channels
	• Old slope failures
	• Exposed pipe crossings
	• Suspended culvert outfalls and ditches
	• Undercut bridge piers
	• Exposed or 'air' tree roots
	• Leaning trees
	• Narrow/deep channel
	• Bank failures, both banks armored/ compacted bed
	• Deep gravel exposure in banks that are topped with fines
		• Terraces
		• Old channels
		• Exposed pipe crossings
		• Suspended culvert outfalls and ditches
		• Undercut bridge piers
		• Exposed or 'air' tree roots
		• Leaning trees
		• Bank failures, both banks
		• Vertical banks
		• Compacted/compacted bed
		• Deep gravel exposure in banks that are topped with fines
		  • Undercut stone and concrete walls
		  • Abandoned streambeds that appear to be a deposition bar
			• Old channels
			• Exposed pipe crossings
			• Suspended culvert outfalls and ditches
			• Undercut bridge piers
			• Exposed or 'air' tree roots
			• Leaning trees
			• Narrow/deep channel
			• Vertical banks
			• Bank failures, both banks
			• Deep gravel exposure in banks that are topped with fines
			• Undercut stone and concrete walls
			• Abandoned streambeds that appear to be a deposition bar

Aggradation	• Buried structures such as culverts and outfalls • Buried soils • Large uncompacted point bars • Eroding banks at shallows • Contracting or reduced bridge space • Deep, fine sediment over coarse gravels in bank • Many unvegetated point bars • Outlet of tributaries buried in sediment • Rills or remnant channels in riparian areas	• Buried structures such as culverts and outfalls • Buried soils • Large uncompacted point bars • Eroding banks at shallows • Contracting or reduced bridge space • Deep, fine sediment over coarse gravels in bank • Many unvegetated point bars • Angular bed material in an environment where rounded is expected • Outlet of tributaries buried in sediment • Rills or remnant channels in riparian areas	• Buried structures such as culverts and outfalls • Buried soils • Large silt/clay banks • Eroding banks at shallows • Contracting or reduced bridge space • Deep, fine sediment over coarse gravels in bank • Many unvegetated point bars • Angular bed material in an environment where rounded is expected • Outlet of tributaries buried in sediment • Rills or remnant channels in riparian areas
Stability	• Vegetated bars and banks • Compacted weed covered bed • Bank erosion rare • Old structures in position • Armoring of sediment • Older culverts and outfalls exiting at or near grade • Mouth of tributaries at or near existing main stem stream grade • Vegetated banks • Roots of large trees anchored in soil • Evidence of frequent overbank flows • Algae growth on substrate	• Vegetated bars and banks • Compacted weed covered bed • Bank erosion rare • Old structures in position • Older culverts and outfalls exiting at or near grade • Armoring of sediment • Mouth of tributaries at or near existing main stem stream grade • Vegetated banks • Roots of large trees anchored in soil • Evidence of frequent overbank flows • Algae growth on substrate	• Vegetated bars and banks • Weed covered bed • Bank erosion rare • Old structures in position • Older culverts and outfalls exiting at or near grade • Armoring of sediment • Mouth of tributaries at or near existing main stem stream grade • Vegetated banks • Roots of large trees anchored in soil • Evidence of frequent overbank flows • Algae growth on substrate

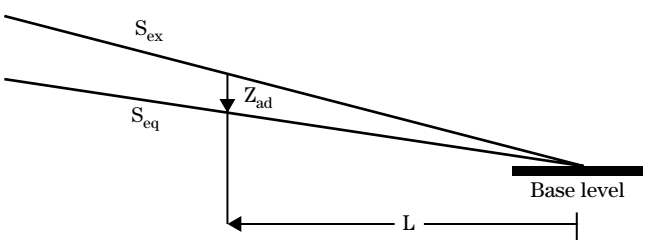
654.1306 Equilibrium slope calculations

Equilibrium or stable slope calculations are often used to support or refine visual assessments. The calculation of a stable or equilibrium slope may also serve as a form of sediment impact assessment, as well as being an integral part of the restoration design.

The equilibrium slope of a channel is defined as the slope at which the sediment transport capacity of the reach is in balance with the sediment transported into it. If the sediment transport capacity were to exceed the sediment supply, channel bed degradation will occur until the channel bed slope is reduced to the extent that the boundary shear stress is less than what is needed to mobilize the bed material. This new, lower slope is the equilibrium slope, S_{eq}. Possible causes of the sediment transport capacity exceeding sediment supply could include an upstream reduction in sediment yield (such as in a stream reach below a dam), an increase in sediment transport capacity during high discharges, or construction of a straight channel, resulting in increased stream gradient. This lowered, degraded bed may result in undermining or collapse of riparian structures or bank instability.

Equilibrium slope calculations are typically used for threshold streams. In the context of a sediment impact assessment, they are applied to a range of design flows. As illustrated in figure 13–3, slope adjustment in a threshold reach occurs by degradation proceeding from the upstream end to the downstream, and the downstream extent of degradation is often limited by a base level control. The z_{ad} is often referred to as the general scour depth.

Figure 13–3 Definition of equilibrium slope, S_{eq}. Relationship between existing slope, S_{ex}, equilibrium slope and the potential bed reduction, z_{ad}, for a reach of length L with base-level control



A variety of techniques can be used to calculate the limiting or equilibrium slope. One approach that is suitable for gravel-bed streams is the Meyer-Peter and Müller bed load transport equation, rearranged as follows:

$$S_L = \frac{K_1 \times D_{50} \times \left(\frac{n}{D_{90}^{\frac{1}{6}}} \right)^{\frac{3}{2}}}{d} \qquad \text{(eq. 13–2)}$$

where:
S_L = limiting slope
n = Manning's n
K_1 = conversion constant
D_S = particle size
d = flow depth

Similar equations, based on range in sediment particle size application, should be applied for other channel types. Note that the calculated equilibrium bed slope may be limited by resistant layers in the bed (such as bedrock) or by the formation of an armor layer. The overall depth of scour required to leave a stable armor layer can be assessed with the following equation:

$$\Delta Z = \frac{2 \times D_a}{P_c} \qquad \text{(eq. 13–3)}$$

where:
ΔZ = scour depth
D_a = size of armoring material (threshold grain size for incipient motion)
P_c = percent of material coarser than armoring size

The threshold particle size for incipient motion, the largest particle that can be lifted and transported by the flow, can be calculated as follows:

$$D_a = \frac{\tau_c}{0.047(\gamma_s - \gamma)} \qquad \text{(eq. 13–4)}$$

where:
D_a = particle size
τ_c = grain resistance boundary shear stress ($\frac{1}{8} f \rho \upsilon^2$)
γ_s = 165.4 lb/ft^3
f = friction factor ($= 8 \frac{g}{C^2}$)
C = $\frac{1.49}{n} R^{\frac{1}{6}}$
R = hydraulic radius
ρ = 1. 94 slugs
υ = velocity (designer should account for bends)
n = near-field Manning's n (0. 025)

The assessment of this potential degradation for different flow levels is often used to determine the appropriate spacing and size of grade control structures. Further information about these analytical techniques and equations is provided in NEH654.08 and in NEH654 TS14C.

654.1307 Sediment rating curve analysis

The sediment rating curve analysis is a relatively simple technique that can be used to assess the sediment transport characteristics of an existing or proposed stream project. The approach is to use sediment rating curves to compare the sediment transport capacity of the supply reach to the existing and proposed project reach conditions. This approach relies on the technique of analogy. If the existing channel is stable, then sediment transport capacity in the project channel may be compared to that in the existing channel. If the supply reach is not fully alluvial, a carefully chosen reference reach may be used as a surrogate for the supply reach. This analysis is suitable for streams where the sediment supply is not limited in either the upstream (supply) or project reaches; that is, where the stream is certainly alluvial in nature. It is generally not suitable for threshold streams.

This qualitative technique does not require stream gage data or sediment gage data. It does require an estimate of the sediment grain size distribution from the supply reach, an estimated range of peak flows, and a description of hydraulic characteristics of both the study and supply reaches. By comparing the sediment rating curves of the two reaches, an estimate can be made of the sediment transport capacity of the study reach, relative to the capacity of the sediment supply reach. The basic steps are:

Step 1 Collect hydraulic information for the upstream, existing, and proposed project reaches. Hydraulic information can come from normal depth calculations, hydraulic modeling (such as the U.S. Army Corps of Engineers (USACE) HEC–RAS) based on new surveys, or with the use of existing flood plain information, such FEMA's flood plain maps.

Step 2 Collect sediment gradation for upstream, existing, and proposed project reaches. Guidance for sediment sampling is provided in NEH654 TS13A.

Step 3 Estimate a range of peak flows for the project reach. Peak flows can be estimated using regional regression curves or hydrologic modeling.

Step 4 Calculate sediment transport capacity for the range of peak flows in the upstream, existing, and proposed reaches. Information useful for the selection of appropriate sediment transport relationships is provided in NEH654.09.

Step 5 Create a sediment rating curve for the upstream, existing, and proposed reaches.

Step 6 Compare the sediment rating curves for these conditions to assess project performance.

By comparing the sediment rating curves of the two reaches, an estimate can be made of the sediment transport capacity, relative to the capacity of the sediment supply (fig. 13–4).

The comparison of the two sediment rating curves shown in figure 13–4 indicates that there is a strong possibility that the existing study reach is depositional for flows above Q_1. The proposed project conditions can be assessed in a similar manner as illustrated in figure 13–5.

A comparison of the two sediment rating curves in figure 13–5 indicates that the project reach should be able to transport the incoming sediment load through a discharge of Q_2. Above this discharge, deposition is possible, for example, at Q_3. These discharges can be compared to the peak discharges of estimated storm frequencies to provide a qualitative estimate of project life. This estimated condition should be checked by field observations to detect evidence of an aggradational trend, as well as the assessment of dominant channel processes. To improve channel stability, the sediment rating curve for the project channel should be as close as possible to the sediment rating curve for the supply reach.

Since there is no calibration of gage data or use of flow-duration data, the actual quantity of sediment deposition cannot be estimated. In addition, this approach does not account for changes in sediment transport capacity that may occur as sediment is deposited in the section and changes its geometry. However, this technique does provide the designer with a qualitative appraisal of anticipated project performance.

Figure 13–4 Sediment rating curve analysis for existing conditions

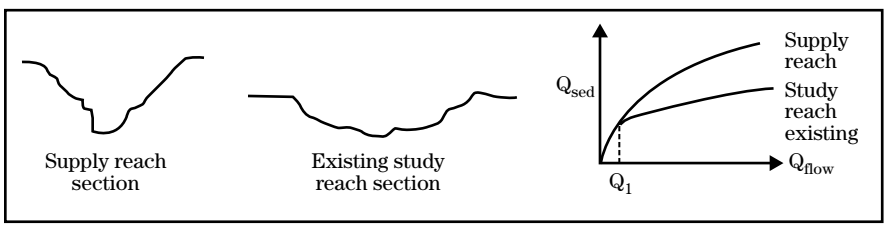
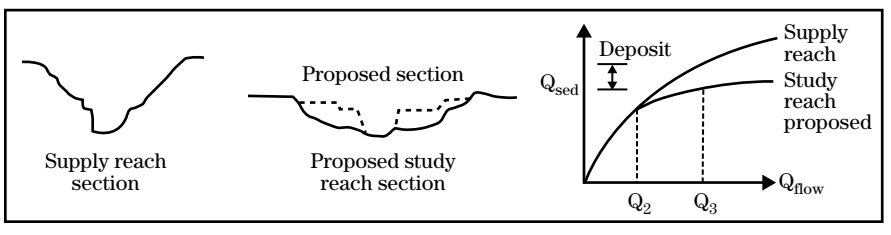


Figure 13–5 Sediment rating curve analysis for proposed conditions



654.1308 Sediment budget analysis

A sediment budget analysis is a quantitative assessment of channel stability using the magnitude and frequency of all sediment-transporting flows. A sediment budget analysis should be conducted for all realigned and constructed alluvial channels, after preliminary dimensions are determined, using the channel-forming discharge. Slight adjustments to the design may be required, after which another sediment budget analysis is conducted.

The stream's sediment budget is estimated by comparing the mean annual sediment load for the project channel with that of the supply reach(es). The mean annual sediment load from each reach is calculated by numerically integrating the annual flow-duration curve with a bed-material sediment rating curve. While the sediment load is typically calculated for annual conditions, it may also be assessed for a flow event of interest, depending on project conditions and purposes. If more sediment comes into the project area than can be passed, the excess will likely be deposited in the reach. If more sediment can be transported than what is coming into the reach, then erosion or degradation can be anticipated.

The following steps are recommended for conducting a sediment budget analysis.

Step 1 Assemble information about the stream. Collect data from the supply reach(es) upstream, the project reach, and downstream from the project reach. This includes geometric, sediment, and hydrologic information. Much of this information may have been collected during initial assessments and data collection. It may be necessary to construct flow-duration curves from 15-minute data (rather than daily) in areas where a large amount of sediment transport can occur during storms of duration much less than 24 hours. All sediment sources should be quantified, especially nonalluvial sources such as mass failures, landslides, debris flows, and soil creep. Additionally, the rates and volumes of sediment stored in the landscape should be estimated including in the channel, in wetlands, in lakes and ponds, on the flood plain, and on alluvial fans.

Step 2 Calculate hydraulic parameters for a typical or average reach for a range of discharges. This range should extend from the average annual low flow to the peak of the design flood. Average hydraulic parameters can be determined from normal depth calculations for a typical cross-sectional geometry, or from a backwater computer program such as HEC–RAS.

Step 3 Select an appropriate sediment transport function for the study reach. This can be achieved by comparing calculated sediment transport to measured data, taking care to ensure that bed-material load is being compared. When no data are available, one may rely on experience with similar streams in the region. Data ranges used in the development of various sediment transport functions are provided in NEH654.09. A review of this information may serve as guidance in selecting the appropriate function. However, if there are no available data for calibration, this analysis becomes more qualitative in nature.

Step 4 Calculate sediment transport rating curves. Apply calculated hydraulic parameters to the selected sediment transport functions for a range of flows. Curves should be developed for the existing channel in the assessment reach, upstream of the assessment reach (the supply reach), and downstream. Sediment transport rating curves should also be determined for any tributaries that might be affected by the assessment reach.

Step 5 Calculate sediment yield. Sediment yield should be calculated using the flow-duration sediment discharge rating curve method for the supply reach, assessment reach, and downstream reach. Use a flow-duration curve to obtain average annual sediment yield and a flood hydrograph to obtain sediment yield during a flood event. The calculation of average annual sediment yield is typically accomplished with the flow-duration sediment discharge method (USACE 1995a). This method requires sufficient gage data to develop the flow-duration curve and requires either measured bed-material load data or calculation of a sediment discharge rating curve, using an appropriate sediment transport relationship.

Often, sufficient gage data are not available to calculate a flow-duration curve for the project reach. If so, two approaches can be used to com-

pute average annual sediment yield. The first is to synthesize a flow-duration curve using either the drainage area flow-duration curve method or the regionalized duration method (Biedenharn et al. 2000). Then use standard methods to compute sediment yield.

If information is available for calibration, this technique can be used to estimate the actual quantity of deposition. Even without calibration information, this technique can provide relative comparisons of stability for various alternatives. Note that this approach typically uses average reach conditions. It does not account for changes in sediment transport capacity that may occur as sediment is deposited in the section and changes its geometry. The level of confidence that can be assigned to the sediment budget approach is a function of the reliability of the available data about the stream and the project. Specific techniques are addressed in more detail in NEH654 TS13A and TS13B, Thomas et al. (1994), and EM 1110–2–4000 (USACE 1995a).

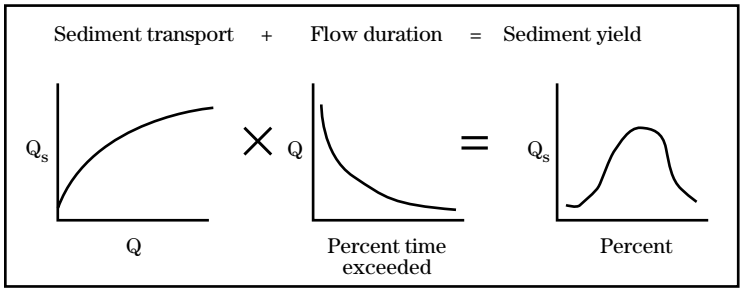
Step 6 Calculate trap efficiency by comparing the supply reach and assessment reach sediment yields. A positive trap efficiency indicates deposition and a negative value indicates erosion. If the assessment reach is stable, the trap efficiency is near zero.

An example sediment budget analysis conducted as part of the reconnaissance level planning study for a flood damage reduction project is provided in NEH654 TS13B.

654.1309 Computer models

Sediment budget analysis is typically accomplished using a computer program such as the USACE SAM or HEC–RAS program. However, a sediment budget can be analyzed with a spreadsheet program, as well. Where bed-material sediment transport is significant and highly variable, it may be necessary to use a numerical model that incorporates solution of the sediment continuity equation. Most computer models involve integrating a sediment transport function to a flow-duration relationship to estimate sediment yield —either by event, annually, or for multiple years (fig. 13–6).

Figure 13–6 Sediment budget



654.1310 Nonequilibrium sediment transport

A sediment impact assessment should include a nonequilibrium sediment transport model for high risk or high cost projects. River systems are governed by complicated dependency relationships, where changing one significant geometric feature or boundary condition affects other geometric features and flow characteristics, both temporally and spatially. Changes at any given location in a stream system are directly related to the inflow of sediment from upstream.

HEC–6 (USACE 1993c) is a one-dimensional, moveable boundary, open channel flow numerical model designed to simulate and predict changes in river profiles resulting from scour and deposition over moderate time periods (typically years, although applications to single flood events are possible). This model simulates the sediment transport capacity of a reach by mathematically modeling the interaction between the sediment inflow and the hydraulic properties of the reach. In this model, a continuous discharge record is partitioned into a series of steady flows of variable discharge and duration. For each discharge, a water surface profile is calculated, providing energy slope, velocity, depth, and other variables at each cross section. Potential sediment transport rates are then computed at each section. These rates, combined with the duration of the flow, permit a volumetric accounting of sediment within each reach. The amount of scour or deposition at each section is then computed, and the cross-sectional geometry is adjusted for the changing sediment volume. Computations then proceed to the next flow in the sequence, and the cycle is repeated using the updated cross-sectional geometry. Sediment calculations are performed by grain-size fractions, allowing the simulation of hydraulic sorting and armoring.

HEC–6 is a powerful tool that allows the designer to estimate long-term response of the channel to a predicted series of water and sediment inflows. The use of a complex program such as HEC–6 involves a significant investment of engineering skill and time. The time required to perform a HEC–6 analysis can be upwards of 10 times that of the USACE SAM type analysis (Fripp, Webb, and Bhamidipaty 1996). However, it is often more advantageous to invest in this effort than to deal with the consequences of project failure. The critical decision to use a numerical model should be based on whether significant changes are expected to occur in the system as a result of the proposed design work.

The primary limitation of HEC–6 is that it is one-dimensional; that is, geometry is adjusted only in the vertical direction, and average hydraulic parameters are assumed in the computations. Changes in channel width or planform cannot be simulated. This analysis is typically based on one-dimensional, steady-flow models, while natural flows are three-dimensional and unsteady. In most cases, the three-dimensional effects of meander bends are accounted for with empirical geomorphic approaches and professional judgment (Copeland et al. 2001). For more complete information on details regarding the assumptions and limitations of specific models, the original documentation associated with each of them should be reviewed.

Finally, while a computer model such as USACE SAM or HEC–6 might provide a more precise answer, there is no reason to suppose that it gives a more certain answer. Computed answers might be highly precise, but are tied to original assumptions, which may not be accurate. Complicated models do not necessarily provide more accurate answers by themselves. If too little information is available as input to the models and no verification data are collected, it is unlikely that a detailed model will provide a more accurate answer than a simpler model. In all cases, field measurements and local experience should be used to complement the use of computer models.

654.1311 Choosing the appropriate technique

The choice of the appropriate technique to estimate the sediment impact of a proposed project includes not only an assessment of the project goals and watershed condition, but also the potential impacts of project failure. Visual and qualitative assessments are appropriate for sites where there is low risk and minimal change to an otherwise stable system. These can be accomplished with the aid of primarily judgment-based tools. As a project becomes more complex, and where there is a higher risk to life and property, more analytical approaches are used. Many analytical techniques are available that typically require the calculation of hydraulic parameters for the range of natural discharges, such as velocity and shear stress. All of these techniques require data determined from field observations and measurements, as well as calculations. Table 13–2 illustrates typical assessment techniques for estimating the impacts of sediment on different project types and watershed conditions.

As the risk and uncertainty increase, the use of more detailed models is recommended. Table 13–2 shows increasing complexity, from Lane's stream balance approach, to USACE SAM, to HEC–6. However, the use of increasingly complicated models is not necessarily recommended. On its own, a more complicated analysis will not necessarily be sufficient or more accurate. Any model is dependent on the skill and experience of the practitioner, as well as the input data. Engineering judgment becomes more critical with increasing risk, and the required field work and data collection become more labor intensive. Therefore, the suitable assessment column should be regarded as a cumulative recommendation that increases with increasing risk.

Since each stream system and project is unique, practitioners should review the assumptions and data requirements and consider their own experiences when determining the appropriate technique to use.

Table 13–2 Selection guidance for sediment impact assessment technique

Project type	Site/watershed assessment	Risk to life, property, or project investment	Suitable sediment impact assessment
Bank stabilization No significant change to cross section, slope, or planform	Relatively stable watershed and site	Low	Confirm that there is no significant change in the local hydraulic conditions from pre- to post project and note watershed stability
Bank stabilization No significant change to cross section, slope, or planform	Moderately active watershed and site	Moderate	Assess stable channel grade at design flows. Field check indications of future channel evolutionary change
Bank stabilization No significant change to cross section, slope, or planform	Moderately active watershed and site	High	Rating curve comparison of above and through site
Channel modification Small change to cross section, slope, or planform	Moderately active watershed and site	Low	Rating curve comparison of above and through site, as well as pre- and post project
Channel modification Significant change to cross section, slope, or planform	Moderately active watershed and site	Moderate	Sediment budget analysis with USACE SAM* type analysis
Channel modification Significant change to cross section, slope, or planform	Active watershed and site	High	Long-term numerical modeling with HEC–6* type analysis

* SAM and HEC–6 are now incorporated into HEC–RAS.

654.1312 Conclusion

It is strongly recommended that a sediment budget analysis be conducted for all projects that will involve a significant change to the existing stream channel. Sediment impact assessments can range widely in effort and output, but assess the stability of the project based on conditions of flow, coupled with sediment yield and transport. Visual or qualitative techniques may be used for relatively simple projects, analytical techniques for more complex projects. While no model or assessment eliminates all possibility of a project not performing as intended, the use of the appropriate tool as described in this chapter reduces the possibility of poor project performance.

United States
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**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 14 Treatment Technique Design

Issued August 2007

Cover photos: *Top*—Treatment techniques for streambank stabilization and stream restoration require specific design tools. Management and removal of disturbance factors should be balanced with structural approaches.

Bottom—Treatments range from simple to complex. Design tools assist the user in properly installing a treatment.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 14 Treatment Technique Design

654.1400 Purpose

Stream design and restoration often include specific treatments in the riparian area, on the bank, and in the bed of a stream. Treatments can include techniques that provide ecological enhancement, as well as protection of these areas. This chapter provides an overview of some of the frequently used treatment techniques for bank protection, grade protection, and habitat enhancement using a wide range of plant materials, earth materials, and other inert materials. In addition, analysis techniques that are needed for successful designs are provided. This chapter contains a brief overview of each analysis approach or treatment technique. Refer to the section in the listing of technical supplements for performance criteria, specific analysis, and design guidelines for each technique. Where information is available, the benefits, flexibility, risk, and cost of each technique are presented from a physical, as well as an ecological perspective.

The reader should not interpret the listed techniques as an endorsement of any particular product mentioned and should not infer that one treatment or approach is superior to another. The list of approaches is not exhaustive. There are other techniques, as well as variants of each of those described, that may be appropriate and applicable. Finally, while this chapter provides techniques that focus on the treatment of local problems, the use of several of these techniques, as well as other design elements, often can provide a more holistic approach to complex restoration projects.

654.1401 Introduction

A wide variety of analysis techniques can be applied to channel design and stream restoration. The selection and design of the different techniques depends upon the project goals, watershed conditions, and consequences of failure. All techniques contain some inherent flexibility and inherent risk. The tolerance for risk by the landowner and the public must be considered as the designer selects not only the technique to use but also the level of design analysis to apply. Finally, a selection of an appropriate treatment technique and level of analysis must consider cost. Cost effectiveness includes both the initial project costs, as well as operation, maintenance, and replacement costs. Much of the information presented in NEH654.02 and NEH654.04 should be reviewed and be included in these important decisions.

The design and restoration of a stream often requires the application of a combination of technologies. Techniques that are part of a traditional engineering approach can be altered or enhanced to provide habitat benefits. Many of the treatment techniques described herein are used in conjunction with other techniques to achieve project goals. For example, systems composed of living plant materials are often used in association with inert materials such as wood or rock, as well as manufactured products. In addition, the use of several design analysis techniques may be required for the successful application of a single treatment technique. Information on the reach and watershed that was assessed and calculated, as described in earlier chapters, may provide the required input for the designs and assessments.

Many of the treatment techniques described have been implemented by themselves to address small, local issues. This approach has sometimes been unjustly referred to as applying a band-aid solution. However, the band-aid approach may be completely justifiable in a scenario where there is only localized instability. It only becomes a band-aid when there is an attempt to address systemwide instability with a localized solution.

Some of the techniques described are sequential. For example, the installation of habitat features on an unstable stream must be done after the stream has been stabilized. Techniques such as the channel evolution model, addressed in NEH654.03 and NEH654.13, may be useful in making this assessment.

Some of the treatments described in this chapter should be implemented concurrently. For example, while it is often simpler to plant vegetation into a conventional bank protection project after construction, better results are achieved if the vegetation is incorporated directly into the treatment during construction. To adequately do so, provisions for vegetating should be addressed during the planning and design stages of the project.

654.1402 Design analysis

Design analysis, using sound physical principles and well-established engineering formulae, are used in the implementation of both soft and hard treatments. This section contains some of the techniques that have broad applicability to many treatment approaches described in this chapter.

The level of design analysis needed to employ these treatment techniques depends on both the treatment technique employed, as well as site conditions. The level of analysis should also match the cost of the project under consideration and level of risk associated with the project.

(a) Do Nothing option

The Do Nothing option is also sometimes referred to as the No Action alternative. This option is placed as the first entry under the design analysis section of this chapter to emphasize the importance of this consideration. It is covered briefly, but it is an important analysis. While it may seem self-evident that the planners and designers have discarded the Do Nothing approach if treatment options are being investigated, it is strongly suggested that this decision be continually revisited. This is also known as the Future, Without-action alternative, since the primary objective is to describe not only the problems as they exist today but also to predict a direction or magnitude of change in conditions. Natural stabilization may be occurring, but not quick enough to satisfy goals and objectives. Conversely, problems may be accelerating or affecting more area in the future, which brings the need for development of other restoration alternatives into focus.

Any treatment approach carries with it some level of both known and potential impact. These impacts can be both ecological and physical. Impacts that should be considered include:

- how the treatment interacts with the local environment
- how the treatment may alter, accelerate, or limit natural processes on a reach or watershed scale

- how the treatment may affect the social dynamics on a local or watershed scale
- alteration to the natural environment that is required for the construction of the treatment
- aesthetics—how the treatment interacts with the visual scene
- scale of impact on a temporal basis—is the cost of treatment justified based on sustainability of impact over time

These potential impacts should be weighed against the intended benefits of the treatment. These assessments often require a strong and well-coordinated interdisciplinary approach.

The Do Nothing option should constantly remain as a possibility. The resources, both physical and ecological, that may be lost by not implementing the project must be weighed against the impacts and costs of the project. By continually assessing this option, the designer can gain confidence that the selected design is appropriate and needed.

(b) Soil properties and special geotechnical problems related to stream stabilization projects

Many channel bank stability problems have a sizable geotechnical component. Although streambanks may be protected from erosive forces of flowing water, forces acting on soils in the bank can induce slope failures. Problems that are geotechnical in nature require a solution that is geotechnically based.

Analyzing bank slopes for geotechnical stability requires an understanding of a complex system of forces. The forces involved in bank instability problems include:

- gravity acting on the soils in the slope
- internal resistance of soils in the slope
- seepage forces in the soils in the slope
- tractive stresses imposed on the soils by flowing water

Knowledge of the site-specific soil characteristics and strength properties is required to understand, predict performance, and design stream restorations and stabilization. Soil characteristics and shear strength parameters are required for various stream stabilization techniques such as bank sloping, retaining wall design, sheet pile design, and pile foundation design.

NEH654 TS14A contains a descriptions of soil characteristics and special geotechnical problems, with a particular focus on bank protection. Guidance on recognizing these problems in the field is presented, along with a description of typical measures for solving them. A particular focus of NEH654 TS14A includes:

- stabilizing very steep slopes caused by erosion at the toe of the slope
- piping/sapping of streambanks, together with sloughing of saturated zones of sands and silts with low clay content
- shallow slope failures in blocky-structured, highly plastic clays
- severe erosion on dispersive clays

(c) Scour calculation

Scour is one of the major causes of failure for stream and river projects. It is important to adequately assess and predict scour in the course of any stream or river design. Designers of treatments such as barbs, revetments, or weirs that are placed on or adjacent to streambeds must estimate the probable maximum scour during the design life of the structure to ensure that the structure will either adjust to or account for this potential change. NEH654 TS14B provides guidance useful in performing scour depth computations.

Although the term scour includes both bed and bank erosion, the emphasis in NEH654 TS14B is on erosion that acts mainly downward or vertically such as bed erosion at the toe of a revetment or adjacent to a bank barb. Scour can be classified as one of three types, as shown in table 14–1.

A treatment may experience one or combinations of these scour types.

Many Federal and state agencies, as well as academic institutions, have developed methods and approaches for estimating these types of scour, and several of those techniques are briefly described in

NEH654 TS14B. Each of these techniques is developed for different types of conditions. The successful use of these techniques requires an understanding of both their inherent limitations, as well as their advantages.

(d) Stone sizing criteria

Many channel protection techniques involve rock or stone as a stand-alone treatment or as a component of an integrated system. Rock is often used where long-term durability is needed, velocities are high, periods of inundation are long, and there is a significant threat to life and property. NEH654 TS14C contains information useful in determining the required particle size to resist fluvial forces, regardless of the application of the stone.

The design of stone or riprap requires engineering analysis. Stone sizing should be approached with care because rock treatments can be expensive and can give a false sense of security if not applied appropriately. Since stone sizing methods are normally developed for a specific application, care should be exercised matching the selected method with the intended use. For example, a design technique developed for conventional riprap revetment may contain inherent assumptions that limit its applicability to a stone barb. The forces that are acting on the barb may be outside the range that were considered for the revetment and may lead to the barb being damaged during less than design flows.

Table 14–1 Scour types

Type of scour	Definition
General	Commonly affects the entire channel cross section, but general scour may affect one side or reach more than another
Bedform	Usually found in sand-bed streams, this is the troughs between crests of bedforms
Local	Commonly affects the streambed immediately adjacent to some obstruction to flow

Many Federal and state agencies have developed methods and approaches for sizing riprap, and several of those techniques are briefly described in NEH654 TS14C. NEH654 TS14C also describes some of the typical applications of both integrated systems and stand alone riprap treatments.

(e) Use of geosynthetics in stream restoration and stabilization projects

A variety of geosynthetic materials may be used for various function and applications in stream restoration and stabilization projects. A geosynthetic is defined as a planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering-related material as part of a manmade project structure or system (American Society for Testing and Materials International (ASTM D4439). Geosynthetics used in stream restoration and stabilization include geotextiles, geogrids, geonets, geocells, and rolled erosion control products. NEH654 TS14D addresses the design of these products.

(f) Use and design of soil anchors

Many treatments do not rely solely on their weight or positioning for their stability. Some external anchoring is needed to resist the fluvial forces of the stream or river. If the treatment relies on an anchor for stability, proper design and installation is essential for project success. NEH654 TS14E covers three of the more common anchoring methods that are in use.

- driven soil anchors
- screw-in soil anchors
- cabling to boulders

These approaches have been used on structures such as rootwads, large woody debris structures, and brush barbs. Depending on the site conditions and design of the treatment, these methods may provide either temporary or permanent anchoring.

The focus of NEH654 TS14E is primarily on driven soil anchors. It provides guidance for estimating the pull-out capacity required of the anchor, given expected streamflows, soil characteristics, and the nature of the object that is to be anchored. Installation guidance is also provided.

(g) Pile foundations

Piles are also used to transfer foundation forces through relatively weak soil to stronger strata to minimize settlement and provide strength. The most likely applications for pile foundations in stream restoration and stabilization projects are as support for bank stabilization (retaining wall) structures and as anchors for large woody material. Piles may be used to support ancillary structures such as culverts, structural channels, bridges, and pumping station structures. NEH654 TS14F addresses the design and analysis required for pile foundation design. Installation issues are also addressed.

654.1403 Treatment techniques

Treatment techniques address a variety of stream stabilization and habitat enhancement techniques. While these treatments are addressed in separate sections, environmentally sensitive stream design will often require combining techniques. There are well-established techniques that are not listed here, including variants of some of the ones that are addressed. Depending on site conditions and project goals, these other treatments may be appropriate, as well.

(a) Grade stabilization

One of the most challenging problems facing river engineers today is the stabilization of degrading channels. Channel degradation leads to damage of both riparian infrastructure, as well as the environment. Bank protection is generally ineffective over the long term if the channel continues to degrade. When systemwide channel degradation exists, a comprehensive treatment plan is usually required. This usually involves the implementation of one or more grade control structures to arrest the degradation process. Another more involved approach would be to change the channel gradient through a reconstruction of the channel, incorporating suitable meander bend geometry.

While grade control can be applied to any alteration in the watershed that provides stability to the streambed, the most common method for establishing grade control is the construction of inchannel structures. A wide variety of structures have been employed to provide grade control in channel systems. These range from simple loose rock structures to reinforced concrete weirs and vary in scale from small streams to large rivers. NEH654 TS14G provides a description of some of the more common types of grade control structures and describes the various design factors that should be considered when selecting and siting grade control structures.

(b) Flow changing techniques

Flow changing devices are a broad category of treatments that can be used to divert flows away from eroding banks. These include devices known as deflectors,

bendway weirs, vanes, spurs, kickers, and barbs. While there are variants in their design and behavior and names, they are basically structures that:

- project from a streambank
- are oriented upstream
- redirect streamflow away from an eroding bank
- alter secondary currents
- promote deposition at the toe of the bank

These treatments are typically constructed of large boulders and stone, but timber and brush have also been successfully used as part of stream design and restoration. NEH654 TS14H describes the attributes and design criteria for many flow-changing techniques. However, the primary focus of NEH654 TS14H is on the analysis, design, and installation of stream barbs. NEH654 TS14H draws on recent field evaluations that focus on areas where these structures have performed well, as well as areas where their performance has been less than satisfactory. A design description includes cautions and warnings related to specific design features. A step-by-step design procedure is also provided.

(c) Soil bioengineering

Stabilizing streambanks with natural vegetation has many advantages over hard armor linings. Compared to streams without vegetated banks, streams with well-stabilized vegetation on their banks have better water quality and fish and wildlife habitats. Vegetation is an extremely important component of biological and chemical health, as well as the stability of the system.

Streambank soil bioengineering is defined as the use of live and dead plant materials in combination with natural and synthetic support materials for slope stabilization, erosion reduction, and vegetative establishment (Allen and Leech 1997). Streambank soil bioengineering uses plants as primary structural components to stabilize and reduce erosion on streambanks, rather than just for aesthetics. As a result of increased public appreciation of the environment, many Federal, state, and local governments, as well as grass roots organizations, are actively engaged in implementing soil bioengineering treatments to stabilize streambanks.

NEH654 TS14I provides guidance for the analysis, design, and installation of many commonly used soil bioengineering techniques. Integrated approaches are addressed, as well as techniques that solely use plants to provide stabilization. Installation guidelines and materials requirements are described in detail. NEH654 TS14I addresses many of the regional concerns and issues that should be considered for the successful application of these techniques.

(d) Large woody material for habitat and bank protection

Large woody materials (LWM) structures are intended to provide habitat and stabilization, until woody riparian vegetation and stable bank slopes can be established. LWM normally decays within a few years, unless it is continuously submerged, but this decay depends on climatic conditions, wood type, and density. Therefore, structures made entirely or partially of woody materials are not suited for long-term stabilization, unless wood is preserved by continuous wetting or chemicals. Woody structures are best applied to channels that are at least moderately stable, have gravel or finer bed material, and that have a deficit of habitats created by wood. NEH654 TS14J addresses the analysis, design and installation of LWM structures.

(e) Streambank armor protection with riprap structures

Structural measures for streambank protection, particularly rock riprap, have been used extensively and with great success for many years. Many situations still require rock riprap to some degree. It is one of the most effective protection measures at the toe of an eroding or unstable slope. Rock is a fairly common commodity in most areas of the country and readily available to most sites. Rock riprap measures have a great attraction as a material of choice for emergency type programs, where quick response and immediate effectiveness are critical.

NEH654 TS14K describes some of the basic principles and techniques used to treat streambank erosion with the more traditional structural measures such as rock riprap and rock-filled gabions. These design basics are applicable to any structure that involves the use of stone. This section also describes the challenges inher-

ent in integrating more vegetatively oriented solutions into these techniques without materially increasing the exposure time and risks involved with failures. This combined approach is desirable to produce a better long-term solution that will be complementary to the natural environment and more self-sustaining. NEH654 TS14K also addresses where stone can be used to provide habitat enhancement, either as part of a traditional bank stabilization structure or as instream habitat boulders.

(f) Articulating concrete block revetment systems for stream restoration and stabilization projects

A variety of natural and constructed materials are available to provide erosion protection in stream restoration and stabilization projects. One of these products is an articulating concrete block (ACB) revetment system. An ACB revetment system is a matrix of interconnected concrete block units installed to provide an erosion resistant revetment with specific hydraulic characteristics. The individual units are connected by geometric interlock, cables, ropes, geotextiles, geogrids, or a combination thereof and typically overlay a geotextile for subsoil retention. An ACB revetment system may be used to provide permanent erosion protection where vegetation and other soil bioengineering practices are not stable for the design event. Typical applications may include entire channel cross-sectional protection, toe and lower side slope protection, stream crossings, grade stabilization structures, and other high energy environments.

NEH654 TS14L describes the ACBs currently available and some of the benefits of their use. A summary of hydraulic performance testing is presented along with design procedure for open channel flow. Critical features are described for typical installations, including subgrade preparation, ancillary components (such as drainage layers), filter placement, ACB placement, system termination, and anchors and penetrations.

(g) Vegetated rock walls

A vegetated rock wall is a mixed-construction biotechnical slope protection. They are primarily used in urban and suburban applications where limited area is available and where there is a need for static bank stabilization. They may be considered to be an alternative to a conventional concrete channel. While vegetated rock walls are expensive, they provide more habitat benefits and are generally considered to be more aesthetically pleasing.

NEH654 TS14M describes the analysis, design, and installation requirements for these structures. Both structural, mechanical and vegetative elements work together to prevent surface erosion and shallow mass movement by stabilizing and protecting the toe of steep slopes. These walls differ from conventional retaining structures because they are placed against relatively undisturbed earth and are not designed to resist large earth pressures.

(h) Fish passage and screening design

Fish passage and screen design is often an important component in stream restoration and water resource management. A wide variety of design issues depend on the project region and species of interest. NEH654 TS14N provides an overview of fish passage and screen design including biological considerations. This section includes a generalized assessment and design approach. Additional references for more information regarding design of fish passage and screen structures are provided.

(i) Stream habitat enhancement using LUNKERS

Little Underwater Neighborhood Keepers Encompassing Rheotactic Salmonids (LUNKERS) are structures that are designed to provide both stability and edge cover for aquatic habitat. While their use has primarily focused on providing trout habitat, they are applicable to other species, as well. LUNKERS have also been used in many projects to enhance the integrity of stream channel geomorphology and bank stability. Where flood volumes and velocities are to be mitigated, LUNKERS can contribute to bank stability and establishment of a secure riparian corridor.

NEH654 TS14O provides step-by-step guidance for the analysis, design, and installation of these structures. A particular focus is on the placement, anchoring, and finished grading for LUNKER structures to result

in stream channels that function efficiently without lateral scour.

(j) Gully stabilization

Gullies develop in response to concentrated flow. Basically, the forces created by flowing water exceed the resisting soil forces. Unchecked, the gullies erode and deliver sediment through a variety of processes that cause loss in soil productivity, channel entrenchment and headward advance, and expansion into the landscape. The processes increase the channel network, bank slope, bank height, and streambank instability resulting from the headward migration of nickpoints. NEH654 TS14P describes the major elements involved with gully formation processes and problem assessment. Alternate approaches to treatment may be considered, depending on gully specifics and landowner desire for effectiveness, cost, and reliability. The information and examples provided in NEH654 TS14P should help in the determination of the approach that may be most suitable for the circumstances.

(k) Abutment design for small bridges

Bridges are installed in a variety of NRCS applications including farm and rural access roads, livestock crossings, emergency watershed protection work, and recreation facilities. They may also be used to replace existing culverts that act as barriers to fish passage. NEH654 TS14Q presents a procedure for determining the ultimate and allowable bearing capacity for shallow strip footings adjacent to slopes. The procedure is appropriate for the design of abutments for the relatively small bridges typically involved in NRCS work.

(l) Design and use of sheet pile walls in stream restoration and stabilization

Sheet pile may be used in a variety of applications for stream restoration and stabilization. It is typically used to provide stability to a stream, stream slopes, or other manmade structures in high-risk situations. Typical applications of sheet pile include toe walls, flanking and undermining protection, grade stabilization, slope stabilization, and earth retaining walls. While sheet pile can be combined with soil bioengineering techniques, it does have some ecologic and geomorphic disadvantages.

NEH654 TS14R describes typical applications for cantilever sheet pile walls in stream restoration and stabilization projects. It also describes the types of sheet pile material, loads applied to the sheet pile, failure modes, design for cantilever wall stability, structural design of the piles, and some construction considerations.

(m) Sizing stream setbacks to help maintain stream stability

Many local communities, watershed groups, counties, and states are developing setback ordinances to help protect stream systems. NEH654 TS14S briefly outlines several guidelines and presents an empirically based equation that predicts the streamway width required to allow a stream to self-adjust its meander pattern. NEH654 TS14S does not cover stream setbacks that are required due to local or state laws or cost-sharing program rules.

654.1404 Conclusion

Treatment technique design contains an overview of some of the frequently used treatment techniques for bank protection, grade protection, and habitat enhancement, as well as analysis techniques for their design. Specifics related to each of the presented treatment and analysis approaches are included in the technical supplements of this handbook.

Many of these treatment techniques have been used and are applicable for small, local issues. While they have been considered to be band-aid solutions, in many cases, a band-aid is all that is needed or justified. In addition, many of the techniques described in this chapter have been used as components of larger, more extensive restoration and design projects.

The reader should not interpret descriptions herein as an endorsement of any product that is mentioned, nor should one treatment or approach be inferred as superior to another. The choice of a particular treatment or combination of treatments should be based on the stakeholders' goals and objectives, watershed conditions, and site condition.

United States
Department of
Agriculture

**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 15 Project Implementation

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Cover photo: Appropriate designs will enable elements of the restoration plan to be implemented and meet the restoration goals, while minimizing disturbance. Rushed implementation without proper designs can result in failure.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 15 Project Implementation

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Chapter 15 Project Implementation

654.1500 Purpose

This chapter addresses general project implementation issues with an emphasis on U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) programs, requirements, and guidance. The three phases involved in project implementation are design, contracts and agreements, and installation. This chapter describes how the phases are interrelated, how each phase requires knowledge of the limitations or restrictions of the other phases, and provides a general overview of the process of project implementation with appropriate references to other NRCS and government contracting documents. For the purposes of this document, the planning phase, including determination of the nature and source of problems, as well as the desired objectives and future conditions, is presented in detail in National Engineering Handbook (NEH) 654.01, 654.02, and 654.03.

654.1501 Introduction

Project implementation is divided into three phases:

- design
- contracts and agreements
- installation

All stream restoration projects should follow these three phases. The level of effort required for implementation may vary substantially from project to project. The level of effort depends on the complexity of the project, tolerance for risk, and the available resources. As described in NEH654.02, the design phase may trigger cycling back through the planning process if preliminary designs cannot meet the planned project objectives.

Stream restoration projects may include design elements that specifically focus on removing perturbations or sources of impairment of ecosystem functions. Critical design elements may include nonstructural approaches requiring changed management, altered access, or possibly changes in institutional regulations. This chapter focuses on the design elements of a stream restoration plan that require construction and how to implement them. Restoration plans can, however, range from simple changes in resource management to largely structural alternatives. The design elements of a restoration plan to be implemented depend heavily on the results of a planning process that adequately identifies the nature of the stream's problems, their sources, and realistic goals and expectations for the implemented project actions.

Specific aspects of the other phases must be considered when working on a single phase in the implementation process. This requires a general understanding of all three phases. For example, the completeness of the design package depends on the type of construction contract or agreement that will be used. The type of construction contract or agreement depends on the staff available to direct the work and/or provide quality assurance (QA).

Direction throughout the implementation process is provided by NRCS policies, technical guidance, and program guidelines. Any project involving the NRCS

is required to follow these established policies and guidelines. The division of responsibilities between the NRCS and a program sponsor is detailed in a project agreement of which there are several different types. These agreements detail the responsibilities of both parties. Sponsors using NRCS program funds are required to follow many of the same implementation policies and guidelines as are required of the NRCS.

The NRCS has been implementing projects for many years, and over that time many technical resources have been developed. Although stream restoration projects may be a complex blend of ecology and engineering, the general process of project implementation is the same as for other types of NRCS projects.

654.1502 Planning

(a) Importance

Planning is the foundation of any successful stream restoration project. The planning process provides an opportunity to investigate, discuss opportunities, and formulate realistic goals and feasible and constructible alternatives. Proper planning provides direction for design, procurement, and installation. During the planning phase, potential problems and/or restrictions are identified. The earlier potential project problems are identified, the easier it is to make changes to the plan without requiring additional time, effort, and funding. See NEH654.02 for a thorough description of the planning process.

654.1503 Design

The planning process defines the purpose for the design, defines realistic goals for the restoration, formulates alternatives to achieve these goals, and determines the level of detail in the design package. The design process requires a more in-depth look at the processes causing the problems and acting on the design features. The design follows NRCS policy in the NRCS National Engineering Manual (NEM) 511, Design.

Problems, objectives, and selected alternatives from the planning process are used by the designer as the basis for the design. The designer will also need other information such as potential permitting requirements, funding, material availability, site or reach limitations, and expected construction period. This information should be collected during the planning process.

(a) Design package

The level of detail of the design package depends on the type of contract or agreement chosen during planning. For example, a firm, fixed-price contract requires a complete design package that will allow contractors to understand in detail what is to be constructed; whereas, a time and material contract may only require minimal drawings and no specifications. The level of detail for design is decided during planning based on available resources and complexity of the project. A complete design package typically contains the following:

- design report
- construction drawings
- material specifications
- design specifications
- engineer's cost estimate
- bid schedule
- estimated construction schedule

(b) Design scheduling

Stream restoration projects may have short periods of time during the year when construction activities can take place. These limited construction periods may be due to harvesting and planting seasons, varying flow levels, fish migration and spawning, or other limitations related to sediment concentrations and limitations on ingress and egress. In addition, the plant materials used in many soil bioengineering support practices for the design require specific harvesting, storage and handling requirements, and planting times. When setting design completion dates and scheduling design staff, consideration must be given to the time required by the contracting process to meet construction scheduling needs.

Funding thresholds and/or construction schedules may require the design to allow for phases of construction. If all of the design cannot be constructed at one time, additional work is generally required to protect what has been completed. Work required to protect each phase should be included in the drawings.

(c) Constructability

For structural alternatives, constructability always must be considered throughout the design. Examples of constructability issues include:

- material availability
- equipment availability
- site and equipment access
- labor requirements
- dewatering requirements
- special measures to protect riparian conditions

When considering constructability, the designers must consider not only if it can be built, but also how can it easily be built. The designers should always be thinking of alternative designs that would have equal or better results but are easier to construct.

(d) Design layout

The physical location of design elements in a stream restoration project can be critical to its success. Although there may be considerable flexibility in the

location (dimensions, alignment, and slope of some design attributes), other design elements may require strict adherence to specific criteria or limitations. However, designers should resist placing unnecessarily restrictive tolerances, as this will increase construction and inspection costs. By providing sufficient information on the design drawings to locate these features in the field, as well as the appropriate tolerances, the responsibility for accurate placement can be placed on the contractor. The design elements can also be checked for accurate placement by someone other than the designer.

It is important for a designer to consider how the design will be transferred from the plans into the field, or fitting the design into the landscape. Only in rare circumstances should a designer direct the construction of major projects without laying out the design in the field. The most common methods used to locate features on a drawing include referencing to a baseline or centerline, creating a grid, or using a global positioning system (GPS). Table 15–1 lists these methods and outlines some of the advantages and disadvantages of each.

Regardless of which method is used, benchmarks must be established and identified on the plans. Benchmarks are objects in the field of known location that will not be disturbed throughout the construction of the project. The location of baselines, centerlines, and local coordinate systems are referenced to benchmarks. When using GPS coordinates, benchmarks are useful as a means of checking the accuracy of the GPS system.

(e) Quality assurance and quality control

When a complete design package is not required, the level of design detail on the plans and specifications depends on the experience and availability of QA and quality control (QC) staff and the experience and reliability of the contractor. For example, minimal drawings and no specifications may be sufficient if a time and material or a labor-hour contract is employed, experienced construction personnel familiar with the design will be onsite, and the contractor is experienced and reliable. Precautions with this method include limited information to perform a review and reliance on the availability of experienced construction oversight.

Table 15–1 Advantages and disadvantages of commonly used layout methods

	Advantages	Disadvantages
Baseline	• Only two points required to locate baseline	• Difficult to reference when features are along a curve
Centerline	• Easy to reference feature locations using stations • Follows the direction of the stream	• Can be difficult or impossible to stake out in the middle of a stream • Curve data required to locate in the field
Grid	• Easy to reference features using CAD	• Difficult to use on large sites • Time consuming to layout grid
Global positioning system (GPS)	• No referencing to other control points • Points can be located quickly in the field • Easy to reference features using CAD	• Satellite reception can be a problem in heavily wooded areas

654.1504 Contracts and agreements

Stream restoration projects may be accomplished using contracts and/or certain types of agreements. There are several factors to consider when choosing the type of contract or agreement best suited for a specific stream restoration project. These factors include:

- program requirements
- project agreement requirements
- extent of design
- complexity of design
- human and equipment resources
- schedule or timeframe
- cost of installation

Stream restoration projects constructed under a specific NRCS program must be installed in conformance with those program's guidelines. Program guidelines often restrict the type of contract or agreement that may be used for installing the practices.

Project agreements include any agreement entered into by the NRCS and sponsors in which detailed working arrangements are established for the installation of cost-shared measures. The responsibilities of all parties involved and details of the actual implementation procedures for the specific project are detailed in the project agreement.

The type of contract depends on the time frame for starting and completing the work. When work must begin and be completed quickly, equipment rental may be more appropriate due to the limited time to obtain or design detailed plans and specifications. This is also contingent upon the availability of qualified NRCS or sponsor representatives needed to direct the work, as noted above.

(a) Contracting

When describing the various methods of contracting for the installation of practices for NRCS projects, it is necessary to divide contracts into two main categories: Federal contracts and non-Federal contracts. These contracts can be further divided into formal or informal, depending on the value of the contract. Under the Federal Acquisition Regulations (FAR) as of 2005, formal contracts must be used for projects with a value greater than $100,000, and informal contracts and contracting procedures can be used for projects with a value of $100,000 or less. Informal contracts are those put in place using simplified acquisition procedures. The advantages of using simplified acquisition procedures are:

- simplicity and ease of buying
- purchases can be made more expeditiously
- usually less paperwork
- usually lower administration costs

There are several types of Federal and non-Federal contracts, each having specific advantages and disadvantages.

It is important to understand the advantages and disadvantages of the types of contracts available for use in the installation of stream restoration measures to complement the planning, design, and construction efforts required for a successful project. Some advantages might be minimal administrative burden, minimal construction oversight, and maximum incentive for the contractor to control cost. Conversely, some types of contracts require maximum administrative burden, maximum construction oversight, and no incentive for the contractor to control costs. Table 15–2 summarizes the various items to consider when selecting the type of contract best suited for a specific situation.

Federal contracts

Federal contracts are governed by the FAR. Information can be found at the following Web site:

http://www.arnet.gov/far

NRCS contracting regulations also include the Agriculture Acquisition Regulation and the NRCS Acquisition Regulations. Federal contracts that might be applicable to stream restoration work include fixed-price, cost-reimbursement, incentive, time-and-materials, labor-hour, equipment rental, and letter contracts. These contracts can be formal or informal (simplified procedures), depending on the value of the contract.

Federal contracts are normally used when project sponsors do not have the capacity to solicit, award, and/or administer a locally awarded contract and/or have the necessary resources to accomplish the work with their own forces under certain types of agreement actions. A warranted NRCS contracting officer can provide pertinent information and guidance for Federal contracting procedures. A wide variety of Federal contract types may be used under the acquisition regulations. The selection of the appropriate type of contract, as well as the method of solicitation, is situation driven. NRCS Federal contract actions must comply with all applicable acquisition regulations and policy.

Fixed-price contracts—Place the maximum risk and full responsibility on the contractor for all costs and resulting profit or loss associated with the work. This type of contract provides the maximum incentive for the contractor to control costs and perform effectively and imposes a minimum administrative burden on NRCS and/or sponsors. A fixed-price contract requires the contractor to understand, in detail, what is to be constructed before bidding to do the work. This requires a design that includes detailed drawings, specifications, and a bid schedule containing a bid item for each major item of work. The designer must provide a cost estimate by bid item so that the cost of the work can be estimated and the contracting officer can ac-

Table 15–2 Contract comparisons

Contract type	Federal contract	Non-Federal contract	Administrative burden	Construction oversight	Incentive for cost control	Required design effort	NRCS/sponsor's risk	Contractor's risk	Comments
Fixed-price	√	√	low	low	high	high	low	high	Most preferable type of construction contract. Must have accurate cost estimate and construction details
Cost reimbursement	√	√	high	high	low	low	high	low	Use only when uncertainties in contract performance do not permit use of any type of fixed-price contract
Incentive	√	√	—	—	high	—	—	—	Rarely used for construction. May be fixed-price or cost-reimbursement contract
Time-and-materials	√	√	high	high	low	low	high	low	Use only when accurate cost estimate, extent or duration of work cannot be established. See limitations at FAR 16.601(c)
Labor-hour	√	√	high	high	low	low	high	low	Same as time-and-materials contract, but without materials being supplied by contractor
Letter	√	√	high	high	low	low	high	low	Requires national level approval. Only use when the head of the contracting activity determines in writing that no other contract type is suitable. FAR 16.603–3
Equipment rental	√	√	high	high	low	low	high	low	Similar to time-and-materials or labor-hour contracts and is use when nature of work and limited design details preclude using a fixed-price construction contract

Note: — indicates the rating could be either high or low

cess the reasonableness of the bids. Most fixed-price contracts are awarded after contractors have submitted a sealed bid in response to an Invitation for Bids (IFB). The IFB includes the drawings and specifications for the work and specific contract requirements. This is the most preferred type of contract and should be used to the maximum extent practicable. Fixed-price contracts can be accomplished using either simplified or formal acquisition procedures, depending on the value of the project. The design effort and level of detail may be the same for simplified fixed-price contracts as it is for formal fixed-price contracts.

Cost-reimbursement contracts—Suitable for use only when the cost of the work cannot be estimated with sufficient accuracy to use a fixed-price contract. The cost of the work is estimated for the purpose of obligating funds; however, a detailed cost analysis is not required. The contractor must have an accounting system adequate for determining incurred costs that are reimbursable. This type of contract requires significantly more government oversight during the construction phase to document that efficient construction methods and efficient cost controls are being used. It provides little incentive for the contractor to control costs and perform effectively and imposes a much larger administrative burden on the contractor, NRCS, and/or the sponsors.

Incentive contracts—Link the contractor's profit to performance by establishing reasonable and attainable targets that are clearly communicated to the contractor. These contracts are designed to motivate the contractor in specific areas that might not otherwise be emphasized, such as motivation for early completion. Incentive contracts discourage inefficiency and waste. Incentive contracts can be fixed-price incentive contracts or cost-reimbursable incentive contracts. These types of contracts are normally used for performance-based service contracts and rarely, if ever, are used for construction work.

Time-and-materials contracts—Used to procure supplies or services on the basis of direct labor and materials costs. Time-and-materials contracts should be used only when it is not possible to accurately estimate the extent or duration of work or to anticipate costs with any degree of confidence. With this type of contract, there is no incentive to the contractor to control costs, significant government oversight is required, and a much larger administrative burden is imposed on NRCS and/or the sponsor.

Labor-hour contracts—A variation of the time-and-materials contract, differing only in that materials are not supplied by the contractor.

Equipment rental contracts—Used in instances where a fixed-price construction contract would be impractical because of the nature of the work and when it would not be feasible to prepare detailed drawings and specifications. It requires substantial construction oversight and imposes an additional administrative burden on NRCS.

Letter contracts—Written preliminary contractual instruments that authorize the contractor to begin work immediately. A letter contract should be as complete and definite as feasible under the circumstances, and there are requirements for scheduling price negotiations and establishing set prices as soon as feasible. A letter contract may be used only after the head of the contracting activity or a designee at the national level determines in writing that no other contract is suitable.

Non-Federal contracts

Non-Federal contracts, like Federal contracts, can be categorized as formal or informal (using simplified procedures). They can also take the form of fixed-price, cost-reimbursement, or incentive type contracts in accordance with the Contracting Local Organization's (CLO) procurement regulations and the Federal requirements imposed on the CLO as provided in the Code of Federal Regulations (CFR) at 7 CFR 3016.36. CLO contracts are those administered by a non-Federal entity such as state government, county government, or other local sponsor. The contract language, including general and special contract clauses, will likely differ from one entity to another. The CLO, NRCS, or both, will provide a QA inspector to verify that work is performed according to design and contract requirements. In other instances, the CLO may require substantial NRCS assistance to develop and administer the contract and may incorporate NRCS general and special provisions in the contract.

The amount of NRCS involvement in non-Federal contracts may depend on the capabilities of the contracting organization.

Fixed-price CLO contracts—Can include both construction contracts and vegetative contracts. A fixed-price CLO contract is similar to a fixed-price FAR contract in that it requires the same degree of detailed design, and it places the maximum responsibility on the contractor, with the maximum incentive to perform in an efficient cost-effective manner. Also, note that a CLO contract awarded under informal or simplified procedures can require the same degree of detailed design as a formal contract. As with fixed-price FAR contracts, the level of construction oversight and contract administration may be minimized with a fixed-price CLO contract. This is not to infer that no QA inspection is needed, only that the intensity of inspection may be less on this type of contract than on other contract types, such as equipment rental contracts where the inspector must verify the hours of equipment and personnel and must direct the work.

Time-and-materials CLO contracts—Non-Federal contracts used to procure supplies or services on the basis of direct labor and materials costs. Time-and-materials contracts should be used only when it is not possible to accurately estimate the extent or duration of work or to anticipate costs with any degree of confidence. With this type of contract, there is no incentive to the contractor to control costs, significant CLO oversight is required, and a much larger administrative burden is imposed on the CLO and perhaps the NRCS (depending on the NRCS level of involvement).

Labor-hour contracts—A variation of the time-and-materials contract, differing only in that materials are not supplied by the contractor.

Equipment rental contracts—Non-Federal contracts used in instances where a fixed-price CLO construction contract would be impractical because of the nature of the work, and it would not be feasible to prepare detailed drawings and specifications. It requires substantial construction oversight and imposes an administrative burden on the CLO and/or NRCS.

Contract value

The value of the contract dictates whether simplified or if formal contracting procedures must be used. As of 2005, simplified acquisition procedures under the FAR may be used for work that costs $100,000 or less, but more formal acquisition procedures are required for work in excess of $100,000. Fixed-price contracts, cost-reimbursement contracts, incentive contracts, time-and-materials contracts, labor-hour contracts, letter contracts, or equipment rental contracts may be used under both simplified and formal contracting procedures. Table 15–3 lists the various contracting procedures required for construction or service (equipment rental) contracts at each dollar threshold where changes in the procedural requirements are required by the FAR.

While it is most often the size of the procurement that determines whether to use simplified or formal procedures, there can be some apparent exceptions in practice. For example, exigency Emergency Watershed Protection (EWP) solicitations with estimated costs in excess of $100,000 require a formal bid opening and award of a formal contract. However, the advertising time and scope can be shortened because of the exigency. In some extreme cases, the NRCS would only notify 5 or 10 contractors, and only 2 or 3 days would be needed to develop the solicitation, hold the site showing, and award the contract. But, the result should be a formal contract for those actions exceeding the simplified acquisition threshold of $100,000.

(b) Project agreements and other assistance relationships

The NRCS and a sponsor can jointly install works of improvement using project (cooperative) agreements. The NRCS policy regarding project agreements is found in the National Contracts, Grants and Cooperative Agreement Manual (NCGCAM) GM 120–510 through 517. Project (cooperative) agreements must define the contracting, and/or use of the sponsor's forces, cost-sharing (as applicable), and administrative procedures that will be used to carry out the selected method of installation. The sponsor may choose to contract for or perform, using their own forces, engineering designs, cost estimates, installation of the stream restoration measures, and construction oversight and associated inspection services. The agreements must define the contracting, cost-sharing (as applicable), and administrative procedures needed to carry out the selected method.

The NRCS must assure the quality of the design, contracting, and any construction carried out by project sponsors through design quality reviews, construction oversight, or other means established by the state conservationist (STC) and in accordance with the NCGCAM and NEM.

Table 15–3 Dollar thresholds requiring changes in procedures by the FAR

	Construction				Service (equipment rental)				
Alternate methods of contracting	**Less than $2,000**	**More than $2,000**	**More than $25,000**	**More than $100,000**	**More than $2,500**	**Less than $10,000**	**More than $10,000**	**More than $25,000**	**More than $100,000**
Verbal quotes	x				x	x			
Written quotes (RFQ)		x	x				x	x	
Sealed bid (IFB)				x2/			*	*	*
Negotiated (RFP) (rarely used)							*	*	*
8(a) Set Aside	*	*	*	*	*	*	*	*	*
FedBizOpps—less than 30 days			x1/					x1/	
FedBizOpps—minimum 30 days				x1/					x1/
Davis Bacon		x	x	x					
Service Contract Act					x	x	x	x	x1/
Bid security				x2/					
Performance and payment security			x3/	x			x3/		

* if applicable
1/ Unless EWP Exigency or 8(a) Set Aside
2/ Unless 8(a) Set Aside
3/ Alternate payment protection
Can use one solicitation with multiple award contracts
Can use Indefinite Quantity Contract with minimum/maximum qualifies, rather than Fixed Price with quantity variations
EWP work can be done using Master Package

Structural practices designed by non-NRCS personnel will be done under the direction and supervision of a licensed professional engineer in the state where the measures will be installed. All design reviews will be in accordance with the NEM 501 and with any procedures established by the state conservation engineer (SCE). Predesign conferences for more complex measures must be held with appropriate representatives from the NRCS, the sponsor(s), and the architectural and engineering (A&E) firm, if used.

The cooperative agreement approach can extend the capacity of NRCS to provide assistance under NRCS programs. It can also provide the sponsor with primary control of the installation process, and it can match the level of cooperative work to the sponsor's capability and resources. For example, the sponsor may lack or be unable to obtain contract administration assistance, but may have personnel or be able to secure professional engineering and other technical services that can evaluate, design measures, and prepare contract documents. The NRCS could then provide contract administration and construction assistance after the sponsor prepares the drawings and specifications. The project (cooperative) agreement would list the division of responsibilities with explicit details of the tasks to be performed by the sponsor and the tasks to be performed by the NRCS.

Regardless of the type of agreement that will be used, all project (financial assistance cooperative) agreements must meet the applicable requirements in the NCGCAM GM 120–510 through 517, as well as the applicable Federal regulations in 7 CFR 3015 and 3016.

In addition, the NRCS must ensure that all required prerequisites to signing a project agreement for the installation of project measures are met in accordance with NCGCAM 514.35.

For projects installed using contracts, there must be a project agreement between the sponsor and the NRCS authorizing the implementation of the stream restoration measures using either a Federal contract or non-Federal contract.

Project agreement for a Federal contract
A project agreement for Federal contracting may be used for installation of stream restoration measures in accordance with the NCGCAM 510.10(b) and 510.11. This work is normally performed under a competitively awarded contract. The NCGCAM, sections 510 and 514, provides applicable guidance and detail for the use of this type of project agreement.

Project agreement for a non-Federal contract
A project agreement for non-Federal contracting may be used in accordance with the NCGCAM 510.10(b), 510.11, and 510.30 through 510.39. This work should also be performed under a competitively awarded contract. The NCGCAM, sections 510 and 514, provides applicable guidance and detail for the use of this type of project agreement.

In addition, project installation may be accomplished using contract and/or agreement actions. Project agreements can be used to install works of improvement as follows.

Project agreement for force account
Force account agreements may be used for project installation in accordance with the NCGCAM 510.15. This method is used when the sponsor performs the work using its own equipment and personnel. The sponsor may supplement their own equipment through rental of relatively minor amounts of equipment. However, force account agreements can offer the potential for additional costs and poor quality work. Because of these factors, the sponsors may install project measures by force account only under the conditions indicated in the NCGCAM 510.15(c). The NCGCAM, sections 510 and 514, provides applicable guidance and detail for the use of this type of project agreement. The Sponsoring Local Organization (SLO) must keep accurate records of cost of all work performed. It requires substantial construction oversight and imposes an administrative burden on the parties to the agreement. Difficulty may arise in coordinating the force account work with the ongoing duties and other work that is required of the SLO workforce.

Project agreement for division of work
Division of work agreements may be used for cost-sharing land treatment measures. These are measures that are authorized under Public Law 83–566 (PL 566) watershed work. This type of project agreement may be used for project installation in accordance with NCGCAM 510.16. The work is divided between NRCS and SLO with the details of each organization's responsibilities spelled out in the project agreement. If the work is to be shared on a division of work basis, it must be described in the watershed plan, and

cost estimates must be included in the supporting tables. The SLO is not required to keep records of expenditures. Detailed NRCS cost estimates are maintained to document that PL 566 costs do not exceed the authorized rate.

Project agreement for performance of work
Performance of work agreements require that the value of work to be performed by the SLO be determined by negotiation between the SLO and NRCS and be included in the project agreement. The NRCS must estimate the cost of the work to establish the maximum value of work before signing the agreement. This type of project agreement may be used for project installation in accordance with NCGCAM 510.17. This is applicable if the works of improvement are to be cost shared on a percentage basis. The percentage rate of cost sharing is to be included in the watershed plan agreement. This work should normally be performed under a competitively awarded contract. However, the sponsors may be able to perform certain elements of the project work with their own forces or with contributed labor, equipment or materials in lieu of providing cash. The NCGCAM 510.17 and 514 provide applicable guidance and detail for the use of this type of project agreement.

Project agreement for average cost
A project agreement for average cost may be used for installation of land treatment measures in accordance with NCGCAM 510.18. This method cannot be used with units of government. Average cost land treatment contracts (financial assistance agreements) are authorized under PL 566 with the cost of the work being the average cost of similar work within a defined area over a set period of time. The average cost is usually set by county and is determined by averaging the cost of all similar work for which there are records for a 1-year period. For example, the per-unit cost of work, such as bank shaping, would be averaged for all of the bank shaping that falls within a specific level or category. Bank shaping might be categorized as light, medium, or heavy based on the effort required to shape an acre. An average cost of light, medium, or heavy bank shaping would then be established for a county based on previous year costs.

Long-term contracts (financial assistance agreements) are authorized under PL 566 for contracts between the SLO and a landowner or between the NRCS and a landowner. The contract period ranges from 3 to 10 years with all cost-shared measures being installed at least 2 years before the financial assistance agreement expires. The work items are scheduled according to a plan with the purpose of reducing erosion and sediment damages within a PL 566 watershed. The work includes items like shaping and vegetating eroded areas, building grade control structures, and other measures that are approved for the program. Stream restoration may be approved for this type of program.

(c) Personnel for administering contracts and agreements

Contracting is a team effort that relies on individual team members to perform specific tasks. The titles of team members are sometimes different for non-Federal contracts than for Federal contracts, but their tasks transcend from one to the other. The six principal responsible positions on a team are administrative officer, contracting officer (CO), contract specialist, SCE, GR/COR, and government inspector.

NRCS state administrative officer (SAO)
The SAO is responsible for all administrative matters for CLO contracts and most agreements. This includes making sure funds are available and that people and equipment needed to administer the agreement and any resulting contract will be available when needed. For Federal contracts, the NRCS-warranted contracting officer is responsible for contractual matters in accordance with FAR 1.602.

Contracting officer (CO)
The CO is responsible for administering the contract including ensuring that the proper type of contract is being used and funds are spent according to regulations. For CLO contracts, the NRCS contract specialist or SAO is there to lend guidance to the CO who is a CLO official or employee.

NRCS contract specialist
The NRCS contract specialist assists the administrative officer in contract matters for CLO contracts and agreements. The NRCS contract specialist is the CO on Federal contracts. The contract specialist works closely with the design engineer to ensure that the design package included in the solicitation package contains the information needed for the work to be contracted.

State conservation engineer (SCE)
The SCE is responsible for the design and ultimately responsible for ensuring proper construction of the works of improvement. The SCE or an assigned staff member works closely with the CO on Federal contracts or the NRCS administrative officer and NRCS contract specialist for CLO contracts to provide design information needed to define and contract the work. The SCE is also involved in making sure there is adequate inspection staff to provide the required level of construction oversight and QA.

Government representative (GR) on non-Federal contracts or contracting officer's representative (COR) on FAR contracts
The GR or COR is responsible to the SEC and the CO to see that the work is carried out as designed and in accordance with the contract requirements.

Construction inspector
The construction inspector is responsible for the day-to-day QA inspection required to ensure that the work is installed according to the design, industry standards, and contract requirements. The construction inspector is responsible to the GR/COR and CO to assure that the quality of the work is consistent with contract requirements.

The availability of personnel to perform specific tasks should be the first consideration when planning a project. For example, consider a stream restoration project that is planned to use volunteer labor under a force account agreement with the sponsor and rented equipment through a Federal or non-Federal equipment rental contract. The project will use live vegetation that must be planted during a specific planting season. This type of work would require contract personnel to prepare the equipment rental contract within a narrow timeframe and require significant construction oversight by trained field personnel. This normally requires much more oversight and administrative effort than would be the case with a firm fixed-price contract. A fixed-price contract would require a very detailed and thorough design package, but would likely require less contract administration and construction oversight. It might also be possible to do the work under a locally led CLO contract with relatively little burden on NRCS contracting personnel and minimal NRCS construction oversight.

654.1505 Installation

Installation of stream restoration projects in accordance with approved drawings and specifications is essential if the project is to serve its intended purpose and expected service life with normal operation and maintenance. The NRCS has standardized construction practices and procedures to ensure that stream restoration and other projects are installed according to design. These procedures provide uniformity in NRCS activities and result in common understanding between all parties involved with the design and installation of stream restoration and other projects. Policy concerning NRCS construction practices and procedures can be found in the NEM 512.

(a) Personnel required during installation

Installation of a stream restoration project is a team effort where each team member has specific responsibilities. The team is made up of the following members:

- owner or contracting officer
- engineer
- specialists for support of specific design elements
- government representative or contracting officer's representative
- construction inspector
- contractor

Owner
The owner is responsible for contracting for construction. For NRCS Federal contracts, the NRCS is considered the owner during construction and according to the FAR, the CO is responsible for ensuring performance of all necessary actions for effective contracting, ensuring compliance with the terms of the contract, and safeguarding the interest of the United States in its contractual relationship. On non-Federal contracts, the CLO is considered the owner during construction and an employee or CLO official serves as CO. A private individual or group is considered the owner on jobs that are administered by a private individual or group.

Engineer
The engineer is responsible for the technical requirements of project installation and represents the owner. The engineer is assigned technical and contract administration duties as outlined in the quality assurance plan (QAP) and in an appointment letter issued by the CO of Federal contracts or issued by the SAO for non-Federal contracts. The engineer may be an NRCS employee, an A&E firm employee, or an employee of the CLO or partnership agency.

Other specialists including biologist, ecologist, forester, fluvial geomorphologist, or others
Other specialists may be needed to support specific elements of the design, monitor specific site conditions for plants and animals, and assure that goals of the planned project are realized through the construction and implementation of the project's design elements. A specialist may be an NRCS employee, an A&E firm employee, or an employee of the CLO or partnership agency.

Government representative
The GR is an NRCS employee who has the responsibility to protect the government's interest and maintain close working relations with the CLO. The GR is an engineer if NRCS has QA responsibility for a construction project. The NRCS administrative officer will appoint a GR by letter for all construction contracts that are administered by others and that use Federal funds. This appointment is not normally provided for contracts handled by private individuals or informal groups.

Contracting officer's representative
The COR is an NRCS employee appointed by the CO and has the responsibility to protect the government's interest on Federal contracts. On NRCS construction projects, the COR is an engineer with responsibility for QA.

Construction inspector
The construction inspector is responsible for QA testing, engineering surveys, daily documentation of construction activities, coordination with the contractor's QC personnel, and maintaining the as-built plans. The NRCS CO or SAO appoints the inspector for Federal and non-Federal contracts respectively. The level of inspection required of the construction inspector is detailed in the QAP. For work other than that performed under formal and some informal contracts, formal appointment of an inspector is not required. However, an inspection staffing plan must be developed and approved in accordance with NCGCAM 516.3 for all project agreements in excess of $25,000 which results in either a non-Federal contract, force account agreement, or a performance of work agreement. The inspector should be made aware of the specific items that require QA inspection with sufficient guidance given to ensure that the inspection is adequate to achieve compliance with the design.

Contractor
The contractor is an individual or firm that installs the stream restoration measures. The contract or agreement with the owner may be formal, as in project installation, or informal, as with an individual landowner or informal group. As mentioned in the contracts and agreements section, provisions are available for the project sponsor(s) to perform work with their own forces under agreements that may include division of work, performance of work, or force account. Thus, the sponsor could also be the contractor.

(b) Preconstruction activities

Preconstruction activities include soliciting contractors, showing the site, selecting the contractor, and meeting with the contractor before construction. The time and effort devoted to preconstruction activities vary according to the complexity and value of the work. The objective in performing these activities is to ensure that the prospective bidders understand the work and contract requirements they must fulfill if they are selected to do the work and acquire the services of a responsible qualified contractor.

Soliciting contractors to bid the work
Solicitation marks the beginning of preconstruction activities. The solicitation package for both formal and informal fixed-price contracts consists of the drawings, specifications, bid schedule, and contract provisions with which the contractor must be familiar to bid the work. For informal work performed under a land treatment contract (financial assistance agreements), the contract used to delineate the work performed may be an agreement between an individual land user and a contractor. In this case, a minimal amount of written materials may be all the contractor needs to bid the work.

For formal contracts valued in excess of $100,000, the solicitation process generally takes 60 to 90 days. During this time, the CO compiles the solicitation package, advertises the work to prospective bidders, shows the site, opens bids, determines the responsive low bidder, and awards the contract. For Federal contracts, a solicitation notice must be posted for a minimum of 30 days on the Federal Business Opportunities (FedBizOpps) Web site so that it is available to all who wish to bid the work. The COR/GR reviews the solicitation package and provides comments to the CO before the solicitation package is issued. The COR/GR is normally responsible for staking the site for the site showing and assisting with the site showing. The construction inspector should also be involved at the earliest possible stage to provide review comments, assist in staking, and assist with the site showing.

When performing work under an Emergency Watershed Program (EWP) exigency, the time for the formal solicitation process may be greatly reduced if the emergency dictates this need and the action is in accordance with FAR 6.302–2. As an example, 5 to 10 contractors are notified and given an opportunity to bid the work, and the development of the solicitation, site showing, and contract award may be accomplished in as little as 2 or 3 days. For all other contracts exceeding the simplified acquisition threshold of $100,000, the work must be advertised on FedBizOpps with a solicitation notice posted a minimum of 30 days.

The amount of time needed to solicit contractors and award the contract may be much less than needed for formal contracts valued in excess of $100,000. Contracts valued at $100,000 or less may be accomplished using a simplified acquisition process with less stringent requirements. See table 15–2 for a list of the items to consider whenever selecting the type of contract and/or agreement best suited for a specific project situation.

The concept of advertising and bidding the work based on drawings and specifications that accurately depict the planned work also applies to informal contracts such as those between landowner and contractor. In some cases, the value of the work does not warrant taking the time to solicit bids from several contractors. The work may be depicted with sketches and/or a field review requiring minimal overall preconstruction activities.

Showing the site to potential bidders

Showing the site to the potential bidders is necessary so that they may inspect the area, determine the scope of the work, and receive answers to questions. Stakes and/or flagging shall be used to identify major items of work and their relationship to other elements of the proposed project. For formal contracts, the site showing is conducted by the CO with assistance from the COR/GR. The CO normally reviews contract items, and the COR/GR reviews the technical items. The inspector(s) should attend so that they have first hand knowledge of items discussed and contractor's concerns. The following items should be identified and discussed at the site showing, as appropriate:

- access roads and site entrance
- rights-of-way and construction limits
- clearing limits
- onsite vegetation or instream features that must be preserved
- location of known utilities
- proposed location of the design features of the project
- existing structures to be removed
- proposed borrow and waste areas
- location of geologic test holes/pits
- contractor's responsibility for pollution control and stormwater permit
- environmental issues
- construction safety
- other important features

Questions should be answered by referring to the solicitation package when the answer can be found in the package. When a question cannot be answered by referring to a specific part of the solicitation package, the question and answer should be recorded and included in an amendment to the solicitation package that is made available before the bid opening to all solicitation package holders. This process can also apply to solicitations using simplified acquisition procedures if site conditions or the complexity of the work warrants the need for a more formal site showing.

Conversely, if the work is relatively simple in nature and informal contracts are used, the items listed above should be reviewed with the contractor before the work begins. In most cases, these items have a bearing on the value of the work and should be reviewed with the contractor before bidding the work.

Awarding a contract
This comes after the CO determines that the low bidder is responsible. A responsible bidder must have the capacity, credit, integrity, tenacity, and perseverance to perform the work as specified. In making this determination, the CO may rely on the COR/GR to review submittals and perform a field review of the contractor's equipment and operations on previous or ongoing jobs. The CO will generally interview others who have dealt with the contractor to obtain their views of the contractor's capacity, integrity, tenacity, and perseverance.

Preconstruction conference
A preconstruction conference should be attended by the CO, COR/GR, inspector, and contractor before beginning construction. For Federal contracts, the term postaward conference is used. For informal work, the owner, engineer, inspector, and contractor should meet before construction to ensure that all have a thorough understanding of the work. The following items should be reviewed, as appropriate:

- drawings and specifications
- contract provisions
- invoicing procedures and progress payments
- contractor's construction schedule
- land rights and construction work limits
- permits
- easements
- work restrictions
- utilities in or near the work area
- construction safety, public safety, and safety plan
- sanitary facilities
- construction office space if provided by the contractor
- construction materials and material approval process
- staking and construction surveying
- contractor QC
- owner QA
- removal of water and dewatering plan(s)
- stream closures
- protected species of plant or animal
- weather and time extensions
- for intermittent inspection, a checklist of items to be inspected where the contractor or owner are required to notify the inspector
- any other item that could have an impact on the work

For formal contract work, the preconstruction conference minutes and all basic information of the conference should be recorded in the official job diary. This may also be applicable for informal contracts as needs dictate.

(c) Construction activities

Work begins
The work begins on formal contracts only after the contractor has received a notice-to-proceed (NTP). The engineer or construction inspector and the contractor schedule to meet onsite at the beginning of work to review last minute details before the start of work. For formal contracts, there are several items that must be in place before the start of the work including:

- contractor's safety plan, first-aid facilities (first-aid kit, stretcher)
- construction barriers as necessary
- sanitary facilities
- hard hat sign(s)
- bulletin board with specific items posted
- stormwater permit, associated posting requirements, and notice-of-intent
- stormwater pollution prevent plan (SWPPP) and associated best management practices

- contractor's quality control plan
- identification and CO approval of several key players on the contractor's staff including the superintendent, safety officer, surveyor, and QC manager
- other items as required by the contract

The first item of business is for the contractor and the construction inspector to hold a tool box safety meeting. In this meeting, the contractor's safety plan is reviewed and emphasis is placed on job-specific safety issues. Other items that may be covered in this meeting include site maintenance, pollution control, and utilities. NRCS places the safety of the public, the contractor's personnel, and NRCS personnel at the forefront of concerns on any construction job. NRCS policy related to safety can be found in the General Manual, Title 110, Part 402 (GM 110–402) and in the NEM 503.

Materials

The materials that are to be incorporated into the work must be approved by the CO or the individual assigned by the CO, such as the engineer. Materials must be evaluated in relation to applicable industry standards and/or specifications to determine that they meet design requirements. Material quality is evaluated by one or more of the following procedures:

- results of laboratory testing performed by the NRCS, consulting firm, and commercial lab
- written certification from the manufacturer with test results attached
- material markings and tags
- examination and/or testing onsite

Final approval of vegetative materials should not be given until immediately before installation to ensure that the materials are alive, have been stored and handled properly, were harvested or grown correctly, and are in the specified condition for the project application.

QA activities and specific contract administration tasks

QA tasks must be performed at the field level. QA activities vary in accordance with the complexity and hazard class of the stream restoration project. For the more complex projects, projects being constructed under formal contract, and any project that requires substantial QA, a QAP must be prepared and used in accordance with the Natural Resources Conservation Service Acquisition Regulations (NRCSAR) and the NCGCAM. The QAP will identify the individuals with the expertise to perform various QA tasks, outline the frequency and timing of testing, estimate the contract completion date, and be co-approved by all responsible supervisors, SCE, and CO (Federal contracts) or administrative officer (non-Federal contracts).

Although it is advisable to have a written QAP, a written QAP is not required for jobs administered by an individual or landowner. In some instances, the QA inspector need only inspect the work at various stages or times during construction. For these instances, a checklist should be provided to the contractor and landowner that would list specific parts or stages of construction that will require QA inspection. The contractor or landowner should notify the inspector in advance of the need for inspection of those items.

The QAP defines NRCS QA duties including the following quality, quantity, and timeliness requirements:

- general description of the work
- items requiring inspection
- timing of inspections
- skills needed by inspectors
- number of staff hours
- equipment and facilities needed
- names and qualifications of personnel
- supervisor's statement of availability

The contractor is responsible for QC to ensure that the work installed meets the minimum requirements of the contract. QC is a bid item for most formal contracts and NRCS Construction Specification 94, Contractor Quality Control is included in the contract. For less formal work, the contractor's QC duties should be defined in a manner that will ensure contractor QC is performed to attain the desired quality of work.

The owner (NRCS or CLO as applicable) is responsible for QA activities including observing construction methods and procedures, reviewing QC testing activities, conducting material testing to evaluate the QC

program, and other measures to ensure compliance with contract provisions.

The performance of QA duties in an efficient economical manner requires:

- providing the proper number of qualified personnel with the knowledge, skills, and abilities necessary to conduct timely and effective inspection as outlined in the QAP
- continuous coordination with the contractor's QC representative to ensure NRCS QA activities are effective
- minimizing interference with the contractor's production activities

Under no circumstances will certification stating the work has been accomplished in compliance with the drawings, specifications, and other contract provisions occur without a physical review and documentation of the work performed. Continuous inspection is required for any activity the quality of which cannot be verified by intermittent inspection and for work that cannot be readily removed or replaced if it fails to meet the requirements of the contract. Intermittent or periodic inspection may be adequate for certain phases of project activities, depending on the complexity of the installation and potential impacts on the health and welfare of the public.

Documentation of construction activities and development of as-built plans is required for formal contract work and should be considered for other more informal work. The construction inspector maintains the official job diary, recording day to day activities and site and weather conditions. Written documentation should be supported by photo documentation. For more information on construction inspection, see NEH 645, and NCGCAM 517.5 through 517.13.

654.1506 Conclusion

Design, contracts and agreements, and installation are the three phases of project implementation. Consideration for specific aspects of the other phases must be given when working on a single phase in the installation process.

Quality planning is required for the efficient implementation of a project. A quality stream restoration planning effort requires an interdisciplinary team that can formulate a plan that is feasible, constructible, effectively addresses the identified problems, and meets the planned goals without adverse effects on the rest of the stream system. Planning provides the basis for the project implementation process.

The selected alternative plan that results from the planning process may range from simple actions to a complex of management and structural measures to achieve the goals of the project. Be aware that stream restoration design elements may include critical nonstructural approaches that may be integral to the success of the project.

Information that is gathered during the planning process and used to formulate a solution is analyzed in greater detail during design. The level of detail required in the design is dependent on the type of contract or agreement being used, complexity of the design, availability of experienced NRCS or CLO staff for QA activities or to direct the work if using equipment rental or similar types of contracts, and experience of potential contractors.

Designing a stream restoration project requires consideration of installation periods that can best be constructed in phases. Phases may require additional design to protect the work completed at each phase. Constructability should always be considered throughout the design. All of these considerations, as well as others, must be addressed since it is easier and often much less expensive to make changes during planning and design than during installation.

Various types of contracts and agreements are available for installing works of improvement that will permit accomplishing the work in the most beneficial manner to all concerned. In addition to formal

contracts, informal contracts and agreements are used in much of the work that is accomplished with cost-shared funds. In selecting the contracts and/or agreements to be used, careful consideration must be given to the type of work, amount of work, desires and capabilities of the sponsors and/or land user, and the interests of the NRCS. Planning and design decisions should be made with installation procedures in mind and consideration given to the availability of qualified personnel needed for construction oversight and agreement during all phases of a project.

NRCS requires QA for all levels of construction to ensure that projects are installed in accordance with the design, industry standards, and contract requirements. The contractor is responsible for QC to ensure that the work installed meets the minimum requirements of the contract. The owner is responsible for QA activities including items related to safety of the public, the contractor's workforce, and NRCS personnel. NRCS has standardized construction practices and procedures to ensure that stream restoration and other projects are installed according to design.

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**Natural
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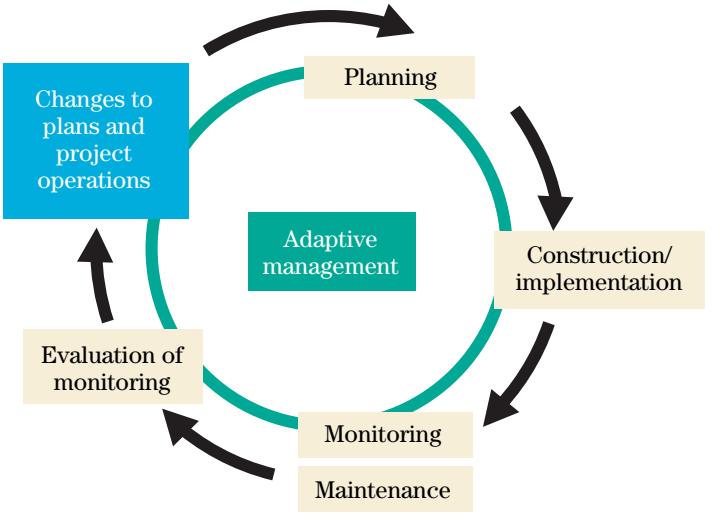
Part 654 Stream Restoration Design National Engineering Handbook

Chapter 16 Maintenance and Monitoring



Issued August 2007

Cover photo: Monitoring during and after implementation enables project managers to determine the level of success achieved and identifies when maintenance is needed.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 16 Maintenance and Monitoring

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Chapter 16 Maintenance and Monitoring

654.1600 Purpose

Any stream restoration project, whether it is designed solely for habitat improvement or strictly to meet some human requirement, is implemented to achieve specific goals or objectives. Continued performance of the project features and health of the biotic resources depend on appropriate maintenance and monitoring of the system. Maintenance and monitoring are actions intended to ensure that the objectives of the stream restoration project are met over time. This chapter provides an overview of key issues in the development of monitoring and maintenance plans. Incorporation of adaptive management as a component of operations is included as a possible approach to maintenance and operation of the project. The user is also directed to the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) National Water Quality Monitoring Handbook for additional detailed information on setting up monitoring plans.

654.1601 Introduction

(a) Relationship of design and implementation to maintenance and monitoring

Maintenance and monitoring are the actions intended to ensure that the objectives of the stream restoration project are attained. Because project objectives, design criteria, and project site conditions determine the specifics of the actual project, maintenance and monitoring plans should follow from the opportunities, constraints, and requirements identified in the planning and design phase. Completion of the construction phase leads to the period of initial project operations when the stream restoration and streambank structures begin to function as designed.

Maintenance is the collection of actions taken to ensure that the stream restoration project performs as designed and to attain project objectives. It ensures the continued functioning of the structures and management measures once they are in place. While projects should be designed so they need a minimal amount of maintenance, some can be required especially in response to extreme flow events.

Monitoring is the process of measuring or assessing specific physical, chemical, and/or biological parameters of a project. Monitoring of any project, whether it is in the channel, streambank, riparian area, and/or adjacent lands, is necessary to ensure that the project is performing as intended. The parameters to be monitored should be directly related to the performance of the project and are linked to the goals and objectives of the project. This is also sometimes referred to as the hypothesis statement, or key questions to be answered.

The monitoring results may identify performance failures and inefficiencies requiring project modifications and changes to structures or operational practices. Performance objectives established for the project allow comparison of monitoring results to identify potential changes that may be needed in response to these performance problems.

(b) Maintenance and monitoring requirements resulting from project objectives

NEH654.02 describes the goals, objectives, and risks of project development. Maintenance and monitoring requirements can be identified initially from examining those actions required to meet the objectives of the project. Table 16–1 compiles different objectives identified in some typical NRCS stream projects. The objectives cited are taken from documentation on various projects. Some of the projects outlined in this table are described in more detail in the case studies of this handbook.

The terms goal and objective are sometimes used interchangeably. However, there are some distinct differences. Project goals are typically defined as the overall desired outcome, such as "restore channel to preflood conditions." Objectives are the more detailed, focused outputs or outcomes that achieve the project goals. The goal of restoring a channel to preflood conditions

Table 16–1 Project goals and objectives

Project	Goal	Objectives
Rose River, VA	Restore channel to preflood condition	Restore the hydrologic function [capacity] of the river by removing large cobble and debris bar that constricts the flood plain Stabilize streambanks Provide safe access for children to fish, stable cattle and tractor crossings
Little Blue River, KS	Remedy large-scale streambank erosion	Reduce excess stream sediment Improve stream channel dimension, pattern, and profile Establish a riparian ecosystem Improve terrestrial habitat Improve water quality Reduce nutrients and chemical pollutants
Rapidan River, VA	Restore the hydrologic function of the stream.	Get the water into one channel, not braided and shallow, that matched the preflood geomorphic dimensions
Goode Road/ Cottonwood Creek, Hutchins, TX	Stabilize banks	Reduce peak flows at lower elevation to protect bridge
Chalk Creek, Summit County, UT	Prevent erosion and reduce runoff of sediment	Protect water quality for cutthroat trout and overall health of the watershed
Red River Basin, ND	Restore riparian zones and stabilize stream channel and banks	
Big Bear Creek, PA	Stabilize channel and banks and improve aquatic habitat	Stabilize channel and banks using soil bioengineering and revegetation, stopping scour at a bridge Restore 3.7 miles of stream to a high-quality, cold-water fishery dominated by native brook trout

may have objectives such as increase hydraulic capacity by removing flood plain constrictions, stabilizing streambanks, and providing safe access for children to fish, and for stable cattle and tractor crossings (Rose River Restoration NRCS–VA). By attaining the project goal to restore the channel to preflood conditions, these objectives are achieved.

To obtain the goals and objectives of the project, potential maintenance and monitoring actions are suggested in table 16–2. This table lists two example projects showing the requirements that could be deduced from the objectives.

Distinctions between maintenance and monitoring requirements are not made in this section. The important point is to understand what parameters should be accounted for so that objectives are met.

Table 16–2 Project objectives and maintenance and monitoring actions

Project	Objectives	Maintenance or monitoring action or evaluation
Rose River, VA	Improve stream channel dimension, pattern, and profile	Channel profiles
	Stabilize streambanks	Channel planforms comparison to preflood conditions
Chalk Creek, Summit County, UT	Protect water quality for cutthroat trout	Survey of quantity/quality of rearing, spawning, and cover habitat
		Monitor habitat requirements, (stream water temperature, bed-material composition, water depth, and velocity)

654.1602 Development of monitoring plans

Monitoring helps determine whether the project is functioning as intended. Monitoring reveals the need for adjustments to design, construction procedures, and management actions. The information collected from monitoring should be made available to landowners and private interests who can make use of the information. Monitoring parameters are components of a project to be assessed that are evaluated to determine whether project objectives are being met (Washington Departments of Fish and Wildlife, Transportation, and Ecology 2003). Monitoring may be performed for a number of purposes such as (adapted from Federal Interagency Stream Restoration Working Group (FISRWG) 1998; North Carolina Stream Restoration Institute and North Carolina Sea Grant 2003):

- Performance evaluation: determine if the stream project design and management measures are functioning properly.
- Ecological or biological assessment: determine if biological resources are responding to altered conditions.
- Trend assessment: evaluate changing environmental conditions by long-term sampling at various spatial and temporal scales.
- Risk assessment: identify causes and sources of physical, chemical, or biological impairment or uncertainties that will affect operation of the project.

(a) Monitoring parameters

Physical, chemical, biological, and off-project parameters are identified to make up the monitoring plan. These parameters should be linked to the objectives, hypotheses, or key questions being tested with monitoring. Measurements of the selected parameters measure the performance and indicate the ecological functioning of the project.

Physical parameters
Physical parameters are the geomorphic and topographic features that compose the channel bed, streambank, and adjacent riparian areas. Table 16–3, adapted from FISRWG (1998), identifies the physical attributes (cross-sectional profiles and specific parameters for the attribute).

Chemical parameters
Improvements in water quality are a primary objective of many stream restoration projects. Stabilization of channels and streambanks results in reduction of sediment loading and movement. Land use changes in the stream corridor affect nutrient and chemical constituents in the water. Table 16–4 lists potential chemical parameters that could be affected by a project (adapted from FISRWG 1998).

Biological parameters
Aquatic and terrestrial communities change in character and abundance as the channel and streambank are stabilized. Streamside vegetation provides habitat and connectivity to other habitats or adjacent riparian areas. The biological attributes (table 16–5 (adapted from FISRWG 1998)) are the communities, structural components, and processes (primary production) that indicate the biological functioning of the stream system.

Off-project parameters
Project monitoring and maintenance focuses on the processes, systems, and impacts to the specific project and adjacent riparian areas. The watershed, upstream, and downstream processes and impacts were likely examined when formulating designs, accounting for the out-of-study area, or off-project constraints to design and construction. Monitoring and maintenance plans should consider off-project actions such as urbanization and other land use changes, sediment loading, and water control actions (detention structures). This investigation of off-project considerations could identify important parameters such as amount of urbanization or additional water users that should also be monitored.

Table 16–3 Physical parameters for consideration in monitoring

Physical attribute	Parameter
Plan view	Sinuosity, width, bars, riffles, pools, boulders, logs
Cross-sectional profile	Bank repose angle Depth bankfull Width Width-to-depth ratio
Longitudinal profile	Bed particle size distribution Water surface slope Bed slope Pool size/shape/profile Riffle size/shape/profile Bar features
Classification of existing streams	Varies with classification system
Assessment of hydrologic flow regimes through monitoring	2-, 5-, 10-year storm hydrographs Discharge and velocity of baseflow
Channel evolutionary track determination	Decreased or increased runoff, flash flood flows Incisement/degradation Overwidening/aggradation Sinuosity trend-evolutionary state, lateral migration Increasing or decreasing sinuosity Bank erosion patterns
Corresponding riparian conditions	Saturated or ponded riparian terraces Alluvium terraces and fluvial levees Upland/well drained/sloped or terraced geomorphology Riparian vegetation composition, community patterns and successional changes
Corresponding watershed trends—past 20 years and future 20 years	Land use/land cover Land management Soil types Topography Regional climate/weather

Table 16–4 Chemical parameters for consideration in monitoring

Chemical attribute	Parameter
Water clarity	Turbidity
Constituents	Dissolved and suspended solids
	Nutrients
	Toxins—natural and manufactured
Organic loading	Biological oxygen demand
Oxygen capacity	Dissolved oxygen
Water quality measures	Temperature
	pH
	Alkalinity/acidity
	Hardness

Table 16–5 Biological parameters for consideration in monitoring

Biological attribute	Parameter
Primary productivity	Periphyton
	Plankton
	Vascular and nonvascular plants
Zooplankton/diatoms	Species
	Numbers
	Diversity
	Biomass
	Macro/micro-organisms
Fish community	Anadromous and resident species
	Specific populations or life stages
	Number of out-migrating smolts
	Number of returning adults
Riparian wildlife/ terrestrial community	Amphibians
	Reptiles
	Mammals
	Birds
	Plants (invasive species)
Riparian vegetation	Structure
	Composition
	Function
	Changes in time (succession, colonization, extirpation)
Habitat structure	Spawning gravel
	Instream cover
	Shade
	Pool/riffle ratio
	Amount and size distribution of large woody debris

(b) Types of monitoring

The types of monitoring corresponding to the purposes of monitoring are:

- ecological or biological monitoring
- performance monitoring
- trends monitoring
- risk monitoring

For stream restoration projects, performance and ecological or biological monitoring are most important and are included here. While trends and risk monitoring may be very important, a complete discussion is beyond the scope of this chapter. More information is available in FISRWG (1998).

(c) Monitoring strategies

The strategy for collecting and using monitoring data is determined by:

- project type and constituent project structures and nonstructural features (Design documents for the project and restoration (Fischenich and Allen 2000; Johnson, Pittman, et al. 2001) provide information on potential project design features and parameters for monitoring.)
- monitoring parameters identified as important for the project's performance
- type and purpose of monitoring
- resources available (funding) for monitoring

Low effort monitoring
All stream restoration projects require some inspection and monitoring to ensure performance and identify problems and unexpected occurrences. Agency funding for projects often ends with construction, and postimplementation or operations funding is dedicated to maintenance costs. Several low-effort or low-tech monitoring strategies are possible:

- Site visits to future ongoing planning or construction projects can incorporate side trips to the project site. A simple windshield survey and walk-through observation trip can provide valuable monitoring information.
- Local volunteer watershed and conservation organizations are often willing to include monitoring as part their activities. Training of the groups to perform the monitoring and reporting is necessary, but these groups can become valuable eyes and ears for the project (U.S. Environmental Protection Agency (EPA) 1997).

Photographic record
Photographic documentation may provide the information needed for monitoring and is a cost-efficient strategy. For monitoring, repeat photography taken from the same location provides a visual record of changing conditions for soils and vegetation (Hall 2001). However, for a qualitative comparison, it is best to compare photos taken during the same season. This is especially important if vegetation is a major project component. Photographic monitoring is especially suited for documenting success of vegetative plantings, impacts of humans and livestock, and changes in channel gradient and bankline stability (Governor's Watershed Enhancement Board 1993). Monitoring for a project is determined by the objectives and parameters identified in the monitoring plan. Monitoring with photography requires:

- determining specific objectives
- using a repeatable technique
- choosing appropriate camera and media (digital and film)
- developing a filing system (Hall 2001)
- establishing fixed and permanent reference points so that photographs can be compared over time

The frequency for acquiring photography is determined by the parameters monitored. Videography and other remote sensing image acquisition may also be an efficient mode of collecting this information.

Monitoring programs and surveys
A monitoring plan establishes the details of the program for measurement of the selected parameters. The monitoring plan includes (adapted from Washington Departments of Fish and Wildlife, Transportation, and Ecology 2003):

- statement of monitoring objectives
- hypotheses or key questions to be answered

- monitoring parameters
- monitoring protocol
- analysis and use of the information

Statement of monitoring objectives—Identifies the purpose and type of monitoring (project performance) and the project objective (improvement in channel stability, bank stability, and improvement in natural conditions). The direct connection of project objectives to monitoring objectives enables monitoring results to be used for modifying designs and operations to better achieve project goals.

Monitoring parameters—Measurement of the parameters identified as being important for monitoring should be specified in the monitoring plan. Monitoring intensity and evaluation techniques are specified for each of the parameters.

Monitoring intensity—Refers to the level of detail required in the monitoring process; that is, the level of detail required for making decisions on the monitoring objectives (Washington Departments of Fish and Wildlife, Transportation, and Ecology 2003). In some cases, yes/no or good/fair/poor responses provide sufficient information. In other cases, quantitative measurements and modeling are needed to answer monitoring questions. Differences between qualitative and quantitative methods (time, analytic requirements) may determine the level of detail possible within project constraints.

Evaluation techniques—The types of analysis methods used (cross-sectional survey) to monitor the selected parameters. The field sampling methods may differ regionally and for species or habitats. Information for field sampling methods should be identified or developed by NRCS regional engineering and natural resources personnel and project design personnel. This approach ensures that the monitoring will reflect the study area conditions.

Monitoring protocols—For each parameter that has been selected to be monitored, a protocol for implementing the evaluation technique is described. As with evaluation techniques, protocols should be identified or developed by those experienced with local or regional conditions. Protocols normally include (Washington Departments of Fish and Wildlife, Transportation, and Ecology 2003):

- specification of methods and geographic extent of measurements
- identification of monitoring period and frequency
- design of monitoring forms for data collection
- description of data-analysis techniques

654.1603 Developing plans for maintenance

Maintenance of constructed projects ensures that the project operates or performs as intended. Maintenance requirements depend on the project type and level of risk, project goals, and level of effort or resources available for maintenance. The important categories of maintenance are (Martin and Fisher 2002):

- hydrology
- geomorphology
- vegetation
- domestic animals/livestock
- wildlife
- people

Types of maintenance are (FISRWG 1998)

- scheduled maintenance
- remedial maintenance
- emergency maintenance

(a) Project type requirements

Channels and flood plains
Projects establish or restore stability to channels and flood plains. The maintenance of design conditions for hydrology and streambank and flood plain stability often requires scheduled maintenance of project features. Project objectives often include sustainability of the system (FISRWG 1998). Establishment of a dynamic equilibrium requires less extensive maintenance efforts than objectives for maximum hydraulic capacity or other objectives requiring more extensive and frequent maintenance actions. The maintenance requirements specific to the structures, materials, and construction methods can be identified by examining the design documentation and local project conditions for the project. Table 16–6 contains lists of parameters in the stream corridor that could be considered for maintenance.

Protection/enhancement measures
Management measures (structures, vegetation, management actions) that protect streambanks, deflect flows, and improve habitat conditions require periodic maintenance. Failure of the measures after construction should be evaluated to determine if the design or construction method should be altered, rather than just repaired (FISRWG 1998).

Table 16–7 contains a list of possible maintenance actions that may be required for specific protection and enhancement features.

Boulders and other instream features should be maintained to ensure proper functioning. Revetments and heavy or hard protection features require inspection and potential repair and addition of materials. The vegetation of soft protection systems requires inspection to determine survival and level of protection from the vegetation. Hybrid measures, using vegetation in combination with geogrids, geotextiles, and cellular blocks, require maintenance of structural components such as loss of geotextile material and replacement or replanting of vegetation (Fischenich and Allen 2000). The intent of some streambank and channel features is to provide temporary stabilization until riparian vegetation develops and establishes more stable channel bank conditions, so that maintenance of protection/enhancement features will become less important over time (FISRWG 1998).

Vegetation
After construction, monitoring should be frequent enough to evaluate how vegetation establishment progresses (Winward 2000). Many projects that rely on soil bioengineering require that the vegetation become firmly established before experiencing a significant flow event. If a significant event occurs before the vegetation is established, replanting may be necessary. If replanting, protective measures, or irrigation are needed after construction, these actions can be undertaken to ensure that vegetation is established in sufficient abundance and distribution. After establishment of vegetation, project operation requires that it be maintained in a specified abundance and location to achieve project objectives, but not become excessive enough to interfere with water, sediment, or wildlife movement. Maintenance requirements range from mowing of terraces to clearing of excess woody debris, depending on the vegetative component. The

Table 16–6 Maintenance actions for channel and flood plain projects

Project location	Maintenance actions
Channel	Structures—repair of: Grade control—rock, concrete, Weirs Rock vanes Island and bar preservation, development Bank toe stabilization—rock, vegetation Rock barbs Removal of: Nuisance aquatic vegetation Woody debris accumulation
Flood plain	Repair or reformation of bank grading Actions to address encroachments Maintaining planned boundaries and conditions for rights of way Replanting or adding new vegetation due to poor establishment or lack of survival
Buffer strips, setbacks, easements	Establishment of boundaries after encroachments by adjacent land uses
Meander bends	Stabilization of eroding or unstable banks Seeding of newly formed areas

Table 16–7 Maintenance actions for different protection/enhancement features

Protection/enhancement features	Maintenance actions
Streambank stability	Repair bank armoring structures (stone filled revetments, soil-covered riprap, cellular blocks, geogrid, gabions, geotextile fabrics, soil cement, bulkheads) Terrace zone—seeding, vegetation establishment, mulching
Stream/habitat features	Repair, replacement, expansion of fish cover structures Repair, replacement of pool/riffle rocks and structures
Vegetation	Removal of excess woody vegetation Repair, maintain irrigation, water availability Replanting, replacement of trampled, dead, or impaired vegetation Maintain, repair, and replace fencing, signage, and barriers for vegetation protection Repair or replacement of brush mattress, matting, or other soil bioengineering materials Seeding or reseeding Mulching for plant and soil stability
Access and human use structures	Clearing of access pathways for humans and livestock Cleaning and repair of recreation structures—picnic tables, boat ramps, parking areas Cleaning and repair of restroom facilities

vegetation section of table 16–6 identifies potential actions to consider in vegetation management.

Access and human-use features
Many projects incorporate access points for human uses such as fishing, wildlife access, and agriculture (cattle). Recreation boat ramps, picnic areas, and restroom facilities can be incorporated into projects, increasing public use and the value of the project. These human-use features require a higher level of maintenance to meet public expectations. Exceeding the carrying capacity of the resource by too many visitors or livestock can lead to degradation and erosion of streamside lands and excessive inputs of nutrients and pollutants to the channel. Table 16–6 identifies maintenance activities for the access and human-use features.

(b) Maintenance considerations

Project-specific factors should be considered in planning for maintenance. The necessary maintenance activities for project design features are modified in light of risk, project goals, and level of effort.

Risk
In considering project monitoring and maintenance, risk pertains to the probability of project failure if maintenance is not performed. Numerous circumstances, from budgetary to natural events such as flooding, can prevent maintenance from occurring. Project planners must evaluate how susceptible a project design is to risk of failure if maintenance does not occur, is reduced in scope, or delayed. Projects that rely on structural features may be at less risk than projects dependent on natural or biological components (vegetation maintenance).

Project goals
Project goals and objectives require that maintenance activities be performed to achieve the levels of hydrologic and environmental outputs. Success or performance criteria may be developed for the objectives, specifying the quantities and levels required for project functioning. These criteria help in identifying the maintenance intensity and evaluation techniques required.

Level of effort
The level of effort or available resources (funding, equipment, labor) for maintenance should be considered in design and planning for maintenance. Maintenance plans should reflect the available personnel and other resources.

654.1604 Monitoring and maintenance plan documentation

Preparation of a plan for monitoring is similar to a plan for maintenance. In fact, these two activities are often considered together as part of the plan documentation. Plans for maintenance and monitoring are developed from goals and objectives (NEH654.02). The steps for plan preparation presented here are adapted from Components of a Monitoring Plan, Part 6B of the FISRWG (1998). The guidelines set out three steps for preparing a plan:

- project planning
- implementing and managing the project
- responding to monitoring results

(a) Planning

There are seven steps for planning a monitoring plan.

Step 1 Define the stream restoration project goals and objectives.

Restate the goals and objectives identified as part of project planning (NEH654.03).

Step 2 Develop a conceptual model of the stream, flood plain, and watershed.

A conceptual model serves to communicate relationships of water, geomorphic conditions, and biota (Henderson and O'Neil 2005). The model can be used to identify changes and impacts in the system.

Step 3 Choose performance criteria.

Performance criteria are standards to evaluate to what extent the project is achieving desired or designed outcomes. The performance criteria identify in quantitative terms (defined metrics) or qualitative terms (absence/presence) the results or outcomes of project operation. The Federal Guidelines (FISRWG 1998) provide three components for choosing performance criteria.

Link performance to goals—Goals and objectives for the project should articulate the specific outcomes and results that are expected and intended from the project. The hydrologic, geotechnical, and ecological needs and opportunities identified in planning should have resulted in clear statements for project performance. Performance criteria are meant to assess progress toward the goals. If the goals and objectives are not clear enough for identifying performance criteria, then clarification, interpretation, or explanation of the goals and objectives must be done. The effort to understand or clarify goals will allow establishment of performance criteria that are closely aligned with stated goals.

Develop the criteria—The primary reason for a maintenance and monitoring plan is to assess progress and to indicate the steps required to fix a system or component of the system that is not successful (FISRWG 1998). To that end, the performance criteria and monitoring parameters should be developed as indicators of success. Performance criteria are usually developed through an iterative process that involves listing measures of performance relative to goals and then refining them to develop the most efficient and relevant set of criteria (FISRWG 1998). Criteria are usually specified as levels of outputs (hydraulic capacities, ranges, minimums, maximums, or threshold measurements).

Maintenance performance criteria—Structural, vegetative, and management measures (such as grazing controls) are incorporated into stream restoration project designs because they provide the desired project outputs in terms of necessary hydraulic capacities, levels of protection, and habitat benefits. The necessary maintenance actions are determined by the requirements of the measures. Information from the design phase can be used for maintenance performance of structural components (design sizes, capacities). Maintenance of natural resource and vegetative components is influenced by design requirements, such as level of protection, and by natural conditions. Maintenance of management measures requires identifying the actions, such as repair of fences, needed so that the management measure functions properly (tables 16–6 and 16–7).

Monitoring performance criteria—Performance criteria for the monitoring plan establish the acceptable or desired levels for the parameters being monitored. The performance criteria are based on comparison of the parameter's measurement

to the agreed on performance criteria. The monitoring parameters identified (tables 16–3 through 16–5) are measured in the field and compared to performance criteria.

Identify reference sites—Reference sites are channel study areas that are similar to the project channel, but not in need of stabilization. These sites represent the study area if it were undisturbed or stable. Figure 16–1 shows the proximity of the Teton River reference site to the Fox Creek restoration site in Idaho. Conditions (hydrologic, geomorphic, habitat) at the reference site represent the conditions that are the goals of the project. By examining the conditions at the reference site, the study team can ascertain the level of success that is possible from the project. Pre- and postconstruction evaluations can measure the change or impact from the project, but the level of success can be judged only relative to reference systems (FISRWG 1998).

Step 4 Choose maintenance and monitoring parameters and methods.

The purpose of maintenance and monitoring is to ensure the project performs the hydrologic, geomorphic, and habitat functions that are the basis of goals and objectives and project design.

Monitoring parameters—

- Table 16–8 (FISRWG 1998) identifies general project objectives and potential evaluation tools and criteria. As pointed out, the goals and objectives lead to identification of particular parameters for monitoring. Tables 16–3 through 16–5 contain more complete lists of parameters.
- The National Research Council (1992) recommends that parameters include physical, hydrological, and ecological measures. In this way, a holistic assessment of the stream and flood plain is possible. Using reference sites, pub-

Figure 16–1 Fox Creek, ID, case study reference site

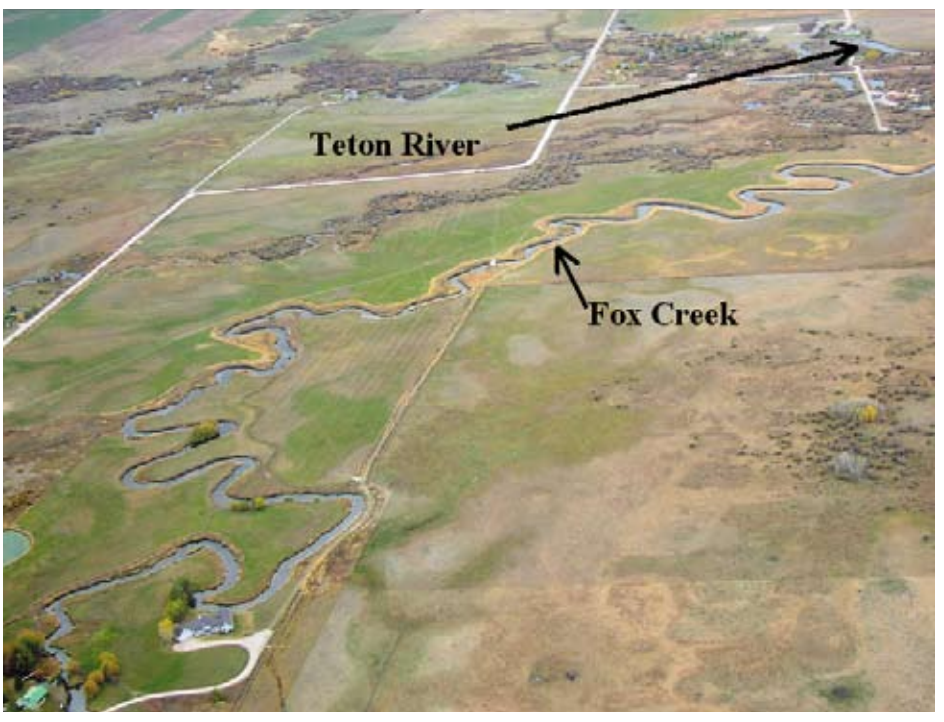


Table 16–8 General project objectives and potential evaluation tools and criteria

General objectives	Potential evaluation tools and criteria
Channel capacity and stability	Channel cross sections Flood stage surveys Width-to-depth ratio Rates of bank of bed erosion Longitudinal profile Aerial photography interpretation
Improve aquatic habitat	Water depths Water velocities Percent overhang, cover, shading Pool/riffle composition Stream temperature Bed-material composition Population assessments for fish, invertebrates, macrophytes
Improve riparian habitat	Percent vegetative cover Species diversity Size distribution Age class distribution Plantings survival Reproductive vigor Wildlife use Aerial photography
Improve water quality	Temperature pH Dissolved oxygen Conductivity Nitrogen Phosphorous Herbicides/pesticides Turbidity/opacity Suspended/floating matter Trash loading Odor
Recreation and community involvement	Visual resource improvement based on landscape control point surveys Recreational use surveys Community participation in management

lished literature, and applicable standards, performance criteria are identified or developed for the physical, hydrological, and ecological monitoring parameters.

- The effects of watershed activities on the project should be considered when identifying parameters. Activities in the watershed can affect the success of the project and cause changes in monitoring parameters not related to the project. Land use changes in urban settings produce changes in water runoff and movement that should be accounted for in monitoring plans. Rural areas similarly undergo land use changes, but the changes are usually slower to take place.
- A holistic view of stream conditions should be pursued using the minimum necessary measurements. While comprehensiveness and redundancy may be desirable, this may be costly and unnecessary.

Monitoring methods—Protocols for monitoring the parameters identified as important are either identified from available sources or developed to meet the channel and regional conditions. Covering sampling and analysis methods for the range of potential monitoring parameters is beyond the scope of this chapter.

Monitoring profiles—Monitoring plans require establishing the physical location from which parameters are measured. Depending on the monitoring parameters needed to determine performance, monitoring is undertaken from a cross-sectional profile, longitudinal profile, or from bankline surveys. However, while it should be noted that other survey techniques and protocols are in use, the ones described herein are the most common.

Channel cross-sectional profile—The channel cross-sectional profile is typically used to monitor bank and channel morphology. The cross section is located across the stream perpendicular to the direction of stream flow. The cross section is used to measure bank and channel elevations, referenced to a benchmark over time. In this way, stability or changes of the bank and channel location can be determined. The channel cross section is used for projects with objectives for stabilizing meandering channels; consolidating multiple, shallow, or braided channels; establishing stable near bank habitat areas; or stabilizing channel slopes.

The cross-sectional survey involves placing endpoints and a benchmark on the stream terrace or stable flood plain, establishing sampling points, taking documentary photographs, and measuring elevations with a surveyor's level (Harrelson, Rawlins, and Potyondy 1994). At least 20 elevation measurements are usually taken at significant breaks of slope that occur across the channel. The active terrace and flood plain may be included in the cross section, dictated by the project and project objectives. Resulting information produces a channel cross section as in figure 16–2 (adapted from Harrelson, Rawlins, and Potyondy 1994). Channel slope can be determined by taking additional elevation measurements upstream and downstream from the cross sections and calculating the changes in slope. In this way, a survey plot of the stream channel and features can be developed (fig. 16–3 (adapted from Harrelson, Rawlins, and Potyondy 1994)).

As elevation measurements are taken, sampling for chemical attributes, sediment, and some biological attributes such as habitat structure (table 16–5) can be obtained at the same time. Harrelson, Rawlins, and Potyondy (1994) provide guidance on basic surveying techniques.

Longitudinal profile—The longitudinal profile establishes how the stream and flood plain change in elevation as the stream flows through the study reach. The slope is determined by successive measurements of water surface, channel bottom, bankfull stage, flood plain, and terraces. Most biological attributes (table 16–5) have distribution, size, or density dimensions requiring measurement over an area, not just a cross section. Establishing longitudinal monitoring sample locations is appropriate for projects that stabilize headcutting, restore riparian vegetation or aquatic habitat, and for some erosion protection projects.

Establishment of a permanent longitudinal profile for monitoring requires identifying a permanent location that has the project features (vegetation) that are important to monitoring. The longitudinal profile should encompass an area 300 to 500 feet along the stream (or approximately 20 times the channel width at bankfull). The survey should be wide enough to measure both banks,

Figure 16–2 Diagram of a cross-sectional survey

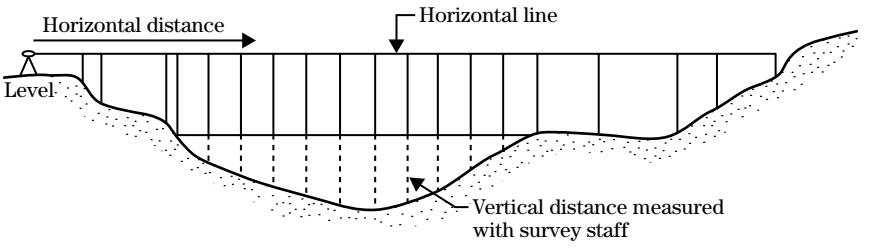
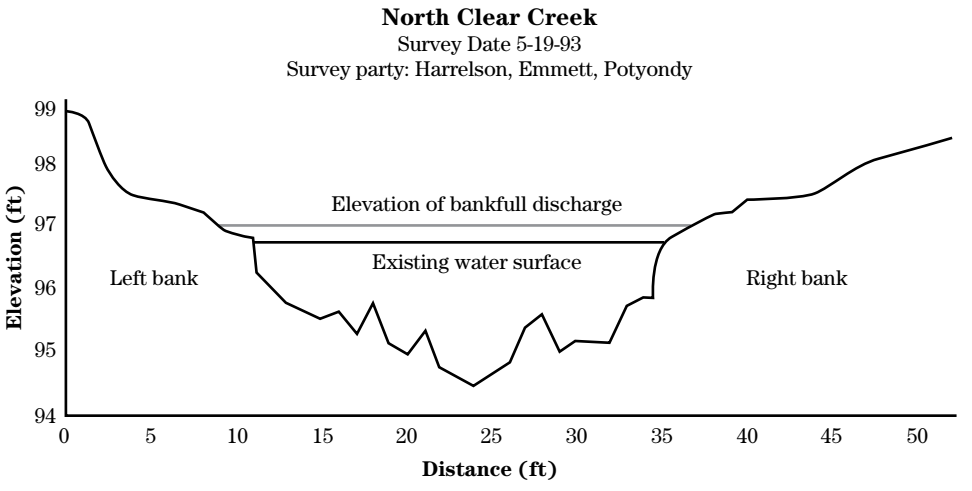


Figure 16–3 Final plot of cross section



the active flood plain, and one or more terraces. A benchmark is established and stations are located at intervals along the longitudinal profile. A surveyor's level is used to measure the elevations of the channel bottom, water surface, terrace, and flood plain. Plotting of the elevation data along the longitudinal profile results in a plot of the slopes (fig. 16–4).

Bankline surveys—The objectives of some projects are to define or stabilize the bankline, provide capacity for a certain bankfull discharge, or maintain a consistent bank width between reaches. For these projects, bankline surveys are used to monitor change in the bank position, determining if the width between the banks is consistent over time and consistent between reaches of the stream. Bankline surveys are implemented by establishing permanent points along the bankline to be measured over time. In low-gradient, meandering systems, the flood plain is well defined and bankfull stage is clearly marked. Where the flood plain is absent or poorly defined, it may be necessary to establish benchmarks or natural indicators as surrogates, such as vegetation, for the top of bank (Harrleson, Rawlins, and Potyondy 1994). The locations of the bankline reference points are documented and physically benchmarked. The bankline's lateral extent into the flood plain and bank-to-bank width are measured and documented over time.

If bank location and width between channel reaches are of concern, a series of reference bankline points are identified. Monitoring the reference points over time will identify changes in location and width of the bankline along the stream. Changes in width between reaches indicate that the cross sections are changing between sections, and the cause or source of these changes should be identified.

Step 5 Estimate costs.

Costs for maintenance and monitoring plans include:

- personnel and management costs to implement plan
- quality assurance
- data management
- field sampling
- data analysis and interpretation
- maintenance and monitoring report preparation
- presentation of results and recommended changes

Figure 16–4 Graphic representation of longitudinal profile

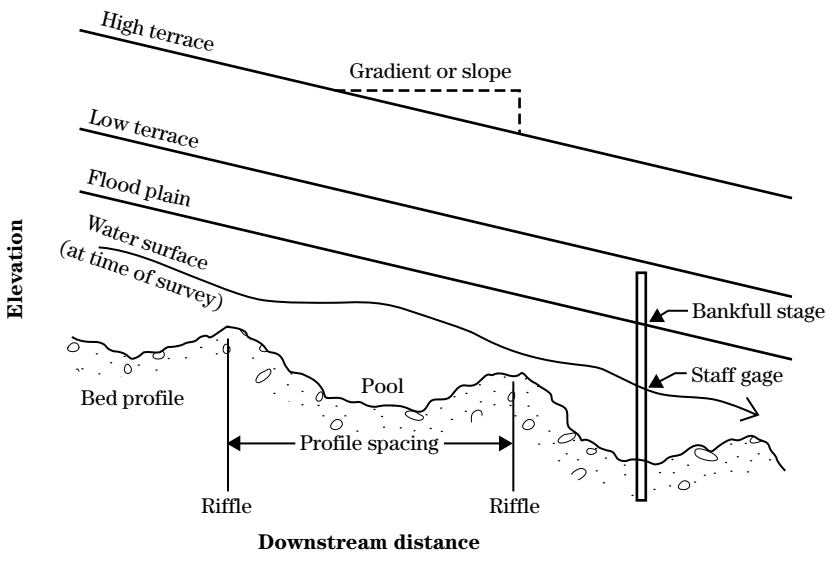


Table 16–9 categorizes the relative costs of monitoring and maintenance as high, medium, and low. Costs vary with region and the intensity and frequency of monitoring and maintenance.

Step 6 Categorize the types and emphasis for data.

The emphasis on data collection changes as the project progresses from planning to implementation to postimplementation.

Planning—

- develop baseline data at the site

Implementation of project—

- monitor construction and management activities
- collect as-built and as-constructed information

Postimplementation—

- collect performance data
- conduct other studies as needed

Step 7 Determine the level of effort and duration.

The level of effort needed for maintenance and monitoring is determined by the goals, objectives, and performance criteria identified in step 3. Maintenance for structures and vegetation is required to meet goals of the project in terms of stabilization, channel capacity, and environmental considerations (FISRWG 1998). The level of effort for monitoring is determined by the goals and objectives and by the end use of the monitoring data. Water quality monitoring to determine whether state water quality standards are met requires a higher level of effort (frequency, data gathered) than monitoring for public access.

Frequency—Frequency of maintenance actions is determined by the type of structure, vegetation, or management measure that is part of the project. Ensuring the proper functioning of the design requires development of maintenance schedules by the responsible agency personnel. For monitoring of physical, chemical, and biological parameters,

Table 16–9 Relative costs of monitoring actions

Type of monitoring and maintenance	Relative costs
Photographic monitoring	Moderate
Windshield survey monitoring	Low
Volunteer monitoring	Low
Cross-sectional profile	High
Longitudinal profile	High
Bankline survey	High
Track watershed trends	Moderate
Chemical parameter monitoring	Low
Biological monitoring	
Primary productivity	Moderate
Fish community	Moderate
Riparian vegetation	High
Habitat structure	High

a sampling plan is developed for the parameter or group of parameters (water quality parameters). Frequency of maintenance and monitoring may decrease as the project and system becomes more established and rates of change decrease. For instance, monitoring of a project may be done annual for the first 3 years, followed by monitoring at 2- to 5-year intervals for the duration of project life.

Duration—Maintenance and monitoring should extend long enough to determine either:

- reasonable assurances of sustainability of the project
- that the system has met performance criteria
- that the system will not likely meet the criteria

Timing—Timing of maintenance activities is important so that structural and vegetative components remain functional. With designs that incorporate soil bioengineering approaches and vegetation, the period after construction is critical to establishment and success of the measures, so higher levels of maintenance and monitoring are required in the immediate postconstruction period. In the winter periods, vegetation and other conditions may not be relevant to the performance criteria and project objectives. Monitoring should be carried out during the time of the year when vegetation and streamflow conditions approximate the conditions used for design.

Sensitivity—The sensitivity of the parameter to change will also determine the level of effort and duration needed to detect a change. In some cases, this may require some statistical analysis. If this is required, it may be appropriate to consult a statistician during the design of the monitoring plan.

(b) Implementing and managing monitoring

Planning for maintenance and monitoring occurs while site design and construction plans are underway and there is normally a great deal of activity. Following construction and beginning of operation of the project, it is important that the monitoring plan is implemented successfully. This takes deliberate effort by the operating agency or authority, with the emphasis then on project operation. Consult the Interagency Guidelines (FISRWG 1998) for suggestions and insights for implementing and managing the monitoring plan.

(c) Responding to monitoring results

Monitoring provides information on performance, biological resources, trends, and risks. The monitoring information serves as the basis for making modifications to the project and operations. The adaptive management section below presents a process for responding to monitoring results. If long-term water quality monitoring shows that the performance criteria are met, then water quality can be considered for deletion from future monitoring. If biological parameters show a lack of nonjuvenile fish, then a fisheries investigation may be indicated. If performance monitoring indicates that performance criteria are not being met, an investigation of the cause should be initiated. This may lead to modification of the project or to the identification of changing conditions within the watershed.

654.1605 Adaptive management

(a) Background for adaptive management

Stream restoration and/or stabilization projects are part of dynamic systems, and over time the project outcomes will likely change (increase or decline) as project life increases. Aquatic habitat, streambank and riparian communities, water flow capacity, and other project conditions may show improvements or deteriorations. Sometimes, there is uncertainty on the point of sustainability (equilibrium), relative to sedimentation, streambank location, or habitat diversity. The prevailing assumptions on which project objectives are based may prove to be erroneous. System relationships and connections may be weak or nonexistent, watershed and local conditions may change, or project design measures may be overkill, and lesser levels of structures, maintenance, or human interference may be called for. The uncertainty in project outcomes and the need for change in project design, operation, and management have given rise to adaptive management. Adaptive management is an approach to natural resource management that incorporates monitoring of project outcomes and uses the monitoring results to make revisions and refinements to ongoing management and operations actions (adapted from National Academy of Science 2002). Figure 16–5 shows the relationship of adaptive management to monitoring, construction, and planning.

(b) Maintenance program and adaptive management

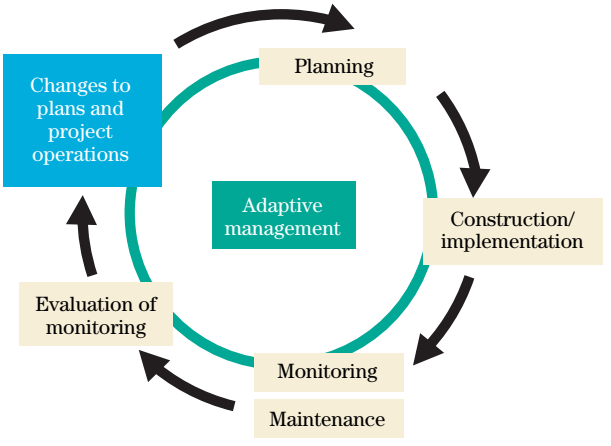
The maintenance and monitoring plans described result in information on project performance (hydrological, geomorphic) and ecological (habitat, water quality) outputs of the projects. In a sense, the system composed of the operating stream restoration project is like an experiment, and the monitoring reports are the findings for the experiment. Assumptions may be proved or disproved, and understandings of relationships may change based on monitoring information. Adaptive management, therefore, incorporates an element of research into conservation projects. Specifically, it is the integration of design, management, and monitoring to systematically test assumptions to adapt and learn (Salafsky, Margoluis, and Redford 2001).

(c) Adaptive management and NRCS projects

The increasing number of projects that incorporate experimental designs and rely on hydrologic, geomorphic, and riparian processes for success has led to application of adaptive management in large and small systems (Save Our Bosque Task Force 2004). Examples of references to adaptive management are (USDA NRCS 1996a):

- Year two process will develop this concept further at the local decisionmaking level. The findings of the interrelationship of ecosystem components will be better understood and promoted for broader planning approaches (ecosystem, whole farm, holistic).
- Steering committee/subcommittee structure with regular meetings and technical input will allow for adjustments to reflect the results of periodic reviews and new scientific information or methodologies. The plan will be revisited regularly and modified as needed.
- Data collected as part of the project monitoring process will be analyzed. Results will be shared to determine if further study is warranted.

Figure 16–5 Adaptive management



(d) Adaptive management in the maintenance and monitoring process

Adaptive management is a part of project operations, most closely related to monitoring. Separate adaptive management programs may be established for larger systems (Raynie and Visser 2002). For most stream restoration projects, reviews of monitoring and inspection information will likely initiate changes in monitoring, maintenance, and operations.

Adaptive management evaluation
Monitoring information is reviewed to answer the following questions (FISRWG 1998):

- Were the project structures, vegetation, and management measures constructed and implemented correctly?
- Did the project measures achieve the desired goals and objectives of the project?
- Are the assumptions used in the project design and cause-effect relationships correct?

Changes resulting from adaptive management evaluations
Revision of project operations, monitoring, and maintenance procedures are identified through adaptive management evaluation. These changes should be incorporated in the project maintenance and monitoring plans and in project operations documentation.

654.1606 Conclusion

Any open channel design work, whether it is a natural stream restoration or a single-purpose design, is done to achieve some specific planned goals or objectives. Maintenance and monitoring plans are often overlooked. However, these are important components of stream design and restoration projects. Monitoring plans ensure that a project is performing as designed and achieving the intended goals. All open channel projects carry some level of inherent risk to life, property and project investment, and monitoring, maintenance, and adaptive management can reduce these risks. Monitoring may also help to avoid catastrophic project failure by identifying problems or performance issues while they can be more cheaply addressed. Finally, the lessons learned from monitoring can be applied to future projects of similar type.

Maintenance is the set of actions taken to ensure that a project's goals or objectives continue to be met. Maintenance may involve the repair of specific project features in response to some damage or the periodic and/or scheduled actions. While projects should be designed to minimize maintenance requirements, the designer should consider what may be required and how it can be linked to the monitoring plan. An ideal maintenance and monitoring plan should provide specific parameters to be assessed to ensure that the project is performing as intended, as well as what maintenance actions should be undertaken.

Adaptive management is an approach to natural resource management that incorporates monitoring of project outcomes and uses the monitoring results to make revisions and refinements to ongoing management and operations actions.

United States
Department of
Agriculture

**Natural
Resources
Conservation
Service**

**Part 654 Stream Restoration Design
National Engineering Handbook**

Chapter 17 Permitting Overview

Issued August 2007

Cover photo: Federal, state, local, and tribal laws, ordinances, and permitting requirements must be thoroughly recognized and understood during the restoration planning process.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Chapter 17 Permitting Overview

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Chapter 17 Permitting Overview

654.1700 Purpose

Stream restoration and design activities are subject to various Federal, state, and local regulatory programs. Most of these regulations are aimed at protecting natural resources and the integrity of the Nation's water resources. This chapter provides a brief overview of the regulatory authorities and programs that may be applicable to stream design work. The focus of this chapter is to provide an awareness-level understanding of this important issue and to list sources where current information can be obtained. The reader should not interpret the information herein as the only source of regulatory requirements. Federal, state, and local regulatory authorities should always be researched and consulted as part of the initial planning and design efforts.

654.1701 Introduction

Every stream design or restoration effort is subject to regulatory requirements. Designers should be aware of project permitting requirements and develop a project plan and budget identifying resources and project approaches that meet permit conditions. Depending on the type of project and its location, these can range from minimal to a full set of required Federal, state, and local permits. The applicable programs and permits can include:

- National Environmental Policy Act
- Endangered Species Act
- National Historic Preservation Act
- Wild and Scenic Rivers Act
- Fish and Wildlife Coordination Act
- Clean Water Act
- Rivers and Harbors Act of 1899
- Magnuson-Stevens Fishery Conservation and Management Act
- Local and state water quality permits
- Water rights
- National Flood Insurance Program
- Local and state flood permits
- Local zoning permits

655.1702 Initiating a permitting process

Permitting agencies should be approached as soon as conceptual plans are developed. In regulatory-intensive areas, as well as in areas of high environmental risk, it may be advisable to consult with them in the early planning stages. In general, designers and planners should provide at least the following to the permitting agency:

- site map
- description of existing environmental conditions (written and maps, photos, drawings)
- description of the proposed work (written and drawings)
- property ownership
- access and staging information
- preferred times of implementation

Refer to permitting agency guidance (usually available online), or contact an agency representative for additional information on what to provide. Local planning agencies often are able to steer designers towards the applicable Federal and state agencies and requirements. Table 17–1 provides some examples (Federal Interagency Stream Restoration Working Group (FISRWG) 1998).

654.1703 U.S. Army Corps of Engineers Regulatory Program

One of the most prominent national permitting requirements is the regulatory program administered by the U.S. Army Corps of Engineers (USACE). The mission of the USACE Regulatory Program is to protect the Nation's aquatic resources, while allowing reasonable development through fair, flexible, and balanced permit decisions. Streams, wetlands, and other water features are important in protecting overall water quality, fish and wildlife habitat, flood storage, natural products, recreation, aesthetics, and navigation.

The USACE evaluates permit applications for most construction activities that occur in the Nation's waters, including wetlands. USACE permits are also necessary for any work in, or affecting the Nation's navigable waters, including construction and dredging. The USACE balances the reasonably foreseeable benefits and detriments of proposed projects and makes permit decisions that recognize the essential values of the Nation's aquatic ecosystems to the general public, as well as the property rights of private citizens who want to use their land. During the permit process, the USACE considers the views of other Federal, state, and local agencies; interest groups; and the general public. The intended result of this public interest review is fair and equitable decisions that allow reasonable use of private property, infrastructure development, and growth of the economy, while offsetting the authorized impacts to the waters of the United States. The adverse impacts to the aquatic environment are offset by mitigation requirements, which may include avoiding and minimizing impacts to, as well as restoring, enhancing, creating, and preserving, aquatic functions and values. The USACE strives to make its permit decisions in a timely manner that minimizes impacts to the regulated public.

The USACE administers and enforces the regulatory program over waters of the United States, including wetlands and navigable waters, through its Regulatory Program. The day-to-day administration of the Regulatory Program is accomplished through the USACE district offices. USACE offices for each zip code and points of contact within the USACE Regulatory Program are listed on the USACE Web site at:

http://www.usace.army.mil/inet/functions/cw/cecwo/reg/

Table 17–1 Examples of permit requirements for restoration activities

Local/state

Permits required	Activities covered	Administered by:
Varies—thresholds and definitions vary by state	Clearing/grading, sensitive/critical areas, water quality, aquatic access	Local grading, planning, or building departments; various state departments

Federal

Permits required			Activities covered	Administered by:
Section 10, Rivers and Harbors Act of 1849			Building of any structure in the channel or along the banks of "navigable waters" of the United States that changes the course, condition, location, or capacity	USACE
Section 404, Federal Clean Water Act	Letters of permission		Minor or routine work with minimum impacts	USACE
	Nationwide permits	3	Repair, rehabilitation or replacement of structures destroyed by storms, fire, or floods in past 2 years	
		13	Bank stabilization less than 500 feet in length solely for erosion protection	
		26	Filling of up to 1 acre of a nontidal wetland or less than 500 linear feet of nontidal stream that is either isolated from other surface waters or upstream of the point in a drainage network where the average annual flow is less than 5 ft^3/s	
		27	Restoration of natural wetland hydrology, vegetation, and function to altered degraded nontidal wetlands, and restoration of natural functions of riparian areas on private lands provided a wetland restoration or creation agreement has been developed	
	Regional permits		Small projects with insignificant environmental impacts	
	Individual permits		Proposed filling or excavation that causes severe impacts, but for which no practical alternative exists; may require an environmental assessment	
Section 401, Federal Clean Water Act			Water quality certification	State agencies
Section 402, Federal Clean Water Act National Pollutant Discharge Elimination System (NPDES)			Point source discharges, as well as nonpoint pollution discharges	State agencies
Endangered Species Act Incidental Take Permit			Otherwise lawful activities that may take listed species	USFWS

(a) Legislative history

The legislative origins of the USACE regulatory program are the Rivers and Harbors Acts of 1890 (superseded) and 1899 (33 U.S.C. 401, et seq.). The authority is granted to the Secretary of the Army. Various sections establish permit requirements to prevent unauthorized obstruction or alteration of any navigable water of the United States. The most frequently exercised authority is contained in Section 10 (33 U.S.C. 403). The references mentioned are available on the USACE Web site at:

http://www.usace.army.mil/inet/functions/cw/cecwo/reg/

Section 10 addresses construction, excavation, or deposition of materials in, over, or under such waters or any work that would affect the course, location, condition, or capacity of those waters. Section 9 addresses dams and dikes, Section 13 addresses refuse disposal, and Section 14 addresses temporary occupation of work built by the United States. Various pieces of legislation have modified these authorities, but not removed them.

In 1972, amendments to the Federal Water Pollution Control Act added what is commonly called Section 404 authority (33 U.S.C. 1344) to the USACE regulatory program. In it, the Secretary of the Army, acting through the Chief of Engineers, is authorized to issue permits, after notice and opportunity for public hearing, for the discharge of dredged or fill material into waters of the United States at specified disposal sites. Selection of such sites must be in accordance with guidelines developed by the U.S. Environmental Protection Agency (EPA) in conjunction with the Secretary of the Army. These guidelines are regulatory and are known as the 404(b)(1) Guidelines.

The discharge of all other pollutants into waters of the United States is regulated under Section 402 of the act, which supersedes the Section 13 permitting authority mentioned above. The Federal Water Pollution Control Act was amended in 1977 and given the common name of Clean Water Act and again amended in 1987 to modify criminal and civil penalty provisions and to add an administrative penalty provision.

Also in 1972, with enactment of the Marine Protection, Research, and Sanctuaries Act, the Secretary of the Army, acting through the Chief of Engineers, was authorized to issue permits under Section 103 for the transportation of dredged material for ocean disposal. This authority also carries with it the requirement of notice and opportunity for public hearing. Disposal sites for such discharges are selected in accordance with criteria developed by EPA in consultation with the Secretary of the Army.

(b) Geographic jurisdiction

The USACE, acting under Section 404 of the Clean Water Act and Section 10 of the River and Harbors Act of 1899, regulates certain activities occurring in waters of the United States and navigable waters of the United States. The term Waters of the United States is defined in 33 CFR 328.3, as:

> *(a) The term "Waters of the United States" means*
>
> *(1) All waters which are currently used, or were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide;*
>
> *(2) All interstate waters including interstate wetlands;*
>
> *(3) All other waters such as intrastate lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation or destruction of which could affect interstate or foreign commerce including any such waters:*
>
> *(i) Which are or could not be used by interstate or foreign travelers for recreation or other purposes; or*
>
> *(ii) From which fish or shellfish are or could be taken and sold in interstate or foreign commerce; or*
>
> *(iii) Which are used or could be used for industrial purpose by industries in interstate commerce;*
>
> *(4) All impoundments of waters otherwise defined as waters of the United States*
>
> *(5) Tributaries of waters identified in Paragraphs (a) (1–4) of this section;*
>
> *(6) The territorial seas;*

(7) Wetlands adjacent to waters (other than waters that are themselves wetlands) identified in Paragraphs (a) (1)–(6) of this section.

Waste treatment systems, including treatment ponds or lagoons designed to meet the requirements of CWA (other than cooling ponds as defined in 40 CFR 123.11(m), which also meet the criteria of this definition), are not waters of the United States.

The limit of USACE jurisdiction under Section 404 in the territorial seas is 3 nautical miles seaward from the baseline along the shore. The limit in tidal waters of the United States extends to the high tide line in the absence of adjacent wetlands and extends to the limit of the wetlands when adjacent wetlands are present.

Ordinary high water

The limit of USACE jurisdiction in nontidal waters of the United States, in the absence of adjacent wetlands, is the ordinary high water mark. The ordinary high water mark is the limit of USACE jurisdiction for such water features as streams, reservoirs, lakes, and ponds. Ordinary high water mark is defined as that line on the shore established by the fluctuations of water and indicated by physical characteristics such as clear, natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas. Even very small ephemeral streams may have an ordinary high water mark and be waters of the United States.

Wetlands

When adjacent wetlands are present, USACE jurisdiction under Section 404 extends beyond the ordinary high water mark to the limit of the adjacent wetlands. Wetlands are defined as those areas that are inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas. Delineations of wetlands must be conducted using the USACE of Engineers Wetland Delineation Manual, USACE Waterways Experiment Station Wetlands Research Program Technical Report Y–87–1, dated January 1987. (Online edition including all supplemental guidance is available on the USACE Web site at:

http://www.usace.army.mil/inet/functions/cw/cecwo/reg/wlman87.pdf

Adjacent is defined as bordering, contiguous, or neighboring. Wetlands separated from other waters of the United States by manmade dikes or barriers, natural river berms, beach dunes and the like are adjacent wetlands. When the water of the United States consists only of wetlands, USACE jurisdiction extends to the limit of the wetland.

Wetlands are areas that are covered by water or have waterlogged soils for long periods during the growing season. Plants growing in wetlands are capable of living in saturated soil conditions for at least part of the growing season. Wetlands such as swamps and marshes are often obvious, but other wetland types are not easily recognized, often because they are dry during part of the year or do not look very wet from the roadside. Some of these wetland types include, but are not limited to:

- bottomland forests
- pocosins
- pine savannahs
- bogs
- wet meadows
- potholes
- wet tundra

The USACE uses the following three characteristics when making wetland delineations:

- vegetation
- soil
- hydrology

Unless an area has been altered or is a rare natural situation, wetland indicators of all three characteristics must be present for an area to be a wetland.

There are some general situations in which an area has a strong probability of being classified as a wetland. These conditions include:

- The area occurs in a flood plain or otherwise has low spots in which water stands at or above the soil surface during the growing season. **Caution: Most wetlands lack both standing water and waterlogged soils during at least part of the growing season.**
- The area has plant communities that commonly occur in areas having ponded or saturated soil conditions for part of the growing season (cypress-gum swamps, bottomland hardwood forests, cordgrass marshes, cattail marshes, bulrush and tule marshes, and sphagnum bogs).
- The area has hydric soils such as peats and mucks. While most soils map show areas of hydric soils that could be wetlands, it is important to note that there are soils, in addition to peat or mucks, that qualify as wetland soils.
- The area is periodically flooded by tides, even if only by strong, wind-driven, or spring tides.
- U.S. Geological Survey (USGS) topographic maps, National Wetland Inventory maps, U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) soil surveys, local NRCS office aerial photos that show wetland areas, and other Federal, state, and local maps and photos that provide wetland information.

If any of the above situations occur or if other conditions exist that indicate that a wetland or other water of the United States may be present, the designer should ask the local USACE office for assistance in making a determination. Procedures for determining the hydrology of wetlands and can provide guidance in determining the minimal effect provisions of wetland determination are provided in NEH630.19.

Navigable waters

Under Section 10 of the River and Harbors Act of 1899, the USACE regulates navigable waters of the United States, a subset of waters of the United States. Navigable waters of the United States are defined at 33 CFR 329 as those waters that are subject to the ebb and flow of the tide and/or are presently used, or have been used in the past, or may be susceptible for use to transport interstate or foreign commerce. A determination of navigability, once made, applies laterally over the entire surface of the water body, and is not extinguished by later actions or events which impede or destroy navigable capacity. The several factors which must be examined when making a determination whether a water body is a navigable water of the United States are listed. Generally, the following conditions must be satisfied:

- past, present, or potential presence of interstate or foreign commerce
- physical capabilities for use by commerce as in paragraph (a) of this section
- defined geographic limits of the water body

The limit of USACE jurisdiction for navigable waters of the United States for oceanic and tidal waters is the mean high tide line shoreward and 3 nautical miles seaward from the point where the shore directly contacts the open sea. The limit for rivers and lakes is the ordinary high water mark and jurisdiction extends to the entire water surface and bed of the water body.

Navigable waters of the United States include many coastal waters including bays and portions of major rivers. You may obtain information about navigable waters located within each USACE district by contacting that district.

Regulated activities

A permit is required from the USACE for discharges of dredged or fill material into waters of the United States (33 CFR Part 323). Dredged material is material that is excavated or dredged from waters of the United States (33 CFR 323.2(c)). The discharge of dredged material is material excavated or dredged from waters of the United States and redeposited into waters of the United States, runoff or overflow from a contained land or water disposal area, or the redeposit of dredged material other than incidental fallback (33 CFR 323.2(d)). Discharge of dredged material does not include discharges associated with onshore processing of dredged material extracted for commercial use, activities involving only cutting or removing vegetation so that root systems are not disturbed, or incidental fallback of dredged material.

Fill material is material that is placed into waters of the United States where the material has the effect of replacing a water with dry land or changing the bottom elevation of any portion of a water of the United States (33 CFR 323.2(e)). However, fill material does not include trash or garbage. The discharge of fill ma-

terial is the addition of fill material into waters of the United States. The definition identifies activities that may be associated with the placement of fill including any structure or infrastructure; impoundments; recreational, industrial, commercial, residential, or other uses; causeways or roads; dams or dikes; artificial islands; property protection or reclamation devices; beach nourishment; levees; sewage treatment facilities; intake and outfall pipes; subaqueous utility lines; construction or maintenance of any liner, berm, or other infrastructure of solid waste landfill; placement of overburden, slurry or tailings or similar mining-related materials; and artificial reefs. Plowing, cultivating, seeding, and harvesting for the production of food, fiber, and forest products are excluded.

All discharges of dredged or fill material into waters of the United States require a permit, unless they are exempted or excepted in regulations. Section 404(f) exemptions (33 CFR 323.4) include normal farming, forestry, and ranching activities; maintenance of recently damaged structures; construction/maintenance of farm ponds and irrigation ditches; construction of temporary sedimentation basins; Section (208)(b)(4) approved state program activities; and construction/maintenance of farm, forest, and mining roads using approved best management practices. However, discharges that are a part of an activity whose purpose is to convert an area of waters of the United States to a use to which it was not previously subject, and the flow or circulation of waters of the United States would be impaired or the reach of such waters reduced, must have a permit.

For additional information about regulated activities and exemptions, the designer should contact the local USACE district office.

(c) Permitting

Activities requiring a permit from the USACE under Section 404 of the Clean Water Act or Section 10 of the Rivers and Harbors Act of 1899 may be permitted by General Permit or Individual Permit. There are three types of general permits: Nationwide General Permit, Regional General Permit, and Programmatic General Permit. There are two types of individual permits: letter of permission and standard individual permit. Standard individual permits require a public notice. Under Section 401 of the Clean Water Act, certification of compliance with state water quality standards by the state Water Quality Agency is required for any discharge of pollutants into waters of the United States. All Section 404 permits, individual or general, require Section 401 water quality certification. Conditions may be associated with state water quality certification and may apply to both individual and general permits.

General permits

A general permit is issued nationwide or regionally for a category or categories of activities that are either similar in nature and cause only minimal individual and cumulative adverse impacts (nationwide and regional general permits) or would result in avoiding unnecessary regulatory control exercised by another Federal, state, or local agency, and the environmental consequences of the activity would be individually and cumulatively minimal (Programmatic General Permit). General permits always include terms and conditions for compliance, may require preconstruction notification of the USACE, and may be issued for a period not to exceed 5 years (See 33 CFR 320.1 (c), 322.2 (f), 323.2 (h), 325.2 (e)(2), and 330).

Nationwide General Permit—A Nationwide General Permit (NWP) is a type of general permit issued nationally. The regulations that govern NWPs are found at 33 CFR 330. NWPs are valid for 5 years from the date of issuance. NWP regional conditions for may be adopted by each USACE district for a particular state.

Preconstruction notification (PCN) to the USACE is required in many cases, and resource agency coordination with the EPA, U.S. Fish and Wildlife Service (USFWS), and state water quality, fish and wildlife, and cultural resource agencies is required in some cases. A PCN to the USACE is required (even if a PCN is not otherwise required) if threatened or endangered species or its critical habitat may be affected by or is in the vicinity of the proposed activity, or if historic properties listed or eligible for listing in the National Register of Historic Places may be affected by the proposed project. No activity may be authorized if the continued existence of a federally listed threatened, endangered, or proposed threatened or endangered species is jeopardized or critical habitat is destroyed or adversely modified.

No activity which may affect historic properties listed or eligible for listing in the National Register of His-

toric Places is authorized until the district engineer has complied with the provisions of 33 CFR part 325, appendix C. The prospective permittee must notify the USACE if the authorized activity may affect any historic properties listed, determined to be eligible, or which the prospective permittee has reason to believe may be eligible for listing on the National Register of Historic Places and may not begin the activity until notified by the USACE that the requirements of the National Historic Preservation Act have been satisfied and that the activity is authorized. For activities that may affect historic properties listed or eligible for listing, in the National Register of Historic Places, the preconstruction notification must state which historic property may be affected by the proposed work and include a vicinity map indicating the location of the historic property.

The USACE has 30 days to review a PCN to determine if it is complete. PCNs must include:

- name, address, telephone number
- location of the proposed project
- brief description of the proposed project
- the project's purpose
- direct and indirect adverse environmental effects
- other nationwide permits or individual permits to be used
- other permit specific items for NWPs 7, 12, 14, 18, 21, 29, 33, 34, 38, 39, 40, 41, 42, 43, 44

If the PCN is not complete, the USACE can request the required information only once. Within 45 days of receipt of a complete PCN, the USACE must advise the prospective permitee of its determination whether the proposed activity meets the terms and conditions of the NWP, or exercise discretionary authority.

Resource agency coordination is required only for NWP activities that require a preconstruction notification to the USACE and result in the loss of greater than a acre of waters of the United States. If resource agency coordination is required, the agencies have 10 calendar days to notify the USACE that they intend to provide comments. If an agency so notifies the USACE, the USACE must wait an additional 15 calendar days for the comments. A signed compliance certification must be submitted by every permittee who has received NWP verification from the USACE. Postconstruction reports are required for nonreporting NWP 39 (residential, commercial, and institutional developments).

A permittee may use more than one NWP to authorize a single and complete project, provided the acreage loss of waters of the United States does not exceed the highest specified acreage limit of the NWPs used to authorize that single and complete project. Projects must be designed and constructed to avoid and minimize adverse effects to waters of the United States to the maximum extent practicable at the project site. Mitigation in all forms (avoidance, minimization, and compensation) may be required to the extent necessary to ensure minimal adverse impacts on the aquatic environment.

NWP 27 authorizes certain activities in waters of the United States associated with the restoration of former waters, the enhancement of degraded tidal and nontidal wetlands and riparian areas, and the restoration and enhancement of nontidal streams and nontidal open water areas. This NWP may be particularly appropriate for Section 404 authorization of many stream restoration projects.

Regional General Permit—A Regional General Permit (RGP) is a type of general permit that is issued regionally. Regulations addressing RGPs are found at 33 CFR 322.2(f), 323.2(h), and 325.2(e)(2). RGPs are similar to NWPs and contain terms and conditions intended to protect the environment including natural and cultural resources. Work that would not comply with those provisions may require authorization by individual permit. However, compliance with the conditions contained in this RGP does not guarantee authorization of the work by RGP. Work or structures that would have unacceptable impacts on the public interest are not authorized. Activities requiring Department of the Army authorization that are not specifically authorized by an RGP are prohibited unless they are authorized by another general permit or an individual permit.

Programmatic General Permit—A Programmatic General Permit (PGP) is a type of general permit that is issued to avoid unnecessary duplication of regulatory control exercised by another Federal, state, or local agency. With a PGP, a permit applicant generally must only apply to one agency, rather than applying

to both agencies for permits for the same work. PGPs have characteristics similar to NWPs and RGPs.

Individual permit

The basic form of authorization is the individual permit. Processing such permits involves evaluation of individual, project-specific applications in what can be considered three steps: preapplication coordination (for larger projects), formal permit application review, and decisionmaking. Preapplication is helpful for more complex cases and is addressed later in this chapter. Once a complete application is received, the formal review process begins. The project manager prepares a public notice (if required), evaluates the impacts of the project and considers all comments received, addresses potential modifications to the project if appropriate, and drafts or oversees drafting of appropriate documentation to support a recommended permit decision. The permit decision document includes the environmental impacts of the project, findings of the public interest review process, and any special evaluation required by the type of activity such as determinations of compliance with the Section 404(b)(1) Guidelines. As noted above, water quality certification from the state water quality agency must be obtained before a Section 404 permit may be issued.

Letter of permission—A letter of permission (LOP) is a type of permit issued through an abbreviated processing procedure that includes coordination with Federal and state fish and wildlife agencies, as required by the Fish and Wildlife Coordination Act, and a public interest evaluation, but without the publishing of an individual public notice (see 33 CFR 325.2(e)(1)). An LOP procedure is an alternative procedure for evaluating individual permit applications for activities in waters of the United States authorized by the USACE. The LOP procedure serves to reduce the administrative procedures and to expedite permit decisions for cases that include only minor work in waters of the United States that do not have significant individual or cumulative environmental impacts and should encounter no appreciable opposition. The LOP may not be used to authorize the transportation of dredged material for the purpose of dumping it in ocean waters. LOPs may be used in those cases subject to Section 10 of the Rivers and Harbors Act of 1899 when, the proposed work would be minor, would not have significant individual or cumulative impacts on environmental values, and should encounter no appreciable opposition.

To use LOPs in cases subject to Section 404 of the Clean Water Act, the USACE must first consult with Federal and state fish and wildlife agencies, the EPA, the state water quality certifying agency, and, if appropriate, the state Coastal Zone Management Agency, on appropriate categories of activities for authorization under LOP procedures. The USACE must also issue a public notice advertising the proposed list and the LOP procedures, request comments, and offer an opportunity for public hearing. Finally, 401 certification must be issued or waived and, if appropriate, CZM consistency concurrence obtained or presumed either on a generic or individual basis.

An LOP may include general conditions and appropriate case-specific provisions necessary to protect the environment including natural and cultural resources. LOP procedures may not have an expiration date, but LOPs issued under the procedure always will have an expiration date. The USACE must conduct a public interest evaluation, but there is no requirement for a public notice. The permittee is responsible for obtaining any additional Federal, state, or local permits that may be required. Refer to the applicable LOP procedure for the application procedures and other requirements in each case.

Work that does not comply with the provisions of LOP procedure may require authorization by standard individual permit. Compliance with the LOP procedure, including the general conditions, does not guarantee authorization of the work by an LOP.

Standard individual permit—Activities that do not qualify for authorization under a general permit or an LOP procedure may qualify for authorization by standard individual permit (SP). Authorization under SP may be obtained only through application with the USACE. These permits are issued for activities that have more than minimal adverse impacts to waters of the United States, and evaluation of each permit application involves more thorough review of the potential environmental and socioeconomic effects of the proposed activity. The applicant must submit required information (33 CFR 325.1(d)) on an Individual Permit Application Form (Form 4345) or an approved alternative form. An alternative analysis and a mitigation plan are not required for a complete application to prepare a public notice, but are very helpful. The SP evaluation process may be summarized as follows:

- preapplication coordination
- Individual Permit Application Form submitted
- after receipt of a complete application, the USACE issues joint public notice for Section 404 and Section 401 water quality certification
- 15- to 30-day Public Notice comment period
- opportunity for public hearing
- USACE reviews comments, evaluates the permit application based on regulations, and completes required documentation
- USACE makes a decision: issue, issue with conditions, or deny

Permit decisions are based on probable impacts associated with the proposed project, including cumulative impacts, on the public interest (33 CFR 320.4). Public review interest factors include:

- conservation
- economics
- aesthetics
- general environment
- wetlands
- cultural values
- fish and wildlife values
- land use
- flood hazards
- property ownership
- flood plain values
- navigation
- recreation
- shore erosion and accretion
- water supply/water quality
- energy needs
- safety
- mineral needs
- food and fiber production
- needs and welfare of people

A permit will be granted if the proposed project is not contrary to the public interest and meets other legal requirements, such as the Section 404(b)(1) Guidelines and state water quality certification.

The Section 404(b)(1) Guidelines are the substantive criteria developed by the EPA and used by the USACE to evaluate proposed discharges into waters of the United States. The USACE may not issue a permit under Section 404 if the proposal does not meet the 404(b)(1) guidelines. The USACE may only issue a permit for the least environmentally damaging practicable alternative. Practicability includes consideration for cost, existing technology, and logistics.

The level of review of a proposed project is commensurate to the level of impact to waters of the United States. The USACE may not issue a permit if the proposed project is not in compliance with other laws (such as Section 401 of the Clean Water Act; National Environmental Policy Act; Fish and Wildlife Coordination Act; Endangered Species Act (threatened or endangered species information); Coastal Zone Management Act; National Historic Preservation Act; Magnuson-Stevens Fishery Conservation and Management Act; Essential Fish Habitat); if the activity would result in significant degradation of aquatic environment (net after mitigation); or if there is not appropriate and practicable mitigation. Public interest reviews, Section 404(b)(1) analyses and National Environmental Policy Act analyses require a resource impact assessment, which may include Hydrogeomorphic Approach (HGM), Wetland Evaluation Technique (WET), Habitat Evaluation Procedures (HEP) or another technique for wetlands, or the EPA's rapid bioassessment method or another technique for streams.

Preapplication coordination

Processing permit applications and requests for verification of general permit authorization may involve preapplication coordination. Preapplication coordination usually involves one or several telephone conversations or meetings between an applicant and USACE district staff and may include interested resource agencies (Federal, state, or local), and sometimes, the interested public. The basic purpose of such conversations or meetings is to provide for informal discussions about the pros and cons of a proposal and potential alternatives before an applicant makes irreversible commitments of resources (funds, detailed designs). The process is designed to provide the applicant with

an assessment of the viability of some of the more obvious alternatives available to accomplish the project purpose, discuss measures for reducing the impacts of the project, and inform the applicant of the factors the USACE must consider in its decisionmaking process. In many cases, preapplication coordination with the USACE has produced project changes that resulted in streamlined regulatory requirements; for example, a general permit was sufficient for the revised project where a standard individual permit would have been required for the original proposal.

Mitigation

A fundamental precept of the Regulatory Program is the Department of the Army's mitigation policy (33 CFR Part 320.4 (r)), which applies to all Regulatory Program authorizations including general permits. When the USACE reviews a project that would require Department of the Army authorization, its evaluation typically includes a determination of whether the applicant has taken sufficient measures to mitigate the project's likely adverse impact on the aquatic ecosystem.

In a Memorandum of Agreement (MOA) signed February 6, 1990, between the USACE and the EPA, mitigation was defined as a sequential process of avoiding, minimizing, and compensating for adverse impacts to the aquatic ecosystem.

- Avoid: Take all appropriate and practicable measures to avoid those adverse impacts to the aquatic ecosystem that are not necessary.
- Minimize: Take all appropriate and practicable measures to minimize those adverse impacts to the aquatic ecosystem that cannot reasonably be avoided.
- Compensate: Implement appropriate and practicable measures to compensate for adverse project impacts to the aquatic ecosystem that cannot reasonably be avoided or further minimized. This step is also referred to as compensatory mitigation. The purpose of compensatory mitigation is to replace those aquatic ecosystem functions that would be lost or impaired as a result of a USACE-authorized activity.

The district engineer will normally require the implementation of all appropriate and practicable compensation as a condition of the Department of the Army authorization. Compensatory mitigation is required at minimum 1:1 ratio (acres mitigated to acres impacted) for all wetland impacts requiring a preconstruction notification for a nationwide permit unless it has been waived (determined on a case-by-case basis).

Regulatory Guidance Letter 02–2 applies to all compensatory mitigation proposals associated with permit applications submitted for approval after December 24, 2002. USACE districts will use watershed and ecosystem approaches when determining compensatory mitigation requirements, consider the resource needs of the watersheds where the impacts will occur, and also consider the resource needs of neighboring watersheds. USACE districts may have districtwide or statewide mitigation guidelines, as well.

Mitigation banking is the restoration, enhancement, creation, and, in exceptional circumstances, preservation undertaken to compensate in advance for adverse impacts to the aquatic ecosystem. Mitigation banking may be appropriate when compensatory mitigation cannot be practicably achieved or would not be as environmentally beneficial at the impact site or a nearby site. The USACE, EPA, RCS, USFWS, and National Oceanic and Atmospheric Administration National Marine Fisheries Service (NOAA Fisheries Service) Federal Mitigation Banking Final Policy (Federal Register: November 28, 1995 (Volume 60, Number 228)) guidance regarding the establishment, use, and operation of mitigation banks for the purpose of providing compensation for adverse impacts to wetlands and other aquatic resources is provided to clarify the manner in which mitigation banks may be used to satisfy mitigation requirements of the CWA Section 404 permit program and the wetland conservation provisions of the Food Security Act (FSA) (Swampbuster provisions). Recognizing the potential benefits mitigation banking offers for streamlining the permit evaluation process and providing more effective mitigation for authorized impacts to wetlands, the agencies encourage the establishment and appropriate use of mitigation banks in the Section 404 and Swampbuster programs.

An in-lieu-fee program allows a permittee to pay a fee to an established trust fund in lieu of implementing specific onsite or offsite compensatory mitigation. The amount of the in-lieu-fee paid will normally represent the fair market cost of replacing those aquatic ecosystem resources that would be lost or impaired as a result of the authorized activity. The trust fund, in turn,

finances mitigation projects that are designed to restore, enhance, create, or preserve aquatic ecosystem functions. The Federal Guidance on the Use of In-lieu-fee Arrangements for Compensatory Mitigation, under Section 404 of the Clean Water Act and Section 10 of the Rivers and Harbors Act, elaborates on in-lieu-fee mitigation arrangements in the Banking Guidance by outlining the circumstances where in-lieu-fee mitigation may be used, consistent with existing regulations and policy.

The USACE supports a strong partnership with states in regulating activities under Section 404 of the Clean Water Act. This is achieved with joint permit processing procedures (joint public notices, meetings, and hearings), programmatic general permits founded on effective state programs, joint EISs, special area management planning, and regional conditioning of nationwide permits, where appropriate.

(d) Clean Water Act, National Pollutant Discharge Elimination System

The EPA administers the Clean Water Act (CWA) program. The primary objective of the CWA is to restore and maintain the Nation's waters. This program covers point discharge permits for construction stormwater runoff, erosion control permit, or ongoing discharges of pollutants. The CWA's primary control program is the National Pollutant Discharge Elimination System (NPDES). The CWA prohibits the discharge of pollutants from point sources (any single identifiable source of pollution, a pipe, ditch, and ship) to waters of the United States without a NPDES permit. The EPA or the approved state environmental control agency has responsibility for administering NPDES permits. More information is available on the following Web site:

http://cfpub.epa.gov/npdes/

As of 1992, any earth disturbance greater than 5 acres was required to have a stormwater permit, and as of March 2003, a permit must be obtained for any earth disturbance greater than 1 acre. This includes sites that are less than 1 acre if they are "…part of a larger common plan of development or sale or within 500 feet of a lake or stream."

States can impose more stringent pollution limits than the Federal rules require. States can also require more frequent monitoring and reporting and the use of numerous best management practices to control the pollution and the regulation of small sites. Although there are some differences between states, all construction stormwater permits will be general permits. While there are exceptions, most of what is required is a Stormwater Management Plan.

654.1704 National Flood Insurance Program

In 1968, Congress created the National Flood Insurance Program (NFIP) to reduce damages and loss of life caused by floods. The Mitigation Division of the Federal Emergency Management Agency (FEMA) manages the NFIP. Nearly 20,000 communities across the United States voluntarily participate in the NFIP via the adoption and enforcement of flood plain management ordinances. In exchange, the NFIP makes federally backed flood insurance available to these communities. However, in general, it is local governments that typically regulate development and other activities in the flood plain, not FEMA.

(a) Flood plain maps

This program is administered with the aid of Flood Hazard Boundary maps, Flood Insurance Rate maps, and Flood Boundary and Floodway maps. Flood plain maps provide the basis for flood management, regulation, and insurance requirements by identifying areas subject to flooding that threaten life safety and damage to property.

These maps identify several areas of flood hazards such as Special Flood Hazard Area (SFHA) floodways and floodway fringes. The SFHA is defined as an area of land that would be inundated by a base flood (typically the 1% chance event). New construction within the SFHA must comply with FEMA requirements. Evaluations to determine these areas follow the procedures in the Flood Insurance Study Guidance and Specifications for Study Contractors, FEMA 37 (FEMA, 11085). FEMA 37 defines a floodway:

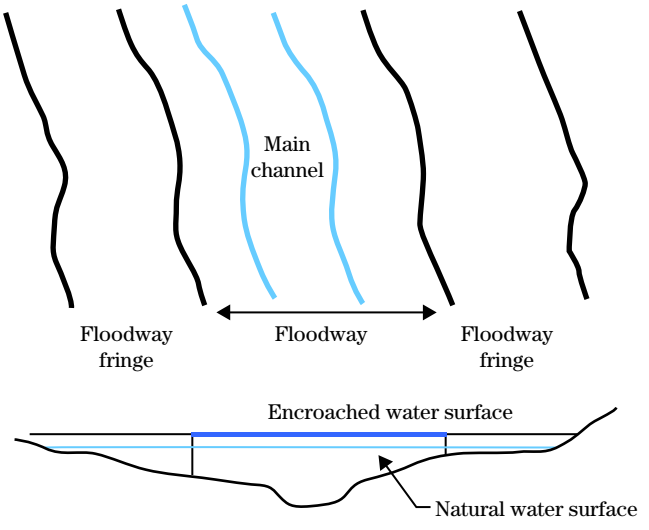
> *…as the channel of a river or other watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water-surface elevation by more than designated height.*

While there are exceptions, the base flood is typically the 1-percent chance event, and the designated height is 1 foot. The floodway is where the water is typically the deepest and fastest. It is the area of the waterway that should be kept free of obstructions to allow floodwaters to move downstream. Since most stream and channel work occurs within this area, a detailed analysis is typically required. The floodway fringe is the area that can be blocked by an encroachment without raising the base flood by more than the designated height. To define a flood map, calculations are made (often using HEC–RAS computer software) to determine how much of the fringe can be completely blocked. Figure 17–1 illustrates this determination.

(b) Types of studies

Several types of analyses may be required as part of a stream project. The most common type of study for simple projects is the so called no rise or no impact analysis. This must show that the proposed work will not impact the preproject base flood elevation. This can be a simple estimate of conveyance before and after the proposed project. In areas of higher risk or for projects involving more significant changes to the channel, a more complicated computer analysis may be required. Certification of this must be obtained before construction. The engineering or no rise analysis must be supported by technical data that is typically based upon the same engineering methodology used

Figure 17–1 Example calculation of a blocked floodway fringe



to develop the original mapped floodway. While local governments may review and approve the no rise submittal, they may request technical assistance from the FEMA regional office.

In some circumstances, it is not possible to demonstrate that a proposed project will not cause a rise in water surface during the base flood. In this situation, it can sometimes be argued that the potential increase in the base flood would not cause damage to habitable structures. For example, the area of possible impact may only be cropland. This type of study is the no damage analysis.

If a project results in significant changes in the flood plain, it may be necessary to revise the flood plain map. These involve the use of Letters of Map Revision (LOMR). The modeling and administrative work to obtain these can be significant.

(c) Flood plain map changes

Flood plain maps are periodically updated and revised to reflect changing conditions, such as new topography, land development, as well as updated mapping studies. They may also be updated to reflect stream and river projects. These flood plain map changes take one of the following forms:

- Publication of a new Flood Insurance Rate Map (FIRM)—the official map of a community on which FEMA has delineated both the special hazard areas and the risk premium zones applicable to the community.
- Letters of Map Change (LOMC)
  - Conditional Letter of Map Amendment (CLOMA)—provides FEMA's comment on whether a proposed project would be excluded from the Special Flood Hazard Area (SFHA) as shown on the NFIP map. This letter does not revise an effective map, but it does indicate whether the project would or would not be removed from the SFHA.
  - Conditional Letter of Map Revision (CLOMR)—provides for a review of whether a proposed project within the SFHA meets the minimum flood plain management criteria of the NFIP. If it does, it may provide what revisions will be made to the community's NFIP map of the project. A CLOMR may be required before a project can be built.
  - Conditional Letter of Map Revision based on Fill (CLOMRF)—similar to above, but is for project impacts based on fill that would exclude an area from the SFHA shown on the NFIP map.
  - Letter of Map Amendment (LOMA)—an amendment to the currently effective FEMA map that establishes that a property is not located in a SFHA. A LOMA is issued only by FEMA.
  - Letter of Map Revision (LOMR)—an official amendment to the currently effective FEMA map and reflects changes in flood zones, delineations, and elevations. A LOMR is issued only by FEMA.
  - Letter of Map Revision based on Fill (LOMRF)—similar to the LOMR, but based on fill.
- Local map changes

More information on this program is provided at the following Web site:

http://www.fema.gov/fhm/

654.1705 Endangered Species Act, as amended 1973

The Endangered Species Act (ESA) is a Federal statute that was designed to protect threatened and endangered species from extinction, excluding all recognized insect pests from this distinction. It is designed to ensure that Federal agencies will not take actions that might jeopardize listed, threatened, or endangered species. Specifically, it states:

> *The purposes of this Act are to provide a means whereby the ecosystems upon which endangered species and threatened species depend may be conserved, to provide a program for the conservation of such endangered species and threatened species, and to take such steps as may be appropriate to achieve the purposes of the treaties and conventions set forth…*

The USFWS (Department of the Interior) and NOAA Fisheries Service (Department of Commerce) are responsible for promulgating the provisions of the ESA. The USFWS's primary responsibilities include designating species as threatened or endangered, establishing recovery plans, designating critical habitat, and assisting the states and other Federal agencies with conservation and implementation of the ESA. Generally, NOAA Fisheries Service is responsible for protecting anadromous salmonids and marine mammals, while the USFWS's scope of authority covers a wider range of terrestrial and freshwater species (birds, butterflies, plants, snails, large mammals, and fish). The agencies maintain a list of endangered and threatened species. Any species that have been removed from endangered status are said to have been delisted.

The ESA mandates cooperation between Federal, state, and foreign governments in the conservation of listed species. The Secretaries of the Interior and Commerce must cooperate with the states to acquire and manage land for conservation purposes and have the authority to enter into cooperative agreements to provide assistance to states that establish programs for the conservation of listed species. For example, Section 6 of the ESA provides Federal financial assistance and incentives to states that develop and maintain conservation programs for resident listed resources.

Under the ESA, *all* Federal agencies must participate in the conservation and protection of threatened and endangered species. Specifically, Section 7 of the ESA charges Federal agencies to aid in the conservation of listed species (Section 7 (a)(1)) and requires Federal agencies to ensure that their activities will not jeopardize the continued existence of listed species or adversely modify designated critical habitats (Section 7 (a)(2)). Under the provisions of Section 7(a)(2), a Federal agency that permits, licenses, funds, or otherwise authorizes activities must consult with the Services to ensure that its actions will not jeopardize the continued existence of any listed species. Consequently, stream restoration projects funded, designed, or authorized by the NRCS are subject to Section 7 consultation, even if the effects or outcome of the project are completely beneficial to listed resources. The following Web sites provide additional information on listed species and the ESA:

http://www.fws.gov/endangered/

http://www.nmfs.noaa.gov/pr/species/esa_species.htm

654.1706 Fish and Wildlife Coordination Act, as amended 1965

The Fish and Wildlife Coordination Act (FWCA), as amended, proposes to assure that fish and wildlife resources receive equal consideration with other values during the planning of federally funded water resources development projects. The act was passed because the goals of water-related projects (flood control, irrigation, navigation, hydroelectric power) may conflict with the goal of conserving fish and wildlife resources. Conversely, project developers can design water development projects to enhance the quality and enjoyment of fish and wildlife resources if such goals are incorporated into project plans. The USFWS, NOAA Fisheries Service, and the state fish and wildlife agencies comment on USACE Individual Permit applications under this authority during the 30-day public comment period.

The Act authorizes the Secretary of the Interior to provide assistance to and cooperate with Federal, state, and public or private agencies and organizations in the development and protection of fish and wildlife resources and habitat, make surveys and investigations of the fish and wildlife in the public domain, and accept donations of land and funds that will further the purposes of the Act. The following Web site provides information on requirements for Federal agencies to manage fish and wildlife species under the FWCA:

http://laws.fws.gov/lawsdigest/fwcoord.html

654.1707 National Environmental Policy Act, as amended 1982

The purposes of the National Environmental Policy Act (NEPA) are to:

- declare a national policy which will encourage productive and enjoyable harmony between people and their environment
- promote efforts that will prevent or eliminate damage to the environment and biosphere and stimulate the health and welfare of society
- enrich the understanding of the ecological systems and natural resources important to the Nation
- establish a Council on Environmental Quality

The NEPA requires that Federal agencies use a systematic, interdisciplinary approach in planning and decisionmaking that may impact the human environment. The Act calls for Federal decisionmakers to consider the environmental impacts of their actions before implementing them. The following Web site provides information on requirements as they relate to NEPA:

http://ceq.eh.doe.gov/nepa/regs/nepa/nepaeqia.htm

654.1708 Magnuson-Stevens Fishery Conservation and Management Act, as amended 1996

The purposes of the Magnuson-Stevens Fishery Conservation and Management Act are to take immediate action to:

- conserve and manage the fishery resources found off the coasts of the United States and the anadromous species and Continental Shelf fishery resources of the United States
- adapt exclusive fishery management authority beyond the exclusive economic zone over such anadromous species and Continental Shelf fishery resources and fishery resources in the special areas

The act supports and encourages the implementation and enforcement of international fishery agreements for the conservation and management of highly migratory species and encourages the negotiation and implementation of additional such agreements as necessary. The following Web site provides information on the requirements:

http://www.nmfs.noaa.gov/sfa/magact/

654.1709 State regulations and permitting—general

Each state has individual statutes and codes that provide the legal framework for developing and managing water resource-related projects. A variety of permits are required to work within rivers, streams, and/or wetlands. State fish and wildlife agencies and land management agencies are the typical implementing agency. Local permit requirements should be fully identified when developing project plans, designs, and construction specifications. The following list has links to examples for state and local permit requirements. Typically, the state environmental protection, environmental quality, ecology, or natural resources is responsible for administrating permits.

California:

http://www.swrcb.ca.gov/quality.html

Maine:

http://www.maine.gov/dep/permits.htm

Oregon:

http://licenseinfo.oregon.gov/

Washington:

http://www.ecy.wa.gov/programs/sea/pac/jarpa.html

Some state wetland regulatory programs also exist as noted at the following Web site:

http://www.aswm.org/swp/states.htm

654.1710 State regulations and permitting—fish passage

Many states have specific language regarding fish passage and fish screen requirements, whereas others are more general in scope. Design engineers and biologists should be aware of all state regulations pertaining to fish passage and screen requirements. A few examples of state codes and regulations that require fish passage or fish screening are listed below. However, some states do not have specific codes requiring fish passage or screens. Contact the local fish and wildlife agency for more information.

California:

http://www.dfg.ca.gov/1600/1600code.html

Maine:

http://www.maine.gov/mdot/environmental-office-homepage/documents/finalfishpassage2003.pdf

Oregon:

http://www.dfw.state.or.us/ODFWhtml/InfoCntrFish/InfoCntrFish.html

Washington:

http://www.wdfw.wa.gov/hab/engineer/habeng.htm

654.1711 Conclusion

Any stream project must meet or exceed all Federal, state, and local regulatory requirements. Regulatory issues can cover a wide range of areas. To ensure full compliance, it is recommended that early contact and frequent consultation be made with all the appropriate agencies.

Appendix A Postscript

No river can return to its source,
yet all rivers must have a beginning.

— Native American Proverb

In an age of heightened environmental sensitivity, green or natural design approaches are finding a strong foothold in the restoration and rehabilitation of stream ecosystems. Streambank soil bioengineering technology, however, has been used around the globe for centuries. A few designed systems were installed in the United States in the 1930s. Even for the next 40 years, few installations were truly integrated with complex stream restoration plans. It was not until the late 1970s that these less structural approaches for stream stabilization began to be used in place of hard engineered solutions. The Winooski River restoration is an early example of soil bioengineering techniques, combined with sound engineering approaches implemented in the 1930s (fig. E–1).

Later in the twentieth century, fluvial geomorphology and other emerging technologies proved important in stream restoration work. This emerging more natural design approach to design has often appeared to be more risky to landowners, regulators, and designers, since the collective experience in successful stream restorations was primarily focused on the physical stability of the system. Designing stream restorations with soil bioengineering practices combined with traditional engineering approaches requires similar attention to the strengths and performance criteria of materials, as well as their long-term durability, maintenance needs, and applicability to achieve the project's goals. This softer approach requires attention to the design requirements of the site conditions, and some term these less traditional methods a combination of science, engineering, and art (fig. E–2).

Figure E–1 Workers installing live willow stakes and brush matting on a terraced streambank, Winooski River, VT (September 1938)

Figure E–2a Streambank soil bioengineering techniques

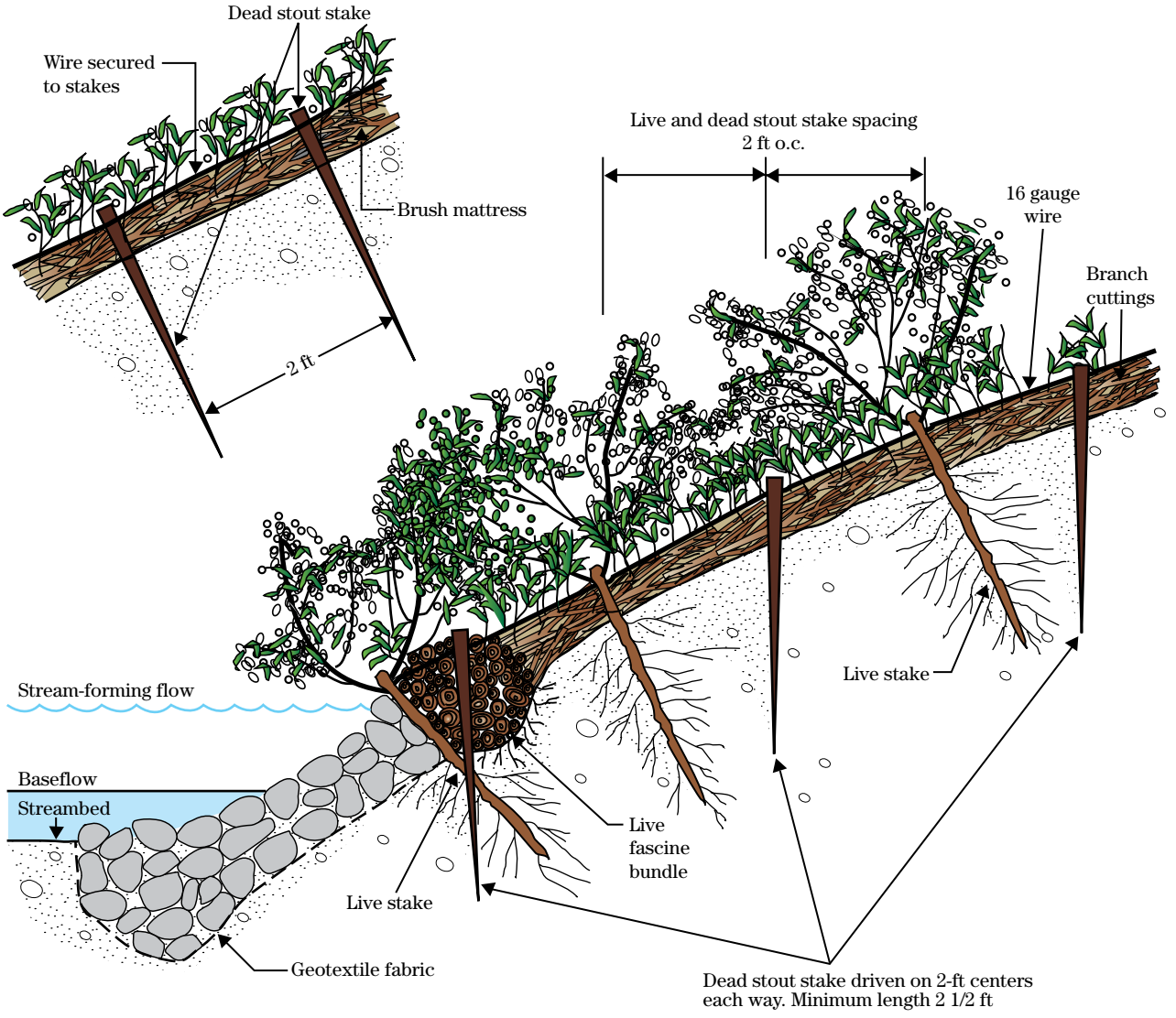
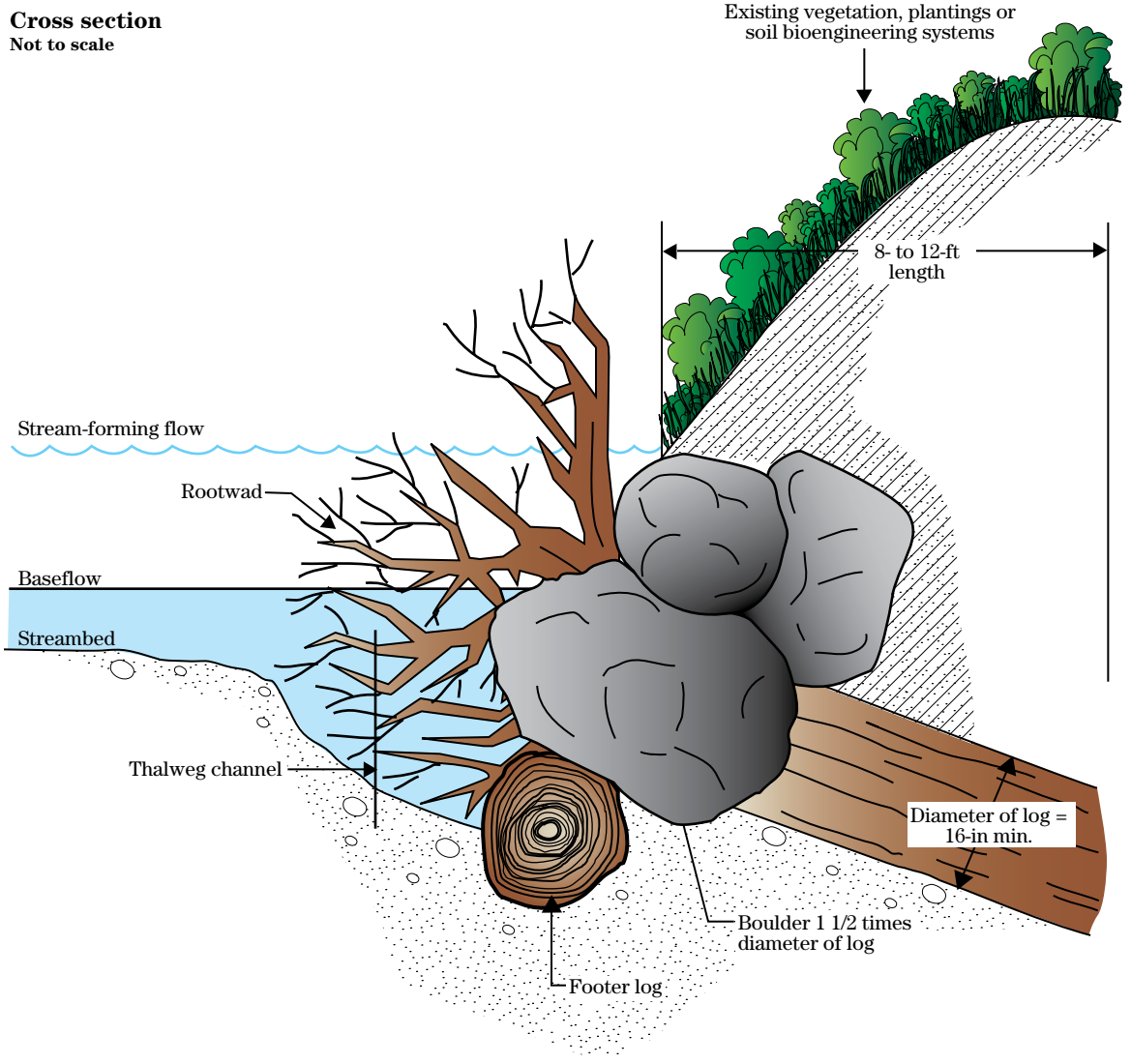


Figure E–2b Streambank soil bioengineering techniques—Continued

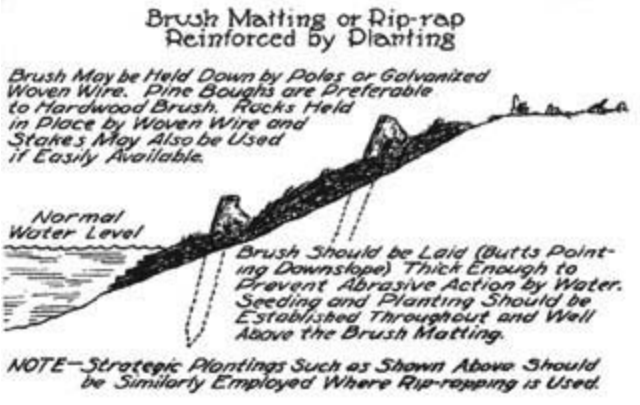


Except for the design details and materials used, the overall approaches are not new. Figure E–3 illustrates an early brush matting design detail that emphasizes plant materials for erosion control and bank stabilization.

The first chapter of this handbook provided an introduction to stream restoration and introduced four major guiding principles for any efforts to restore streams:

- Base designs on ecological principles, as well as physical ones.
- Integrate the disciplines of fluvial geomorphology, hydrology, aquatic and riparian ecology, and hydraulic and geotechnical engineering.
- Design for site-specific response in the context of the watershed scale.
- Consider ecological costs and values, as well as project costs, in addition to long-term costs for maintenance of engineered solutions to channel problems.

Figure E–3 Conceptual drawing that provided some design details for early restorations



Even more fundamental than these underlying principles is the question of what are the intended outcomes in working with stream systems. The needs of the stream must be satisfied, as well as the needs of people who are connected to the stream. Many river restoration projects are, at best, compromises between rehabilitation and restoration, since restoring some streams to historical, ecologically self-sustaining conditions may be impossible due to irreversible changes in watershed land use, cover, or other issues. Most river restoration projects are actually re-creations of the river to meet the changed needs of the watershed. Experience has shown that the most successful and cost-effective designs are those that are self-repairing and sustainable, with little or no need for future human intervention. Where feasible, this should be the primary goal of any river restoration.

A basic principle in stream related work must be to integrate the natural physical, biological, and chemical processes that shape stream systems. The primary question is, “Does this design element replicate what is found in the stream system, and will it result in a naturally functioning and self-repairing ecosystem?” The ultimate challenge is whether the restoration will satisfy these ecological needs, as well as the needs of the land user or community. The balance may be tipped heavily in favor of those who own land near the stream or are connected in some way to the stream. However, opportunities to plan for real restoration of the stream’s ecology exist, even where the first impulse is to just stabilize bank erosion with some enduring measures.

A design is most likely to result in a healthy aquatic or riparian community if the interdependence between flora, fauna, hydrologic and hydraulic characteristics, ground water, and soil are recognized.

Other conditions and variables must also be considered:

- What is the source of the problem? Some rivers are damaged beyond an eroding bank or an unstable reach. The real causes or destabilizing effects may be difficult to understand simply because the scope of disturbance may be so large.
- What are the natural flow conditions for the stream? How do the flow conditions affect the

ecological functions of the stream, and what is the natural flow regime that needs to be restored?

- How does human development affect the stream system? What are the stages of development and their effects on the watershed and its streams? What conditions or changes can be predicted in the near and distant future?
- Is the stream on its way to recovering, or is it continuing to degrade?

These and many other considerations make it difficult for planners and designers to predict appropriate restoration/rehabilitation measures. Nonetheless, the recognition that stream systems are dynamic—that they can be sensitive to changes, as well as resilient to others—should be an underlying principle for any stream work.

This handbook marks another incremental step in our growing understanding of the natural physical, biological, and chemical processes that shape streams and their corridors and the ability to work in harmony with them all. Although some stream work must be focused simply on conveying streamflow within boundaries designed not to move or erode, such as in urban stormwater drainage projects, opportunities abound to plan and design stream projects that improves the environment for plants and animals, as well as for people.

In closing, the Winooski River restoration project is again offered as an example of a long-term successful restoration (figs. E–4 through E–7).

When you put your hand in a flowing stream,

you touch the last that has gone before

and the first of what is still to come.

— Leonardo da Vinci

Figure E–4 Winooski River, VT, in 1938 just before restoration. Note the arrow for common point for following sequence of pictures.

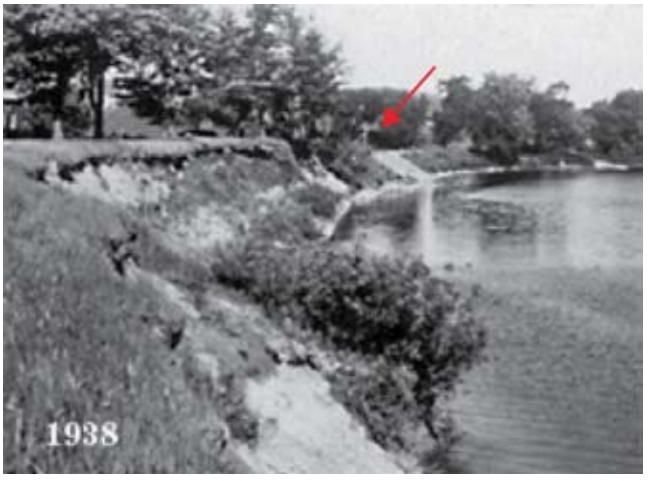


Figure E–5 Winooski River, VT, during construction in 1938. Note the work being done by hand.

Figure E–6 Winooski River in 1938 after completion of restoration work and establishment of vegetation.

Figure E–7 Winooski River in 1995, nearly 60 years after restoration. Tree-boring documented the trees as those that were planted in 1938

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Technical Supplements

Issued August 2007

Cover photos: *Top*—Tools and approaches addressed in the technical supplements provide designers with the ability to size and configure restoration elements and to provide the intended ecological functions.

Bottom—How big, how deep, how wide, starting and stopping points, transitional zones, and what live and inert materials to use are questions to be answered by the design process. Conceptual drawings are just that—concepts—until they are designed for specific site and performance conditions.

Introduction

During the development of this handbook, some specific design tools were recognized as being important for use in stream restorations. These tools are included in the following technical supplements, and they supplement specific handbook chapters. Some tools are specifically structural in nature. Their use is determined by the nature of the selected alternative: structural elements may be needed to achieve the intended project goals and objectives. Designers are nevertheless encouraged to achieve the intended outcomes of the project in an ecologically self-sustaining manner, including all appropriate management, nonstructural, and structural approaches.

Technical Supplement 2

Use of Historic Information for Design

Issued August 2007

Cover photo: Historical maps, aerial photos, and other information can be used to put the stream in context today. Rates of bank erosion, meander migration, channel width, riparian vegetation, and watershed land use and cover conditions can be estimated using historical information and may provide valuable data for the current design.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 2

Use of Historic Information for Design

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Technical Supplement 2

Use of Historic Information for Design

Introduction

This technical supplement describes the use of historic information in the assessment of stream and watershed form and process. Form and process of effects in one place, however, may be the cause of form and/or process elsewhere. The historic assessment of climate and land use can also address causes. This information can be invaluable in the determination and assessment of goals and objectives.

Basic data sources and techniques

The value of the historical approach is shown not only by the number of practitioners who use an historical basis in determining project goals but also by the number of methodological papers on this topic which have appeared in the last quarter century. The reader is directed to Thornes and Brunsden (1977); Hooke and Kain (1982); Gregory and Walling (1979, 1987); Grove (1988); Cooke and Doornkamp (1990); Trimble and Cooke (1991); Trimble (1998); Collins and Montgomery (2001); Trimble (2001); Brown, Petit, and James (2003); Gurnell, Peiry, and Petts (2003). While it is possible to identify some general principles of using historical data, it is impossible to give clinical directions on their use because every application is different; the reader should refer to the works listed above. Instead, this chapter presents some basic approaches using historical data and analyses of the study of stream and watershed forms and processes.

Contemporary descriptions

Written accounts take two basic forms. The first is a description of past events or changes, while the second is the description of contemporary or baseline conditions useful for later comparisons. One must consider the scientific credentials of the observer and the stage of scientific development at time of the observation. Accounts from scientific observers tend to be dependable and are often extremely helpful, especially when they relate to direct observations like the clarity of streams. Observations from less-qualified people also can be helpful, but may need more qualification or interpretation. Newspapers, periodicals, books, government records, and unpublished manuscripts are somewhat less valuable sources.

Weather events and climate

Recorded climatic records, especially regular records kept by governmental agencies, are often of exceptional value to stream studies. Figure TS2–1 shows an example of available precipitation data for the Driftless Area in the Upper Midwest, along with some analyses (U.S. Weather Bureau data; Trimble and Lund 1982). Official climate records extend to the mid-19th century in the United States, but scattered records were collected earlier. Particular storms may have significant geomorphologic effects locally or over larger areas, but little daily data are available. Cuts and fills in streambanks over time can also reflect the geomorphic history of a stream, where these data are available.

Stream and sediment discharge records—Stream discharge data for the United States extend to about 1850, are quite plentiful for this century, and most are easily available on the Internet. Suspended sediment data are available for certain streams, starting as early as about 1905. Web sites for various agencies with data on water quality and quantity are found in Ward and Trimble (2004) and described in NEH654.05.

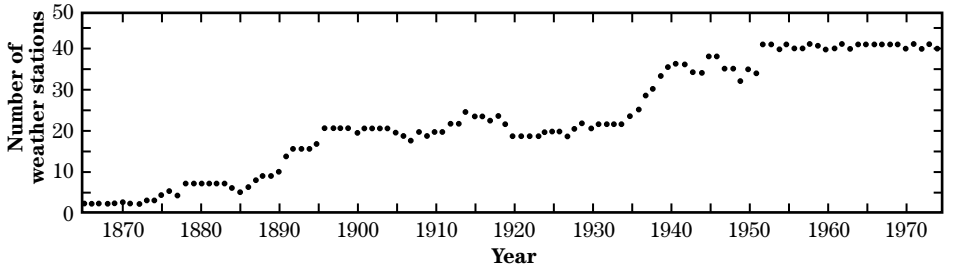
Land use

Land use has been increasingly recognized as a major causal factor in stream change. Reconstructions of land use need to be as precise as possible because increasingly sophisticated information about the hydrologic and geomorphic effects of different land uses and treatments are available. Historical land use may be correlated with contemporaneous geomorphic phenomena in model building.

Agricultural census data may be complex and confusing because categories and definitions often change from one census to the next. For the United States census, county enumerations are for “land in farms” only and sometimes cover only fractional parts of counties. Where available, the census manuscripts,

Figure TS2–1 Precipitation time trends in the Driftless Area of the Upper Midwest, 1867–1974. The sigma notation denotes 1 standard deviation from the mean. (a) Number of weather stations; (b) Average annual precipitation and time trends, (c) Relation of annual precipitation and storms exceeding 2.5 inches in 24 hours; (d) 3-yr moving average annual precipitation.

(a)



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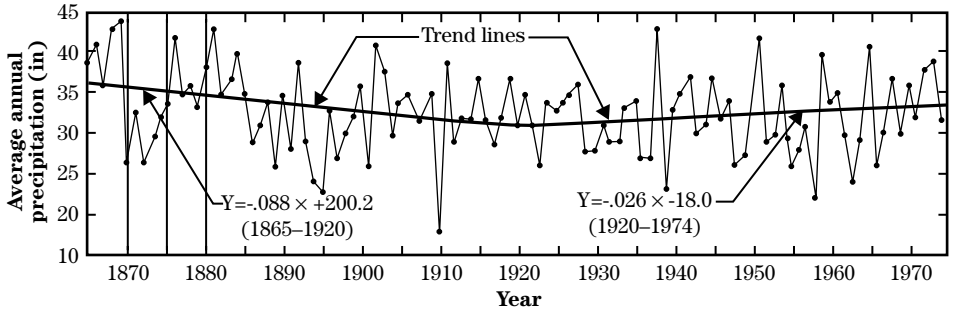

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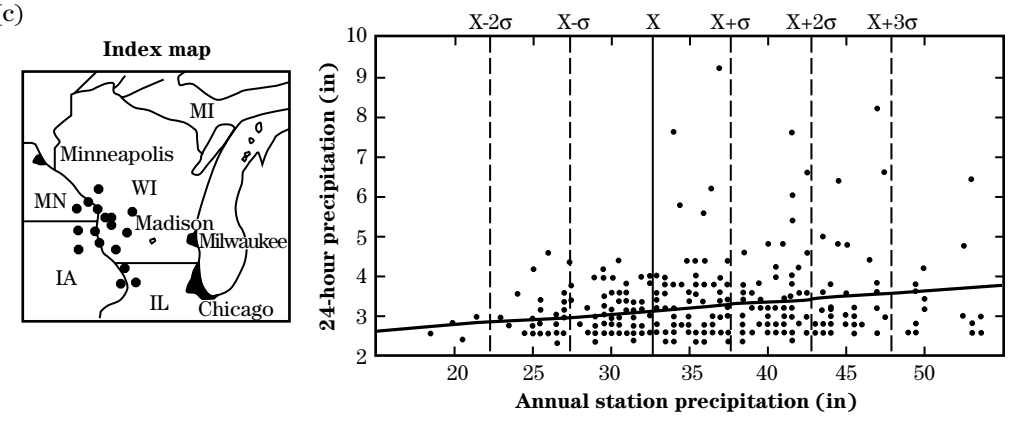

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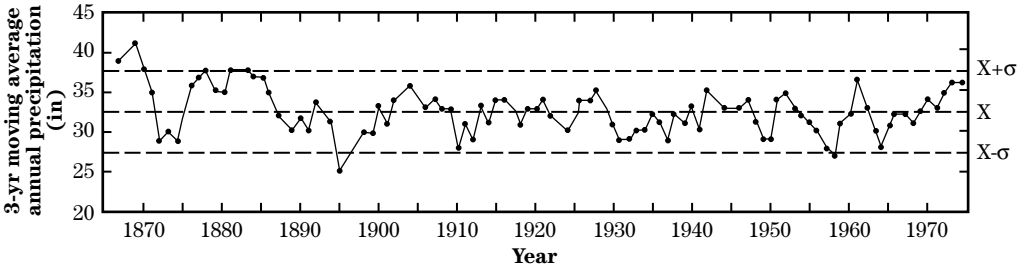


rather than the published reports, give far more detailed information. Another major problem is that areas of enumeration units change with time, so that the boundaries and areas must also be reconstructed (Trimble 1974). Unfortunately, as yet there are no guides to the use of census data in reconstructing historic land use, and a great need clearly exists.

Land use changes

Although not specific to a stream's local geomorphic condition, land use change information should be compiled. This can assist the stream restoration designer in ensuring that a proper analogue is selected for design work. If land use in a watershed has changed substantially, historic and even local geographic analogues may be inappropriate templates for design work. Changes in land use may be gradual, abrupt, or seemingly episodic over time, depending on climate fluctuations, population changes, and shifts in agricultural commodity markets, as illustrated in the example data set shown in figure TS2–2 (U.S. Census of Agriculture data, Trimble and Lund 1982).

Land (cadastral) surveys

While only planimetric land surveys can often supply important information to the fluvial geomorphologist. The original United States land surveys have been used to establish pre-agricultural flood plain conditions, upland vegetation, and stream widths. In other areas and in more recent periods, ongoing land resurveys sometimes give useful descriptions of geomorphological interest. Figure TS2–3 illustrates stream cross-sectional surveys and trends in watershed sediment delivery rates for the Coon Creek Basin in Wisconsin (Trimble and Lund 1982).

Figure TS2–2 Changes of land use, Coon Creek, WI, 1850–1975

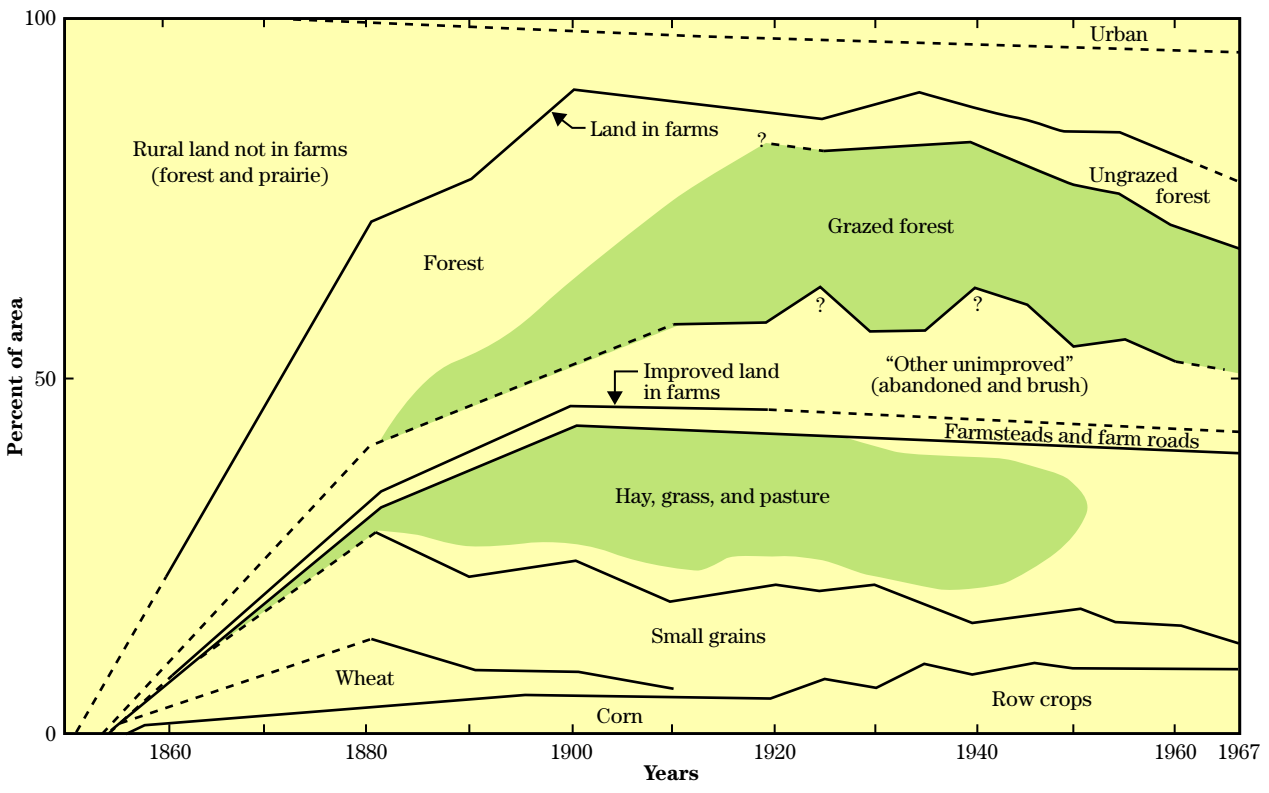
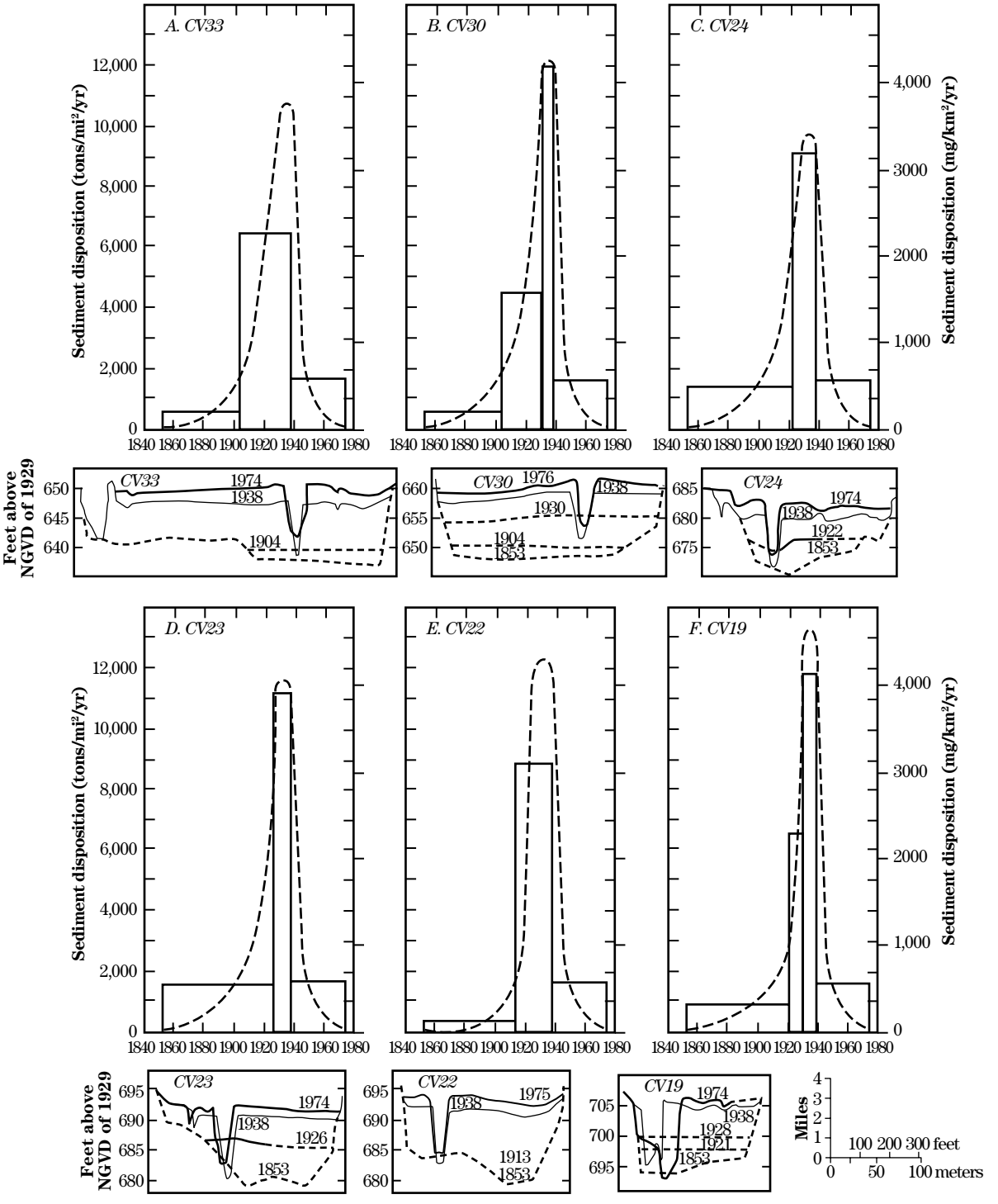


Figure TS2–3 Sediment deposition rates based partially on topographic surveys at selected valley sites in Coon Creek Basin, WI, 1853–1977



Historic condition

A stream's historic condition may be a useful target condition for physical restoration work if the causes of the degraded condition are local. The historic condition integrates many natural and human variables that controlled stream character at that past time. The historic condition can be an appropriate template for restoration design if these variables are relatively unchanged. Sources of information on a stream's historic condition include photographs and maps, written references and reports, people with long-time knowledge of the area, highway and railroad bridge data, and field evidence (table TS2–1). The more accurate information that can be obtained about the historic stream planform pattern, longitudinal profile, and cross-sectional dimension, the greater the information can contribute to development of a template for restoration work. The information about the historic condition, in most cases, is of limited accuracy will probably contribute only a part of the information necessary to develop a restoration design.

Field evidence

Where a channel has been substantially modified from its historic condition (for example, channelized) or relocated by people, some field evidence of the historic stream condition may be preserved in the form of natural surface topographic features, soil patterns, and vegetation patterns. Surface topographic features may help to characterize the historic stream and include relict channel sections, cut banks, levees, stream terraces, and flood plain wetlands. Flood plain wetlands, or wetland soils indicative of the former presence of wetlands, may also help to determine the stream's historic position and pattern. However, recent flood plain fill, alterations, or deposits may obscure remains of natural features. Riparian zone vegetation may obscure natural features, but leaf-off times would yield better results. Aerial photographs should be obtained to aid in locating these natural features if the stream reach to be restored is substantial in length.

Cultural features with documented locations may provide field evidence of the former position of the stream channel and bank prior to any changes in the stream's depth, cross section, or location. Bridge support and streambank stabilization structures that were constructed in or immediately adjacent to the stream may provide particularly good information. In urbanized areas, gravity-fed sewer system pipes and manholes typically follow stream valleys. Exposure of subground portions of these features can provide dramatic evidence of stream position change, with exposure of subground instream cultural features being strong evidence of downcutting. Where numerous cultural features provide evidence of a stream's former vertical and or horizontal position, the more likely that the watershed condition has changed substantially over time and the less likely that these features would be useful as historic analogues, since stream hydrologic condition may have changed substantially since their construction. Local cultural features may also have been the cause of current degraded conditions. If so, it is important to identify potential cultural features that could impact stream conditions and collect sufficient information to determine whether or not a stream is still responding to altered conditions caused by these constructed features.

Table TS2–1 Sources of historic condition information

Source of historic condition information	Aerial photo	Map	Descriptive accounts	Surface photo	Field evidence	Survey notes	Bridge design data
Planform pattern	✓	✓			✓	✓	✓
Channel dimension			✓	✓	✓	✓	✓
Longitudinal profile			✓	✓		✓	✓

Litter

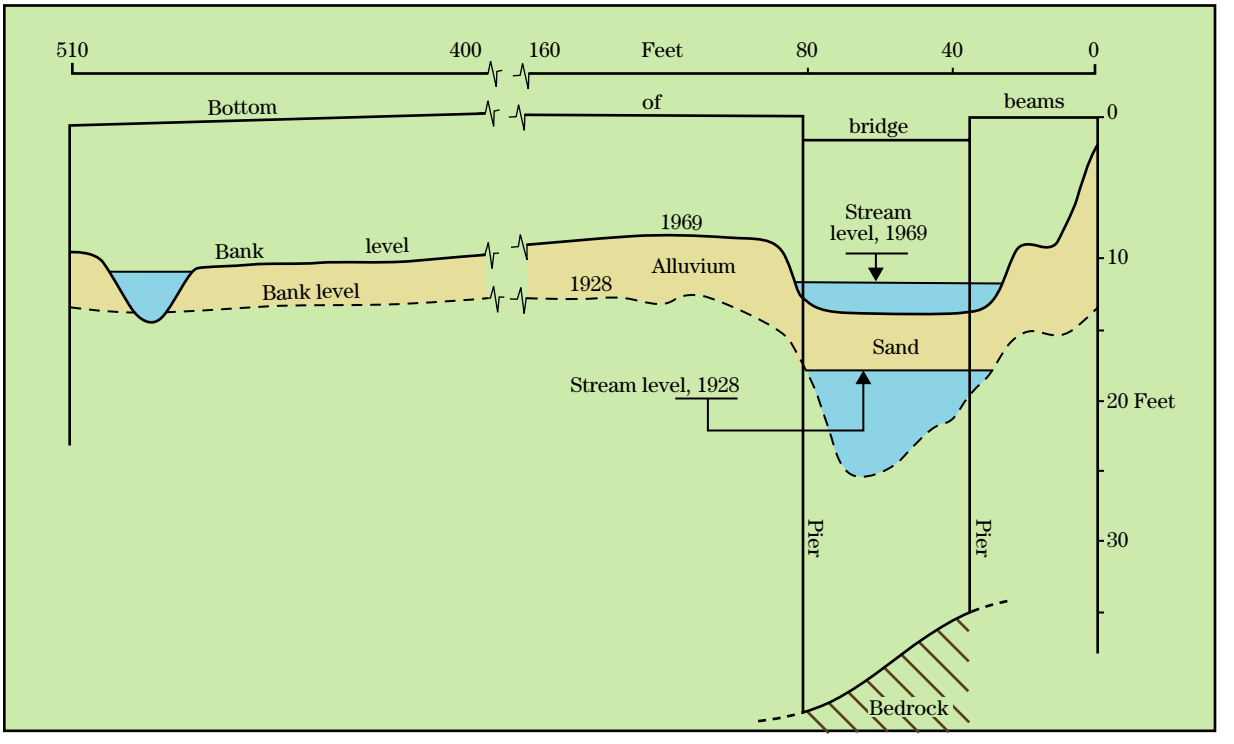
Litter includes mobile artifacts such as tools, vehicle parts, bottles, cans, package wrappers, and any other dateable artifacts. Although litter may be precisely dated in some cases, its location can only give an earliest possible date. For example, a can dateable to 1939 may have been dumped into a stream in 1950 where it was later buried in a point bar in 1952. The point bar may have eroded away in 1960 and the can buried 20 centimeters deep on a downstream flood plain. The only allowable conclusion is that there has been a minimum accretion of 20 centimeters at the final location since 1939. However, finding several items at similar levels with similar dates might allow a stronger inference. For example, an intact dump located in a flood plain with several items of similar dates could be valuable.

Permanent landscape features

Common, permanent landscape features including bridges, dams, mills, reservoirs, fords, fish traps, roads, canals, causeways, and buildings can sometimes act as gages to assist in measuring fluvial change.

Bridges—Inspection of older bridges by a practiced eye can often yield immediate information about stream processes. Reduced cross-sectional flow area under the bridge may indicate that the stream is aggrading. Burial of structural members (wingwalls) that are usually exposed to the stream may also indicate aggradation. Degrading streams, on the other hand, can often be diagnosed by old water lines left on structural members by exceptionally large openings and by the exposure of structural members (footings, pilings) which are usually placed beneath the water surface. Figure TS2–4 shows changes in stream morphology based on comparison of bridge plans with new survey data (Trimble 1970b).

Figure TS2–4 Cross sections of Middle Oconee River at Highway 11 bridge, Jackson County, GA, showing stream aggradation between 1928 and 1969, based on bridge plans and resurvey



Often of greater value are bridge plans. These usually include a stream and valley cross section surveyed before bridge construction that can be resurveyed for comparison. In comparing profiles, it must be ensured that the present bridge has not induced local scour or deposition, which would make comparison difficult or invalid. Often, plans include a detailed topographic map of the stream reach, and some plans may also include surveyed cross sections at some distance upstream or downstream. The latter surveys are particularly valuable because they are less affected by scour effects induced by some bridges.

Often, several bridges were built and rebuilt at the same site. Highway bridge plans in the United States often go back to the turn of the century and railroad plans to the mid- to late 19th century. Fortunately, the same vertical datum is normally used for successive bridges at a site, so that resurveys of century-old profiles are possible. Even when a datum changes, parts of the old bridge may be excavated to reestablish elevations.

Bridges are usually inspected periodically by government agencies. The resulting reports usually contain considerable description and measurements of site conditions and often include photographs that allow time-lapse photography.

Dams, *mills*, *reservoirs*, *fords*, *and fish traps*—Changes in streams often create severe problems in the operations of mills and reservoirs, and such problems may be documented. While nearly all water-powered mills had reservoirs, most of these were channel-type pools that had very low trap efficiency for sediment. More fortunately, some mills, and later hydroelectric and flood-control dams, have a large volumetric capacity in relation to their drainage area and, therefore, have a high trap efficiency so that sediment yield can be measured with some confidence. These data have the advantage of often being long term and include normally unmeasured sediment (bed load), but caution must be taken to consider the sediment from shoreline erosion. Many reservoirs are resurveyed periodically. These survey data can be obtained from government agencies or the people who surveyed the reservoir.

Figure TS2–5 shows a series of detailed surveys of sediment accumulated over time behind a gully-plug dam in Wisconsin (Trimble and Lund 1982).

Figure TS2–6 shows the variability in sediment production rates in watersheds in the Tennessee River Basin (Trimble and Carey 1992).

Roads, *canals*, *and causeways*—Roads (including railroads) and causeways serve as benchmarks to measure changes in stream morphology and process such as lateral movement of streams or gullies. When the location of a road or canal in relation to the stream can be determined from documentary evidence such as old maps or aerial photographs, its present location will allow an average rate of lateral migration to be calculated. Authorities will normally take whatever measures are necessary to protect a road or canal affected by a laterally migrating stream. Some sort of documentation is normally prepared, often with plans and maps. Structures put into place then act as benchmarks to measure future stream movement. Other road protection structures useful for future measurement are dikes, levees, and riprap. Additionally, roads are often raised by fill above normal flooding or aggrading flood plains, and the level of the old road beneath may be determined from construction plans, borings, or excavations.

Buildings—Except for occasional mills, buildings are rarely constructed on active flood plains and even when close to streams, they are usually sited on terraces. Thus, when a structure is affected by sediment, it indicates important changes in the stream or in sediment regime. Generally, when a building is significantly impacted by water and/or sediment, steps are taken to move or raise the building, if possible. If not, the building is usually dismantled, leaving only the foundation, which itself can serve as a benchmark. Once the foundation has been covered with sediment, however, the location must be established from old maps, land plats, eyewitness testimony, or even subsurface radar. Likewise, the chronology must be established from maps, land survey plats, tax records, or eyewitness accounts. Unlike roads and causeways, which may be located by borings, it is best to excavate around as much of the building as possible because it is necessary to see how the building's occupants interfaced with the stream. For example, did an entrance face the stream? Artifacts between the building and the stream such as steps, walks, fences, and small outbuildings would imply that the area was frequented by people at one time, thus implying low frequency of flooding. Buildings also may be occasionally useful for measuring stream channel erosion.

Figure TS2–5 Headwater erosion rates based on sediment accumulation behind a small dam, Vernon County, WI, 1936–1977

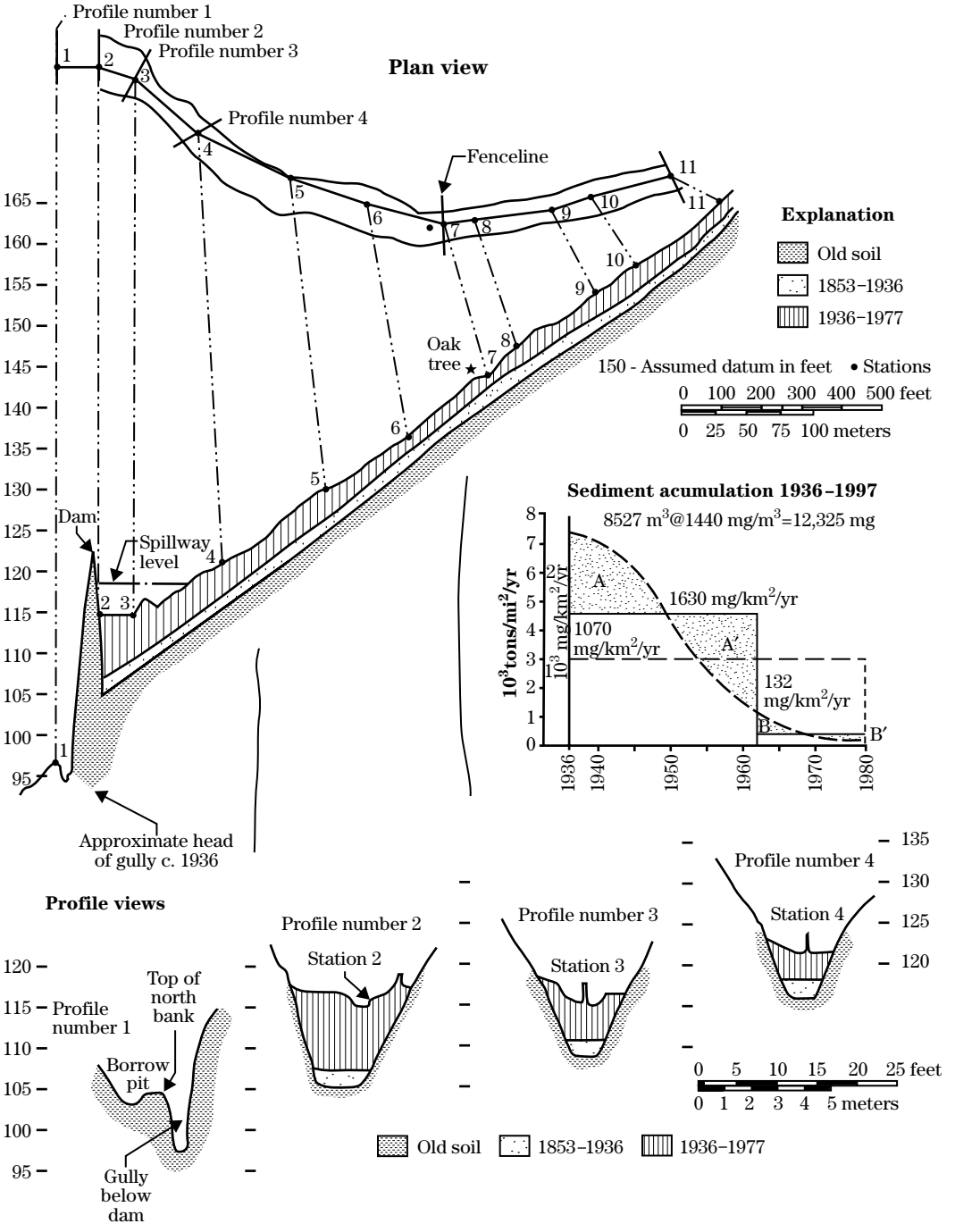
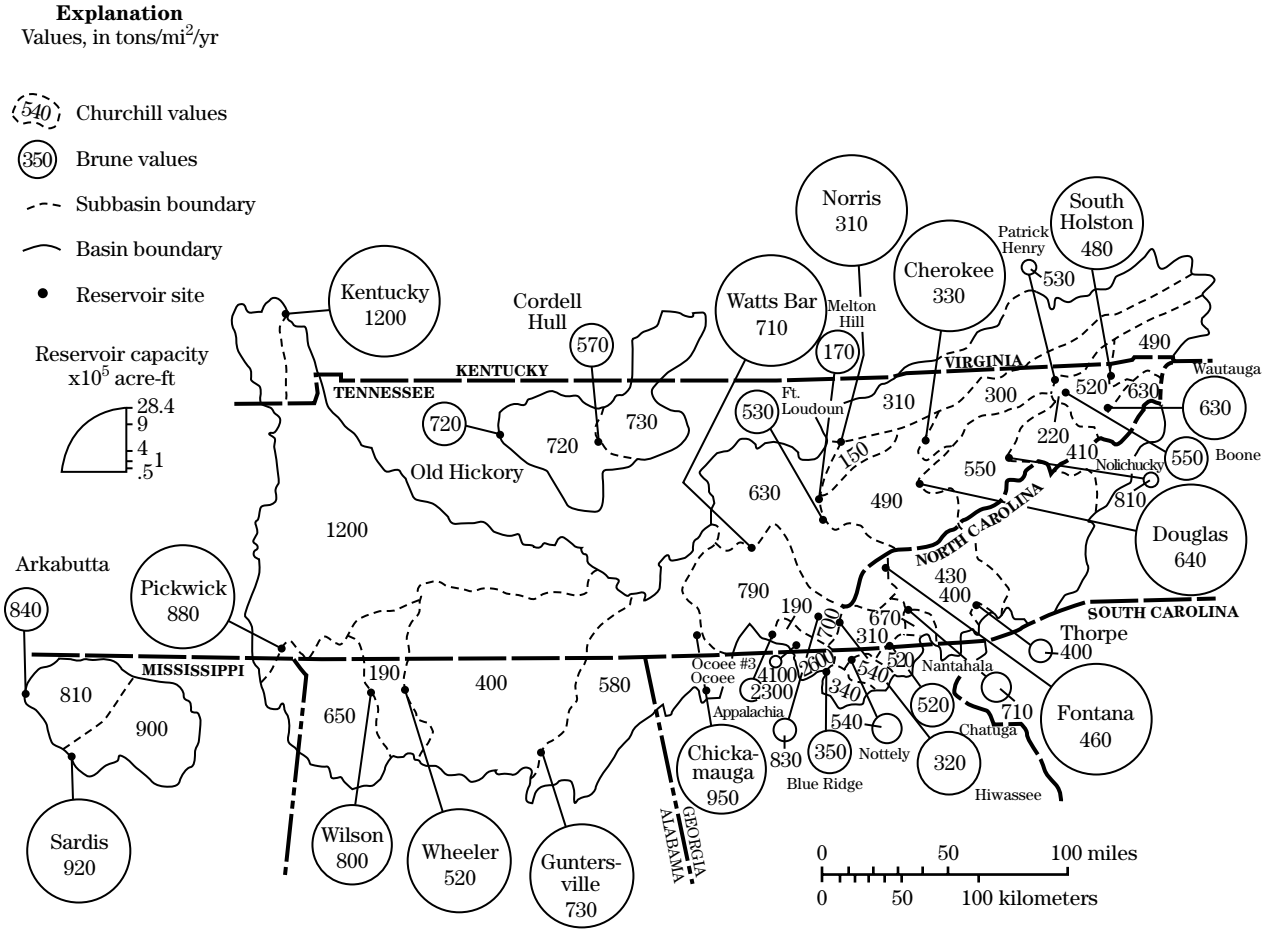


Figure TS2–6 Sediment yields for the Tennessee River Basin c.1940–1975, based on TVA reservoir surveys



Historic maps

Historical maps can be an invaluable source of site-specific information to characterize past stream conditions. Historical maps are housed in many different collections, including libraries and historical societies, as well as local, state, and Federal government agency offices. For many areas of the United States, U.S. Geological Survey (USGS) maps are the oldest accurate maps available. USGS maps date from 1879 when systematic mapping of the country was begun in the West. Web sites of the USGS, National Archives, and Library of Congress are particularly valuable sources for locating older maps. The USGS Web site also contains information on mapping standards.

Information on past stream condition that can be derived from historic maps is limited by map scale, accuracy of original survey work, and climatic controls on stream character. In many areas, USGS topographic maps are often the largest scale historic maps available. Because USGS topographic maps are produced at infrequent intervals, only very long-term trends can be determined. This may prevent a detailed understanding of the effects of short-term physical processes and stream morphological responses.

Before the advent of aerial photography, streams were sketched in the field. The USGS generally mapped streams when water levels were at normal stage. The sketching of shorelines of broad rivers, however, was a perplexing problem due to periodic fluctuations in width. Riparian vegetation and other features also reduced map accuracy, depending on the date of observation. USGS map accuracy improved with the use of aerial photography beginning in the 1930s. Map accuracy further improved following establishment of national accuracy standards in the 1940s and the establishment of better horizontal control features in the 1950s. Since the 1940s, national map accuracy standard requires that 90 percent of defined test points are within 40 feet of their true horizontal position (large scale USGS maps, 1:24,000 scale). Older USGS maps were not subject to this standard. Currently proposed standards require that definite streams be depicted within 0.02 map inches (40 ft at 1:24,000 scale) of their horizontal position.

The mapped accuracy of stream width by the USGS also depends on mapping conventions related to stream width. Before 1954, USGS maps depict streams as double lined only when actual width could be displayed without exaggeration. In 1954, USGS adopted a standard whereby the minimum stream width required for depiction using double lines on a map was 40 feet for 7.5-minute maps and 80 feet for 15-minute maps. These criteria, however, had probably been widely used for several years beforehand. In 1993, the minimum width requirement for a stream to be depicted as a double line on 7.5-minute maps was increased to 50 feet. Streams narrower than this width criterion are depicted as single lines. On USGS maps from the 1800s through the 1950s, streams mapped as single line were depicted as tapering to become narrower towards the headwaters, and small side tributaries were depicted using a smaller weight single line than the mainstream. However, this was done to connote that the stream width decreases proceeding towards the headwaters, rather than to map specific stream widths. From the 1950s onward, all streams too narrow to meet the double-line width criterion are depicted as single bluelines of the same width, regardless of their actual width. Therefore, it is not possible to accurately determine the width of streams mapped as single lines on historic or current USGS maps.

In the vicinity of an engineered feature, such as a road or railroad, mapmakers often displace natural features slightly to allow for depiction of both engineered and natural features in a space on the map otherwise too small to permit both to be mapped to scale. Thus, this displacement possibility should be considered when using maps as sources of information about historic stream planform and width in vicinity of engineered features.

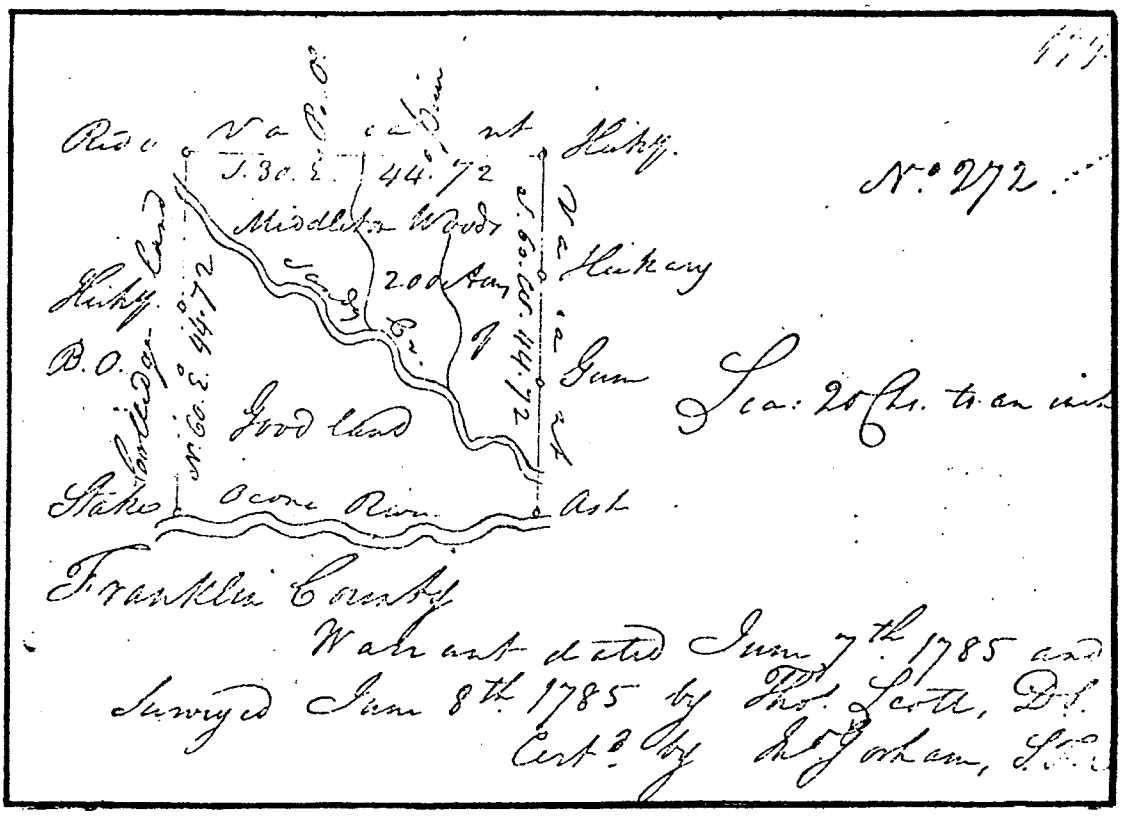
Positions of streams in arid regions are often relatively indefinite, as a consequence of infrequent flow conditions. In these regions, it would be inappropriate to scrutinize historic maps for specific channel location information.

Historic USGS maps can provide only limited information on elevational and longitudinal profile changes. National map accuracy standards established in the 1940s require that 90 percent of tested elevations on all USGS contour maps on all publication scales lie within a half the mapped contour interval. Currently proposed standards require that definite streams be depicted within a half contour interval of their verti-

cal position on USGS maps. In mountainous regions, the wide footage between contour intervals generally limits the ability to detect stream elevation and longitudinal profile changes, where substantial change has occurred. In flat areas, the great lateral distance between contour intervals typically limits the ability to accurately determine elevation changes over time. The range in potential change, however, is inherently much less.

Historic survey maps and associated notes can also be valuable sources of information. Surveys provide far greater detail than do regional maps. Streams were important resources to property owners, and creeks and rivers often form property boundaries, increasing the likelihood that valuable information on stream condition may be recorded in property surveys. Local governments, libraries, and local historical societies may also have historic property survey information. Surveys conducted in association with construction of structures over and along a stream by private individuals, commercial enterprises, and government agencies are of particular value. Government agencies involved in streamside projects potentially include highway departments, water supply and sanitary sewer agencies, the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), U.S. Department of Interior (DOI) Bureau of Land Management (BLM), and USACE. The agencies involved in these past activities can be contacted for potential historic survey information that they have on file. Figure TS2–7 illustrates how the historical condition can be compared to the current condition, based on old land survey maps and information (Trimble 1970b). Because of stream aggradation, the area noted as good land in 1785 was swamp by the 20th century.

Figure TS2–7 Original land survey plat of the confluence of Sandy Creek with the North Oconee River, dated Jan. 8, 1785



Instrumented topographic surveys

While not available for many locations, topographic surveys may be the best quality data available. In this category are general topographic maps, which have existed in the United States for more than 100 years, created mostly by the USGS; precise river surveys, usually by the USACE; detailed maps of coastal areas including stream and estuaries with bathymetry, usually by the U.S. Coast and Geodetic Survey; stream stage-discharge studies by USGS and other agencies; and flood studies by various agencies.

Aerial photographs

The first aerial photographs available date from the 1800s when photos were taken from balloons and kites. Geographic coverage by these photos, however, is very limited. Aerial photography became widely practiced in the 1930s. The Library of Congress maintains a collection of aerial photographs taken between the early 1900s through the 1940s, and information on how to obtain these can be obtained at the Library of Congress Web site. Photographs taken by Federal agencies from the 1930s through 1940s for mapping purposes that cover approximately 80 percent of the area of the lower 48 states are available through the National Archives. Information on how to obtain copies of these photos is on the National Archives Web site. The USGS maintains a collection of aerial photographs taken since the 1940s. Information on obtaining copies of these photos can be obtained by visiting the USGS Web site. Streams located within cropland regions would also likely be included on aerial photography conducted by the USDA. Aerial photography dating from the 1950s is available from the USDA Aerial Photography Field Office. High quality aerial photographs are taken of nearly the entire country every 5 to 7 years by the National Aerial Photography Program. These photos are available from 1987 onward through the USGS.

Figure TS2–8 shows dramatic watershed land use changes and changes in streams and drainage patterns between 1934 and 1967 (Trimble and Lund 1982). Note the rectangular fields in the old system and the contour strip farming in the new. Note also the great decrease of drainage density (gullies) resulting from better land use and decreased overland flow.

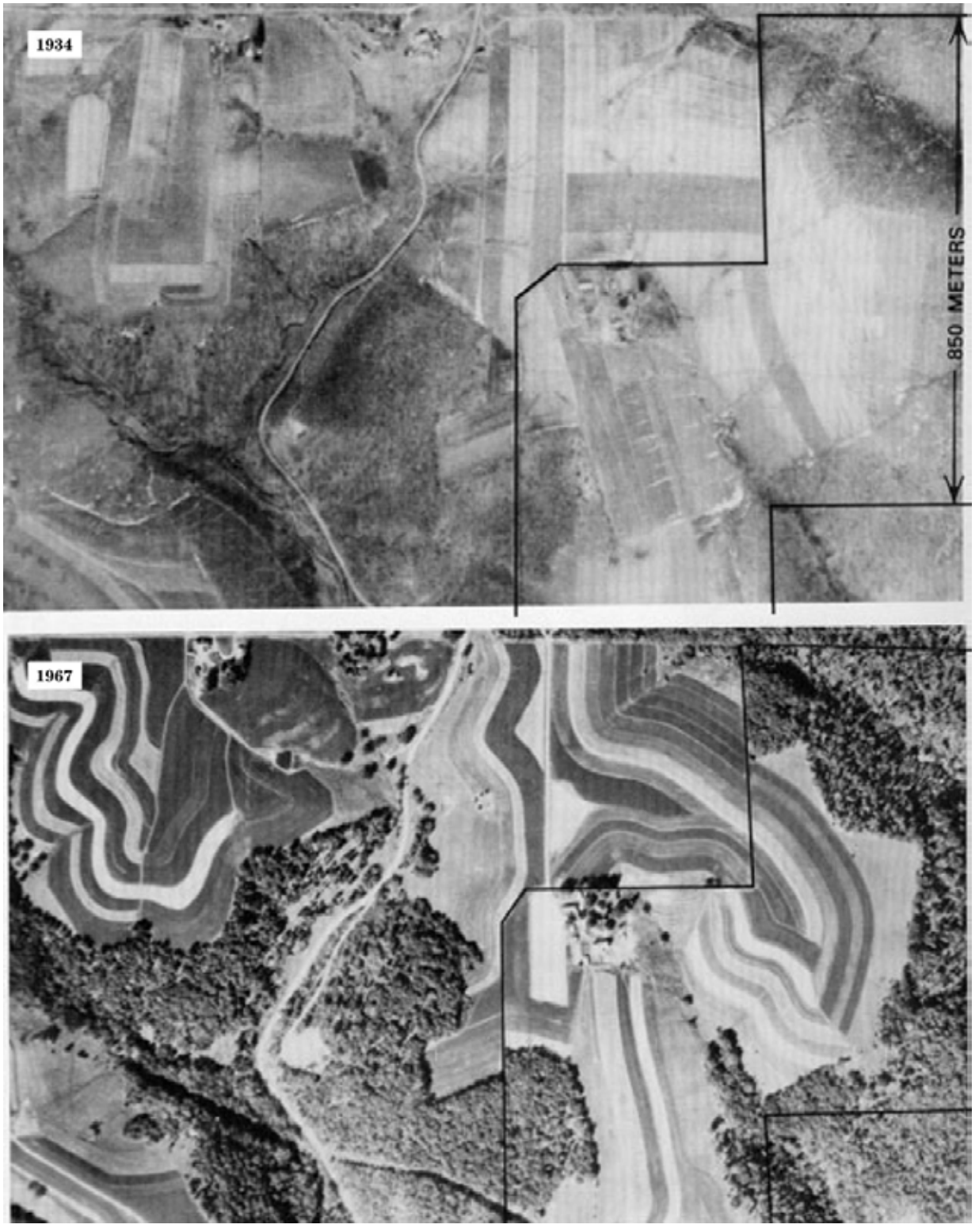
Because of the great number of factors affecting the resolution of aerial photographs, there is no rule of thumb guiding the minimum linear dimension that can be resolved on historic aerial photographs. Major factors influencing aerial photograph resolution include atmospheric conditions, ground conditions, aircraft movement, lens character, film character, camera height (flying height), camera position with respect to the Earth's surface, camera quality, and whether the film was black and white or color. For the USGS, camera height was determined primarily by the desire to compile contours accurately for mapmaking. If the height was appropriate for contour mapping, it was generally good enough to map planimetric features including streams. Historically, color film was much grainier than black and white, further limiting the resolution of historic color aerial photographs. Also, because of potential variation in scale across an aerial photograph due to distortion, information on historic stream condition is of greatest value when taken from a relatively small area of any given photograph.

Aerial photographs, dating from as early as 1917, have been used to demonstrate and date fluvial changes, along with the land use changes which were responsible for those stream changes. The general coverage of stereographic aerial photography in the United States dates from 1937 to 1938, but limited coverage exists from circa 1925. The value of aerial photography is a function of scale, photographic quality, and availability of stereographic coverage. Stream and valley aggradation is difficult to detect on air photos, but some attendant effects, such as the creation of backswamps and vegetational changes, can be seen and measured. On upland areas, air photos can be used to quantify land use and consequent erosion.

Ground-based oblique photography

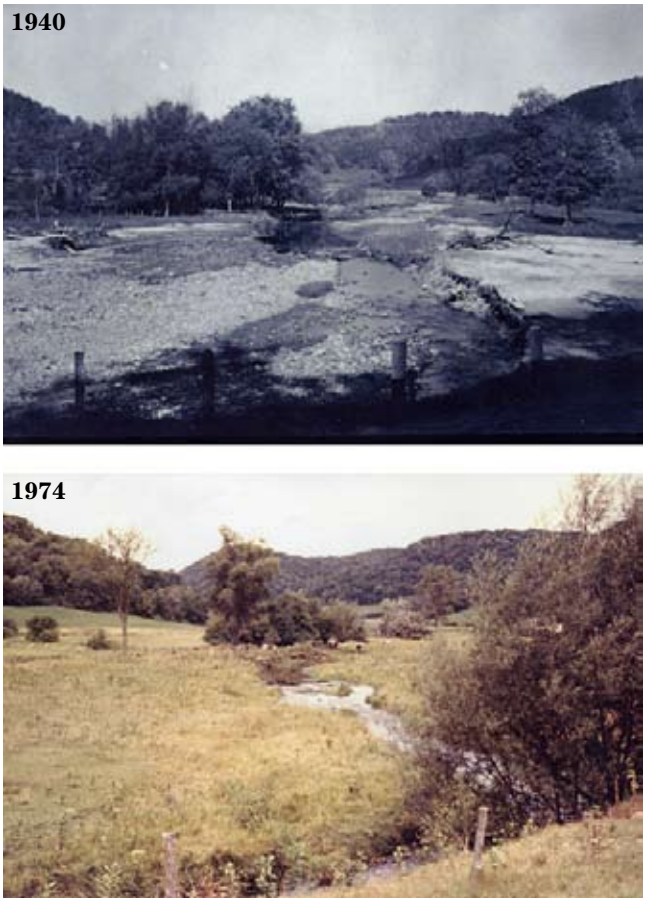
Ground-based oblique photography has been used to date geomorphological processes dating back well into the 19th century. Figure TS2–9 shows an example of the use of such pictures taken at different dates. The 1940 photograph in figure TS2–9 shows a typical tributary in 1940. Note the eroded, shallow channel composed of gravel and cobbles, with coarse sediment deposited by overflows on the flood plain. Such tributaries were de-

Figure TS2–8 Air photos showing changes in land use in Vernon County, WI, between 1934 and 1967



scribed as resembling gravel roads. The 1974 photograph in figure TS2–9 is a remake of the 1940 photograph. The stream channel is narrower, smaller, and more stable. The coarse sediment has been covered with fine material, and the flood plain is vegetated to the edge of the stream. This condition has continued and improved over the past 30 years (Trimble and Crosson 2000). All data are from the U.S. Weather Bureau (Trimble and Lund 1982). Although not as systematically available as aerial photography; ground-based photography has existed longer and generally offers better scale and resolution for time-lapse comparisons. Many major repositories of such photographs exist, but queries should always be made to museums, libraries, and individuals. In some cases, photogrammetric techniques can also be used with oblique photography, making it possible to make precise measurements.

Figure TS2–9 Oblique ground photos showing improvement of tributary streams in Vernon County, WI, 1940 and 1974



Photographs of site

Stream sites of interest by government or commercial interests following the advent of photography in the mid-1800s may have been captured in historic photos. Local residents may also have photographed the stream. These photos offer the advantage of potentially being relatively large in scale in comparison with maps and historic aerial photographs and can provide detailed local information to aid in interpreting changes in fluvial geomorphology over time. Historic photos may show local changes in depositional and erosional features in stream reaches and provide information on stream corridor character and human activities and land use. Unfortunately, locating these photographs can be difficult. Government agencies, local historical societies, and long-time area residents can aid in locating historic photographs. The Library of Congress maintains a national digital library from more than 100 historical collections that might also be worth reviewing.

Descriptive accounts and interviews with residents

Streams in long-settled areas or areas with major historic flooding events or other natural disasters impacting people may have been described in historic accounts. Local libraries can be good sources of this information. This information can be of particular use in cases where a stream condition is still evolving in response to a past disturbance, for which no obvious evidence is readily apparent. People familiar with the stream in the past, particularly long-term area residents that have a mental record of stream evolution, can often provide qualitative information on change in stream character.

Summary and conclusions

Although not always precise, historical data and techniques can provide powerful tools for establishing watershed conditions and stream forms and processes over the past few centuries, particularly during the last few decades. Comparative information can be used to determine the character of watershed and stream changes and may sometimes provide important quantitative measurements to support stream restoration designs.

Technical Supplement 3A

Stream Corridor Inventory and Assessment Techniques

(210–VI–NEH, August 2007)

Issued August 2007

Cover photos: *Top*—Measuring the physical attributes of the stream is a critical step in the planning process. Assessing the ecological condition of the stream, its riparian area, flood plain, and watershed are also as critical as setting restoration goals and objectives.

Bottom—The challenge in restoring streams is to determine the current ecological condition and to project anticipated changes due to the restoration project.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 3A

Stream Corridor Inventory and Assessment Techniques

Introduction **TS3A–1**

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Technical Supplement 3A

Stream Corridor Inventory and Assessment Techniques

Introduction

This technical supplement contains an inventory of assessment techniques. This material was developed by an interdisciplinary team composed primarily of U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) employees (USDA NRCS 2001b). This guide provides the titles, reference citations, descriptive summary, and attributes of a number of stream corridor inventory and assessment techniques that are suitable for local conservation programs. Such programs are typically pursued at the site or project level, with increasing attention given to the landscape scale to optimize future treatments, management, and monitoring. The purpose of this guide is to aid in the selection of the appropriate inventory and assessment techniques to determine the conditions of their stream corridor. It is intended that this material possibly be used to supplement the information provided in this handbook.

The methods contained in this technical supplement are listed in table TS3A–1 along with their attributes. The table provides a description of principal features and a comparison of the techniques. Techniques are grouped by the primary stream corridor setting to which they pertain and are arranged in alphabetical order. Standard dictionary definitions for terms are assumed unless otherwise noted. Explanations of attribute ratings (columns 1–6 of table TS3A–1) are:

- The ***Primary Setting*** that the particular technique addresses (many techniques are used for additional primary or secondary settings):
  - — ***Channel—flood plain***
  - — ***Riparian area***
  - — ***Water quality (properties, contaminants)***
  - — ***Aquatic habitat***
- The ***Sampling Intensity***:
  - — ***Cursory*** (preliminary: observations and estimates of conditions and attributes are made usually without the need for specific measurements or quantification)
  - — ***Detailed*** (comprehensive: conditions and attributes are itemized and specifically measured)
- The required ***Skill Level, Training,*** and ***Time*** to properly carry out the technique are rated as ***High*** (Skill level: specialists with considerable specialized expertise; Training: 3 to 5 days; Time: generally 4 or more hours per site), ***Medium*** (Skill level: specialists with basic specialized expertise; Training: 1 to 3 days; Time: generally 1 to 3 hours per site), or ***Low*** (Skill level: professionals or technicians trained in the technique; Training: 1 day or less; Time: usually less than 1 hour per site).
- The techniques are classification by ***Kind*** (***Inventory***—a collection of data or ***Assessment***—a collection of data and value judgment as to condition), ***Measure Type*** (***Qualitative***—using charts, tables, attribute groupings or illustrations to classify or rate, or ***Quantitative***—measurements, dimensions, quantities) and ***Proximity*** (***Onsite***—observers or data collectors physically at the site, or ***Remote***—observers or data collectors can use satellite imagery or aerial photos).
- The need for a ***Reference Site*** (***Yes, No,*** or ***Optional***)—a reference site is a representative segment or reach of a stream corridor system in dynamic equilibrium with a relatively undisturbed watershed.
- The technique's ***Suitability for Monitoring*** (***High***—suited for statistical analysis with consistent results between different collectors at the same site and accurate detection of change or trend over time, ***Medium***—reproducible or repeatable results, but generally not suited for statistical analysis, or ***Low***—not intended for monitoring purposes).

The ratings for the attributes in table TS3A–2 were developed by a team of interdisciplinary specialists with experience in stream corridor inventories and assessments. For each technique, a full citation, source address, and a brief summary are provided. Readers are encouraged to obtain and test the techniques that appear promising for their settings and requirements.

Table TS3A–1 Attributes of stream corridor assessment techniques

Column notes listed below >	1	2	3	4	5	6
Technique (to obtain a technique's citation and summary, turn to the page number listed in parentheses)	Primary setting (listed first)	Sampling intensity	Skill level, training, time	Kind, measure type, proximity	Reference site needed	Suitability for monitoring
Primary setting—Channel flood plain						
Applied River Morphology. Wildland Hydrology Consultants. D. Rosgen. 1996. Pagosa Springs, CO (14)	C	D	H–H–H	I/A–N–O	Y	M
Channel-Reach Morphology in Mountain Drainage Basins. Geological Society of America Bulletin. D.R. Montgomery and J.M. Buffington. 1997 University of Washington, Seattle, WA (14)	C	C	M–M–M	I–L–O	O	M/H
Incised Channels–Morphology, Dynamics, and Control. S.A. Schumm, M.D. Harvey, and C.C. Watson. 1984. Littleton, CO (16)	C	C	M–M–L	I–L–O	N	M
Procedures for Using Oregon Stream Habitat Data Sheet. USDA NRCS. 1988. Portland, OR (19)	C, R, A	D	M–M–L	I/A–L/N–O	N	M
Rapid Stream Assessment Protocol (RSAT) Field Methods–Appendix A. J. Galli, Sr. 1996. Metro. Washington Council of Governments, Washington, DC (21)	C, R, W, A	C	M–M–L	A–L–O	Y	L
Stream*A*Syst. Oregon State University, Extension Service. 2000. Corvallis, OR (30)	C, R, W	C	L–L–L	A–L–O	N	L
Stream Channel Reference Sites: An Illustrated Guide to Field Technique. USDA Forest Service. 1997. Fort Collins, CO (26)	C	D	H–H–H	I–N–O	Y	M
Stream Corridor Assessment Survey. K. Yetman, MD Dept. of Natural Resources. 2000. Annapolis, MD (26)	C, R, A	C	M–L–L	I/A–L–O	N	L
Stream Inventory Handbook–Level I and II. USDA Forest Service. 1996. Version 9.6. Portland, OR (27)	C, R, A	D	M–M–H	I–N–O	O	H
Streamkeeper's Field Guide–Watershed Inventory and Stream Monitoring Methods. The Adopt-A-Stream Foundation. 1966. Everett, WA (27)	C, R, A, W	D	M–M–M	I/A–L/N–O	Y	M/H
Stream Visual Assessment Protocol. USDA NRCS. 1998. Portland, OR (28)	C, R, W, A	C	L–L–L	A–L–O	N	L

Table TS3A–1 Attributes of stream corridor assessment techniques—Continued

Column notes listed below >	1	2	3	4	5	6
Technique (to obtain a technique's citation and summary, turn to the page number listed in parentheses)	Primary setting (listed first)	Sampling intensity	Skill level, training, time	Kind, measure type, proximity	Reference site needed	Suitability for monitoring
Primary setting—Riparian area						
Guidebook for Application of Hydrogeomorphic Assessments to Riverine Wetlands. U.S. Army Corps of Engineers, Waterways Exp. Station. 1995. Washington, DC (15)	R	D	H–H–H	A–L/N–O	Y	M
Integrated Riparian Evaluation Guide. USDA Forest Service. 1992. Ogden, UT (Level I) (Level II) (Level III) (16)	R, C, A R, C, A R, A	C D D	M–M–L H–H–M H–H–H	I–L–R I/A–N–O I/A–N–O	N	L
Methods for Evaluating Riparian Habitats with Applications to Management. USDA Forest Service. 1987. Ogden, UT (17)	R, C	D	H–H–H	A–N–O	N	H
National Forestry Manual: National Range and Pasture Handbook (Procedures for completing Vegetation Field Forms and Ecological Sites). USDA NRCS. 1997, 1998. Washington, DC (18)	R	D	M–H–H	I–N–O	Y	M
Preliminary Investigation (PI) for Stream Riparian Areas. USDA NRCS, Watershed Science Institute. 1996. Seattle, WA (18)	R, C, A, W	C	M–M–L	I–L/N–O	N	L
Protocols for Classifying, Monitoring and Evaluating Stream Riparian Vegetation on Idaho Rangeland Streams. Division of Environmental Quality. 1992. Boise, ID (19)	R	D	H–H–H	I–N–O	N	H
Rapid Assessment of Riparian Systems (RARS). R.D. Ohmart, et al. 1998. Arizona Game and Fish Department, Phoenix, AZ (20)	R, C	D	M–H–H	A–N–O/R	Y	M
Riparian Area Management: A User Guide to Assessing Proper Functioning Condition and the Supporting Science for Lotic Areas. DOI Bureau of Land Management. 1998. Denver, CO (22)	R, C	C	M–L–L	A–L–O	Y	L
Riparian Area Management—Greenline Riparian—Wetland Monitoring. DOI Bureau of Land Management. 1993. Denver, CO (22)	R	D	M–M–M	I–N–O	N	H

Table TS3A–1 Attributes of stream corridor assessment techniques—Continued

Column notes listed below >	1	2	3	4	5	6
Technique (to obtain a technique's citation and summary, turn to the page number listed in parentheses)	Primary setting (listed first)	Sampling intensity	Skill level, training, time	Kind, measure type, proximity	Reference site needed	Suitability for monitoring
Primary setting—Riparian area—Continued						
Riparian Area Management—Inventory and Monitoring of Riparian Areas. DOI Bureau of Land Management. 1989. Denver, CO (23)	R	D	M/L–H/M/L–H/M/L	I–N–O	N	H
Riparian Area Management—Procedures for Ecological Site Inventory. DOI Bureau of Land Management. 1992. Denver, CO (23)	R, C	D	H–H–H	I–N–O	N	H
Riparian Area Management—Using Aerial Photographs to Assess Proper Functioning Condition of Riparian—Wetland Areas. DOI Bureau of Land Management. 1996. Denver, CO (24)	R, C	C	M–M–L	A–L–R	Y	L
Riparian Reserve Evaluation Techniques and Synthesis in Ecosystem Analysis at the Watershed Scale—Federal Guide for Watershed Analysis, Section II. Multiagency. 1995. Portland, OR (24)	R	D	H–M–H	A–L–O/R	N	M
Role of GIS in Selecting Sites for Riparian Restoration Based on Hydrology and Land Use. Utah State University. 1997. Logan, UT (25)	R	C	H–M–L	I/A–N–R	Y	M
RWRP Lotic Health Assessment. University of Montana. 1999. Missoula, MT (25)	R, C	C	M–M–L	A–L–O	N	M
Technology Policy Paper—Mapping Procedures for Riparian and Other Small Areas. USDA NRCS. 1997. Washington, DC (29)	R, C	D	H–M–M	I–L/N–O	N	L
Primary setting—Water quality						
Adopt-A-Stream Shoreline Survey. Massachusetts Riverways Programs. 1996. Boston, MA (13)	C	L–M–M	I/A–L–O	N	L	
Agricultural Water Quality Index. Robert B. Annis Water Resources Institute, Grand Valley State University. 1998. Allendale, MI (13)	W,C,R,A	C	M–M–M	A–L–O	N	L
Monitoring Protocols to Evaluate Water Quality Effects of Grazing Management on Western Rangeland Streams. U.S. Environmental Protection Agency. 1993. Seattle, WA (17)	W, A, C, R	D	M–H–H	A–N–O	Y	H

Table TS3A–1 Attributes of stream corridor assessment techniques—Continued

Column notes listed below >	1	2	3	4	5	6
Technique (to obtain a technique's citation and summary, turn to the page number listed in parentheses)	Primary setting (listed first)	Sampling intensity	Skill level, training, time	Kind, measure type, proximity	Reference site needed	Suitability for monitoring
Primary setting—Water quality—Continued						
Stream Temperature Investigations: Field and Analytic Methods (for use with SNTEMP: Stream Network Temperature Model). U.S. Fish and Wildlife Service. 1989. Fort Collins, CO (28)	W (temperature)	D	H–M–M	I–N–O	N	H
Water Quality Indicators Guide—Surface Water (chapter 2 and appendices A and F). Terrene Institute. 1996. Washington, DC (30)	W	C	M–M–M	A–L–O	N	L

Column notes:

1 Primary Setting (listed first); **C**hannel flood plain, **R**iparian area, **W**ater quality, **A**quatic
2 Sampling intensity: **C**ursory, **D**etailed
3 Skill level, training, time (each rated as): **H**igh, **M**edium, **L**ow
4 Kind: **I**nventory, **A**ssessment, Measure type: Qua**L**itative, Qua**N**titative; Proximity; **O**nsite, **R**emote
5 Reference site required: **Y**es, **N**o, **O**ptional
6 Suitability for monitoring: **H**igh, **M**edium, **L**ow

Table TS3A–2 Summaries of site assessment and investigation techniques

Adopt-A-Stream Shoreline Survey. Massachusetts Riverways Programs. J.C. Kimball and M. Van Dusen. 1996. Department of Fisheries, Wildlife and Environmental Law Enforcement, 100 Cambridge St., Boston, MA 02202. 62 p.

Summary: The survey's purpose is to help local stream teams determine vital signs of a river or stream, report immediate problems to proper authorities, and prioritize both short-term and long-range work. The water course is divided into reasonably sized segments that can be walked or canoed. Field data sheets include measurement of instream conditions, stream vegetation, streambank and corridor conditions, and presence of observable fish and wildlife species. Other data sheets include a summary sheet for a segment or reach survey, pipe survey, bridge survey, and wetlands survey.

Agricultural Water Quality Index. Robert B. Annis Water Resources Institute. Grand Valley State University, J. Cooper et al. 1998. WRI Publication #MR–98–1, One Campus Drive, Allendale, MI 49401. 75 p.

Summary: The Agricultural Water Quality Index (AWQI) is an assessment protocol that is specifically designed to evaluate the relationship between agricultural operations and water quality in agroecosystems. The AWQI is based on a series of assessments that can be examined separately and accumulated into a total score. Individual assessments include riparian zone metrics (width, completeness, vegetation types, summary), stream channel metrics (flow status, flow stability, channel sinuosity, channel structure, summary), and, optionally, a benthic macroinvertebrates metric (population diversity including indicator types). Specific recommendations for land and water management are associated with the ranked levels of individual metrics. Worksheets and scoring tables are provided.

Applied River Morphology. Wildland Hydrology Consultants. D. Rosgen. 1996. 1481 Stevens Lake Road, Pagosa Springs, CO 81147. 341 p.

Summary: The guide book includes fundamental principles of river behavior, a hierarchical stream inventory, a classification of natural rivers with illustrations, and data summaries and photographs depicting major stream types. The book contains field techniques and forms for:

–Stream classification of a reference reach

–Bank erosion prediction

–Fish habitat structure evaluation

–Sediment relations

–Hydraulics

–Channel stability evaluations

Channel-reach Morphology in Mountain Drainage Basins. Geological Society of America Bulletin, Volume 109, p. 596–611. D.R. Montgomery and J.M. Buffington. 1997. Department of Geological Sciences; request from the Geological Society of America, P.O. Box 9140, Boulder, CO 80301–9140. 15p.

Summary: A classification of channel-reach morphology in mountain drainage basins synthesizes stream morphologies into seven distinct reach types: colluvial, bedrock, and five alluvial channel types (cascade, step pool, plane bed, pool riffle, and dune ripple). Coupling reach-level channel processes with the spatial arrangement of reach morphologies, their links to hillslope processes, and external forcing by confinement, riparian vegetation, and woody material defines a process-based framework within which to assess channel condition and response potential in mountain drainage basins. The classification is broadly applicable with its primary advantage of addressing the role of large woody material.

Table TS3A–2 Summaries of site assessment and investigation techniques—Continued

Fish and Fish Habitat Standard Inventory Procedures Handbook (R1/R4—Northern/Intermountain Regions). USDA Forest Service, Intermountain Research Station. 1997. 324 25th Street, Ogden, UT 84401. 73p.

Summary: The handbook describes the standard inventory procedures for collecting fish habitat and salmonid fish species data for streams managed by the Northern Region (R1) and Intermountain Region (R4) of the Forest Service. The inventory defines the structure (pool/riffle and forming features), pattern (sequence and spacing) and dimensions (length, width, depth, area, and volume) of fish habitat; describes species composition, distribution, and relative abundance of salmonid species; and facilitates the calculation of summary statistics for habitat descriptors. The handbook is illustrated in color and includes data collection forms.

Guidebook for Application of Hydrogeomorphic Assessments to Riverine Wetlands. U.S. Army Corps of Engineers, Waterways Exp. Station. Technical Report WRP–DE–11. M. Brinson et al. 1995. Washington, DC 20314–1000. 207p.

Summary: The guidebook provides the basis (or template) for applying the hydrogeomorphic (HGM) approach for specific physiographic regions for wetland functional assessment of riverine wetlands in context with the Clean Water Act Section 404 Regulatory Program. The concept of a reference standard is used, for example, conditions exhibited by a group of reference wetlands in a physiographic region that correspond to the highest level of functioning. Fifteen functions are identified for the riverine wetland class and are valuated by an index computed using equations of selected variables from a group of 44 variables. Generic equations, detailed information, and field tally sheets are provided to document functions and develop models for a specific regional riverine subclass.

Incised Channels—Morphology, Dynamics and Control. S.A. Schumm, M.D. Harvey, and C.C. Watson. 1984. Water Resources Publications, P.O. Box 2841, Littleton, CO 80161. 200p.

Summary: The original basis of the document was a report on the geomorphic characteristics of channelized streams in northern Mississippi to determine if their future behavior could be predicted. The publication contains a literature review on incised channels, historical information on subject channels, and discussion of geomorphic evolution of incised channels. The concept of entrenched streams is introduced in chapter 5 of the document including the hypothetical sequence of arroyo evolution. A summary of incised channels is listed in chapter 7, including a description of a possible evolutionary sequence.

Integrated Riparian Evaluation Guide (Levels I, II, and III). USDA Forest Service. T. Collins, Regional Soil Scientist, et al. 1992. Regional Office, Intermountain Region, 324 25th Street, Ogden, UT 84401. 60+p.

Summary: The guide provides an integrated approach for: stratifying and classifying riparian areas according to their natural inherent characteristics, and their respective existing conditions; data collection; evaluation of riparian areas; future development and linkage of a riparian data base; preparation of a written narrative to interpret the data and suggest management applications; providing a process to prioritize or rank riparian areas based on management objectives; strengthening the riparian management implications of the Forest Land Management Plan. The approach is split into threes levels: level I is an office procedure, level II is a field procedure, and level III is a more quantitative, site-specific field data collection. Levels are progressive and should be completed in order. The guide includes data collection forms.

Table TS3A–2 Summaries of site assessment and investigation techniques—Continued

Methods for Evaluating Riparian Habitats with Applications to Management. USDA Forest Service. General Technical Report INT–221. Intermountain Research Station, W. Platts et al. 1987. 324 25th Street, Ogden, UT 84401. 177p.

Summary: The report compiles a comprehensive set of methods for resource specialists to use in managing, evaluating, and monitoring riparian conditions adjacent to streams, lakes, ponds, and reservoirs with an emphasis on streams. Issues of sampling kind and intensity, accuracy, and precision are described. Detailed procedures are given for measuring vegetation, classifying riparian communities and soils, using remote sensing, measuring water column attributes, detecting streambank morphology and alteration, mapping woody material, using benthic macroinvertebrates, and evaluating historic riparian habitats. Emphasis is on procedural details, rather than reliance on predefined data collection forms.

Monitoring Protocols to Evaluate Water Quality Effects of Grazing Management on Western Rangeland Streams. U.S. Environmental Protection Agency, Water Division. 1993. Region 10, 1200 Sixth Avenue, Seattle, WA 98101. 179p.

Summary: The document describes a monitoring system to assess grazing impacts on water quality in streams of the western United States. Methods described are reportedly easy to use and cost-effective (reduced sampling frequency, limited need for specialized equipment, and limited laboratory analyses). The protocols focus on attributes of the stream channel, streambank, and streamside vegetation (characteristics are sampled during low-flow summer conditions). Methodology requires an interdisciplinary team. Explanatory illustrations and various field data collection forms are included.

National Forestry Manual; National Range and Pasture Handbook Procedures for completing Vegetation Field Forms and Ecological Sites). USDA Natural Resources Conservation Service. 1998, 1997. P.O. Box 2890, Washington, DC 20013. 100+p.

Summary: The manual and handbook contain detailed procedures for completing vegetation field forms and ecological sites. The National Forestry Manual is applicable to stream riparian areas that are currently forested or have a potential for a plant community dominated by woody plants (trees) with a height potential of at least 4 meters. The National Range and Pasture Handbook is applicable to stream riparian areas that are currently in herbaceous or shrub vegetation or have a potential for a plant community dominated by herbaceous or shrub species. Detailed instructions, coding conventions, and data collection forms are provided in both the manual and handbook. Collected field data and information may be entered into a national database maintained and supported by the NRCS.

Preliminary Investigation (PI) for Stream Riparian Areas. USDA Natural Resources Conservation Service, Watershed Science Institute. 1996. c/o GEO SCI, Box 351310, UW, Seattle, WA 98195–1310. 2p.

Summary: This technique is a single page form that permits the user to record major attributes of a representative segment of a stream reach. It was developed for use with private landowners to focus attention on the existing conditions of their streams. Basic stream attributes (stream order, depth, width, gradient, entrenchment), soil conditions (bank erosion frequency, bed load fine sediments, upper bank compaction), water conditions (turbidity, presence of algae, color, temperature), plants (potential native vegetation, present vegetation, dominant terrestrial plants, aquatic species), air condition, animals (fish species, aquatic macroinvertebrates, land species), and human use attributes are collected.

Table TS3A–2 Summaries of site assessment and investigation techniques—Continued

Procedures for Using [the] Oregon Stream Habitat Data Sheet. USDA Natural Resources Conservation Service. 1998. Biology Technical Note No. 12, 101 SW Main Street, Suite 1300, Portland, OR 97204–3221. 12p.

Summary: The assessment procedure can be used on a broad reach or site-specific scale. Values that are entered on the data sheet can be estimated or measured. The intended use is for planning, baseline data, monitoring, and evaluating restoration alternatives. The procedure is not intended to replace intensive surveys conducted by professional biologists. Users of the procedure are encouraged to complete the watershed overview sheet before the habitat data sheet. The data sheet accommodates entries to identify the site, substrate composition, and bank vegetation. A series of criteria tables are used to assess and score stream habitat condition.

Protocols for Classifying, Monitoring, and Evaluating Stream/Riparian Vegetation on Idaho Rangeland Streams. Division of Environmental Quality. 1992. Report No. 8. Idaho Department of Health and Welfare, E. Cowley. 1410 North Hilton, Boise, ID 83720–9000. 37+p.

Summary: The document defines protocols and procedures for evaluating streamside vegetation and streambank stability for Idaho's small (usually less than 30 feet wide) rangeland streams. It also provides protocols for monitoring stream canopy cover, streambank stability, solar input, and establishing permanent photo points associated with livestock grazing and other activities that affect streamside vegetation and beneficial uses of water. The protocols are directed at three important pollutant sources affecting the biological integrity of streams and lakes that may result from livestock grazing: streambank erosion, water temperature, and vegetation.

Qualitative Habitat Evaluation Index [QHEI]: Rationale, Methods, and Application. State of Ohio Environmental Protection Agency. 1989. Edward T. Rankin, Ecological Assessment Section, P.O. Box 1049, 1800 WaterMark Dr., Columbus, OH 43266–0149. 51p.

Summary: The index is designed to provide a measure of habitat generally corresponding to those physical factors that affect fish communities and which are generally important to other aquatic life, such as invertebrates. The field sheet for the QHEI consists of qualitative descriptors that are checked as appropriate. Highest scores are assigned to the habitat parameters that have been shown to be correlated with streams having high biological diversity and integrity, with progressively lower scores assigned to less desirable habitat features. Individual scores are provided for the habitat components of substrate, instream cover, riparian zone and bank erosion, pool/glide quality, riffle/run quality, and gradient. A total score of 100 is possible.

Rapid Assessment of Riparian Systems (RARS)–draft report. R.D. Ohmart et al. 1998. Arizona Game and Fish Department, 2221 W. Greenway Road, Phoenix, AZ 85023. 130p.

Summary: The assessment was developed to have a tool more applicable to streams in Arizona than those currently being used throughout the West. The technique addresses riparian area classification, channel geomorphology, riparian functional analysis procedure, and riparian monitoring with photography. The objective of the developers was to collect quantitative field data to document and defend functional interpretations. The Tonto National Forest approach (Tonto Riparian Inventory and Monitoring Methods or TRIMM) was the working model for developing the assessment. The Arizona Game and Fish Department can be contacted for the final report and assessment procedure.

Table TS3A–2 Summaries of site assessment and investigation techniques—Continued

Rapid Bioassessment Protocols for Use in Wadeable Streams and Rivers—Periphyton, Benthic Macroinvertebrates, and Fish. Second Edition. U.S. Environmental Protection Agency. Office of Water (4503F), EPA841–B–99–002. 1999. Assessment and Watershed Protection Division, 401 M Street SW, Washington, DC 20460. 104p.

Summary: The document provides states with a practical technical reference for conducting cost-effective biological assessments of lotic systems. The protocols were designed as inexpensive screening tools to determine if a stream is supporting or not supporting a designated aquatic life use. They may also be appropriate for priority setting, point and nonpoint-source evaluations, use attainability analyses and trend monitoring. Worksheets are included. The protocols must be locally adapted and scaled.

Rapid Stream Assessment Protocol (RSAT) Field Methods–Appendix A. J. Galli, Sr. 1996. Dept. of Environmental Programs, Metropolitan Washington Council of Governments, 777 North Capitol St. NE, Washington, DC 20002. 35p.

Summary: The protocol is a synthesis of several techniques with applicability to nonlimestone Piedmont streams with drainage areas less than 150 square miles. RSAT employs both a reference stream and an integrated numerical scoring and verbal ranking approach. Evaluation categories include: channel stability, channel scouring/sediment deposition, physical instream habitat, water quality, riparian habitat conditions, and biological indicators (macroinvertebrates). Parameters are measured at approximately 400-foot intervals along the stream. Data is first recorded via field survey sheets and later transferred into a spreadsheet database.

Riparian Area Management: A User Guide to Assessing Proper Functioning Condition and the Supporting Science for Lotic Areas. U.S. Department of Interior, Bureau of Land Management. 1998. TR 1737–15. P.O. Box 25047, Denver, CO 80225. 126p.

Summary: The guide establishes a method for evaluating the condition of riparian-wetland lotic areas and classifying segments or reaches of streams into proper functioning condition (PFC), functional at risk, nonfunctional, and unknown categories. The qualitative, yet science-based process, considers both abiotic and biotic factors as they relate to physical function. A standard checklist of 17 key questions is provided and enables users to determine the functional condition of a stream reach or segment. PFC must be conducted by an interdisciplinary team trained and familiar with the local conditions being assessed. The supporting science and related quantitative methodologies for each of the 17 questions are provided.

Riparian Area Management—Greenline Riparian-Wetland Monitoring. U.S. Department of Interior, Bureau of Land Management. 1993. TR 1737–8. National Applied Resources Sciences Center, P.O. Box 25047, Denver, CO 80225–0047. 45p.

Summary: The technical reference gives the detailed procedure for the greenline monitoring method. Greenline is a term used to essentially identify nearest-to-stream continuous riparian plant community types using a line intercept transect running parallel to the stream. It is a procedure that is both repeatable for monitoring purposes and a point of reference which minimizes problems associated with changing moisture gradient. Data collection forms are included. (Note: As of the date of this report, the Forest Service is in the process of updating the greenline methodology with plans to republish the technique as a Forest Service technical publication.)

Table TS3A–2 Summaries of site assessment and investigation techniques—Continued

Riparian Area Management—Inventory and Monitoring of Riparian Areas. U.S. Department of Interior, Bureau of Land Management. 1989. TR 1737–3. National Applied Resources Sciences Center, P.O. Box 25047, Denver, CO 80225–0047. 79p.

Summary: The technical reference contains suggested techniques and procedures for performing an extensive inventory and, if warranted, an intensive inventory. Extensive components include drainage pattern, landform, soils information, channel form and condition, vegetation types and ecological sites, flood plain characteristics, and other attributes. Intensive components include detail soil characteristics and properties, channel parameters, vegetation identification and structure, woody species characteristics, and other attributes. A section on monitoring is integrated in the technical reference. Inventory forms are included.

Riparian Area Management—Procedures for Ecological Site Inventory. U.S. Department of Interior, Bureau of Land Management. 1992. TR 1737–7. National Applied Resources Sciences Center, P.O. Box 25047, Denver, CO 80225–0047. 135p.

Summary: The technical reference provides detailed field procedures for describing and documenting riparian-wetland ecological sites (potential vegetation) which are a function of and defined by the interaction of soils, climate, hydrology, and vegetation at riparian-wetland sites. The document contains a standard site field review checklist, site correlation checklist, standard site description, and a completed, sample standard site description. The technical reference is intended for use with the National Range and Pasture Handbook, the National Forestry Manual, and the National Soil Survey Handbook available from the USDA, Natural Resources Conservation Service, P.O. Box 2890, Washington, DC 20013.

Riparian Area Management—Using Aerial Photographs to Assess Proper Functioning Condition of Riparian-Wetland Areas. U.S. Department of Interior, Bureau of Land Management. 1996 (Revised 1999). TR 1737–12. P.O. Box 25047, Denver, CO 80225. 52p.

Summary: The document provides a procedure for using aerial photography to answer proper functioning condition checklist items. It supplements TR1737–15, Riparian area management: A user guide to assessing proper functioning condition and the supporting science for lotic areas. The technical release gives the detailed procedure for gathering existing source material, analyzing equipment needs, defining reaches and areas, interpreting aerial photos, and verifying interpretations in the field. Also included are specific recommendations pertaining to needed aerial photo qualities, photo interpretation examples, and the results of large area case studies in Montana.

Riparian Reserve Evaluation Techniques and Synthesis in Ecosystem Analysis at the Watershed Scale—Federal Guide for Watershed Analysis, Section II. Multiagency. 1995. Version 2.2. Regional Ecosystem Office, P.O. Box 3623, Portland, OR 97208. 42p.

Summary: This supplement is part of the Federal guide developed to help resource managers implement direction in the record of decision (ROD) for amendments to Forest Service and Bureau of Land Management planning documents within the range of the Northern Spotted Owl. The ROD requires watershed analysis prior to the final delineation and management of the riparian reserve network in a watershed. The riparian analysis process is divided into two levels based on anticipated activities: level 1—geared toward small effects along intermittent streams, and level 2—addresses larger magnitude effects.

Table TS3A–2 Summaries of site assessment and investigation techniques—Continued

Role of GIS in Selecting Sites for Riparian Restoration Based on Hydrology and Land Use. Utah State University. 1997. G.D. Russell, C.P. Hawkins, M.P. O'Neill. Watershed Science Unit, Logan, UT 84322–5250. 13p.

Summary: The paper describes an approach to initial site selection in the San Luis Rey River watershed in southern California that uses watershed-level information on basin topography and land cover to rank the potential suitability of all sites within a watershed for either preservation or restoration. The approach requires the use of a geographic information system (GIS) to map relative wetness and land cover within a watershed. Relative potential wetness values were derived from USGS 30-meter digital elevation models; land cover was derived from a Landsat scene covering the 1,500 square kilometers study area. The paper is illustrated with color diagrams and pictures.

RWRP Lotic Health Assessment. University of Montana. 1999. Riparian and Wetland Research Program, School of Forestry. Missoula, MT 59812. 25p.

Summary: The assessment is a method for rapidly addressing a lotic site's overall health or condition. It provides a site rating useful for setting management priorities and stratifying riparian sites for remedial action or more rigorous analytical attention. It is intended to serve as a first approximation, or coarse filter, by which to identify lotic wetlands in need of closer attention so that managers can more efficiently concentrate effort. The term riparian health is used to mean the ability of a riparian reach (including the riparian area and its channel) to perform certain functions. These functions include sediment trapping, bank building and maintenance, water storage, aquifer recharge, flow energy dissipation, maintenance of biotic diversity, and primary production.**Stream Channel Reference Sites: An Illustrated Guide to Field Technique**. USDA Forest Service. General Technical Report RM–245. C. Harrelson et al. 1994. Rocky Mountain Forest and Range Experiment Station, Fort Collins, CO. 61p.

Summary: The guide helps users establish permanent reference sites. The minimum procedure consists of: select a site, map the site and location, measure the channel cross section, survey a longitudinal profile of the channel, measure stream flow, measure bed material, and permanently file the information with the Vigil Network. The document includes basic surveying techniques and provides guidelines for identifying bankfull indicators and measuring other important stream characteristics. The object is to establish the baseline of existing physical conditions for the stream channel. The guide is amply illustrated with diagrams and black and white pictures.

Stream Corridor Assessment Survey. Maryland Department of Natural Resources. 2000 (revised draft). K. Yetman, Watershed Restoration Division, Chesapeake and Coastal Watershed Services, Annapolis, MD 21401. 100+p.

Summary: The survey protocols help users identify environmental problems and prioritize restoration opportunities that exist within Maryland watersheds. The assessment is designed to be done by small teams of well-trained volunteers who walk 2 or more stream miles per day. Potential environmental problems identified during a survey include channelized stream sections, streambank erosion, exposed pipes, inadequate stream buffers, fish blockages, trash dumping sites, near stream construction, pipe outfalls, and general conditions of instream and riparian habitat. In conjunction with the AmeriCorp program, more than 700 miles of Maryland streams have been surveyed using the assessment protocols. This has led to more than $1 million of restoration work to date. One Maryland county has included the assessment as part of the NPDES permit system for municipal stormwater discharges.

Table TS3A–2 Summaries of site assessment and investigation techniques—Continued

Stream Inventory Handbook—Level I and II. USDA Forest Service. 1996. Version 9.6. Region 6, P.O. Box 3623, Portland, OR 97208. 76p.

Summary: The handbook provides standards for a level I (office inventory) and level II (field inventory) of stream systems. The protocol identifies core attributes necessary to evaluate the condition of a stream. It contains instructions and data forms for stream habitat conditions (flow, water quality, historical land use, valley-channel parameters, streambed substrate, flood-prone dimensions, and riparian habitat dimensions). Other data forms are included for inventorying culverts, falls, chutes, dams, marshes, braids, and fish species.

Streamkeeper's Field Guide—Watershed Inventory and Stream Monitoring Methods. The Adopt-A-Stream Foundation. T. Murdoch, M. Cheo and K. O'Laughlin. 1996. 600 128th Street SE, Everett, WA 98208. 296p.

Summary: The guide provides methods for obtaining a holistic picture of a stream's watershed, as well as collecting detailed information. The techniques presented in the guide are fairly simple, inexpensive, and can be accomplished with readily available equipment. Readers not only learn how to evaluate the physical and biological characteristics of streams using the latest quality control and quality assurance planning techniques but can also study a chapter devoted to presenting field data to a wide range of audiences. The section called Streamkeeper Tales includes inspirational examples of volunteers who have used their field data as the basis for protecting and restoring streams. The active voice of the text and the large number of humorous technical illustrations accompanied by poignant editorial cartoons make this book engaging to volunteers and scientists alike.

Stream Temperature Investigations: Field and Analytic Methods (for use with SNTEMP: Stream Network Temperature Model). U.S. Fish and Wildlife Service. 1989. Instream Flow Information Paper No. 13. Biological Report 89 (17). J. Bartholow, National Ecology Research Center, 2627 Redwing Road, Fort Collins, CO 80526–2899. 139p.

Summary: The document provides guidance to the user of the Stream Network Temperature Model (SNTEMP). Planning, executing, and using the results from a stream temperature modeling study are described. Details of field data gathering, instrumentation, and data collection priorities are given for the range of stream geometry, meteorology, and hydrology components necessary for the model's application. Each input variable is defined, and its relative data collection effort is approached from the perspective of sensitivity in predicting stream temperatures. Alternative public domain stream and reservoir temperature models and techniques are also described.

Stream Visual Assessment Protocol. USDA Natural Resources Conservation Service. B. Newton et al. 1998. 101 SW Main St., Suite 1600, Portland, OR 7204–3225. 36p.

Summary: The assessment protocol provides a basic level of stream health valuation based primarily on physical conditions for a stream reach. It is intended to be conducted with the landowner and incorporates talking points for planners to use during an assessment. Assessment elements, which receive a numerical rating based on observations and some rapid measurements, include: channel condition, hydrologic alteration, riparian zone, bank stability, water appearance, nutrient enrichment, barriers to fish movement, instream fish cover, pools, invertebrate habitat, canopy cover, manure presence, salinity, riffle embeddedness, and macroinvertebrates observed. Rating criteria and worksheets are included. The protocol works best if locally modified.

Table TS3A–2 Summaries of site assessment and investigation techniques—Continued

Technology Policy Paper—Mapping Procedures for Riparian and Other Small Areas. USDA Natural Resources Conservation Service. 1997. Soil Survey Division, P.O. Box 2890, Washington, DC 20013. 12p.

Summary: The paper outlines the procedure for mapping riparian and other small areas which were traditionally identified by spot symbols on soil survey maps. Riparian areas are typically very linear and are more difficult to map and display than upland soil polygons. Certain soils, hydrology, and vegetation criteria must be met for an area to be identified and mapped as a riparian area. Cartographic procedures for delineating point and line features are included. Examples of soil map unit descriptions and a sample soils map are provided.

Underwater Methods for Study of Salmonids in the Intermountain West. USDA Forest Service, Intermountain Research Station. 1994. Russell F. Thurow, General Technical Report INT–GTR–307, 324 25th Street, Ogden, UT 84401. 28p.

Summary: Underwater observation with snorkeling gear is a valuable tool for studying fish populations and assessing how fish use habitat in flowing waters. Precise estimates of fish abundance can be obtained using underwater counts. However, several factors, including the behavior of the target fish species and attributes of the physical habitat (stream size, water clarity, temperature, cover), can bias results. This report was developed to assist biologists in identifying and accounting for potential biases and to encourage a standardized procedure for the use of underwater techniques to survey salmonids in streams. The guide addresses the principal resident and anadromous salmonids found in the Intermountain West (Idaho, Montana, Nevada, Utah, and western Wyoming). Color illustrations and pen and ink drawings of target fish are included.

Water Quality Indicators Guide – Surface Water (chapter 2 and appendices A and F). Terrene Institute. 1996. Second Ed. 1717 K St., Suite 801, Washington, DC 20006–1504. 131p.

Summary: The guide examines five major sources of agriculturally related nonpoint source pollution: sediment, nutrients, animal waste, pesticides, and salts. Field sheets are provided to enable the user to observe and record surface water quality problems and to select appropriate remedial practices. Field sheets are arranged in matrix format with environmental indicators given for each of the five major pollutant types. Each indicator is divided into descriptions of the environment from excellent to poor with each description given a weighted numerical ranking. There are two types of field sheets: one for receiving waters and one for the lands that drain into receiving waters.

Stream*A*Syst-A Tool to Help You Examine Stream Conditions on Your Property. Oregon State University, Extension Service, 422 Kerr Administration, Corvallis, OR 97331–2119. 12p.

Summary: The publication consists of a worksheet and action plan developed for use by landowners having a stream or stream systems on their property. The worksheet's 15 questions direct the user to all aspects of stream corridor condition. The action plan correlates individual answers from the worksheet to helpful notes and contact agencies and addresses for further investigation. The assessment system is voluntary, useful for a first approximation of stream corridor conditions, and alerts the landowner of possible concerns.

Technical Supplement 3B

Using Aerial Videography and GIS for Stream Channel Stabilization in the Deep Loess Region of Western Iowa

(210–VI–NEH, August 2007)

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Cover photo: Channel stabilization in western Iowa begins with an understanding of streambed gradients and their natural controls and causes of instability. Channel degradation or incision is widespread in this area. Soils in the deep loess region of the Mississippi River Basin are some of the most highly erosive in the country.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 3B

Using Aerial Videography and GIS for Stream Channel Stabilization in the Deep Loess Region of Western Iowa

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Technical Supplement 3B

Using Aerial Videography and GIS for Stream Channel Stabilization in the Deep Loess Region of Western Iowa

Abstract

This technical supplement presents an example of using an aerial assessment and classification of streams in the deep loess region of western Iowa. This is a large-scale effort directed by the Hungry Canyons Alliance involving approximately 5,250 miles of stream in 22 western Iowa counties. This effort has determined areas of active stream erosion and the impact streambed stabilization structures have had on controlling stream degradation. The stream assessment consisted of flying along streams in a small helicopter videotaping the stream channel and recording positions with a global positioning system receiver. Streams were classified based on a six-stage channel evolution model to describe the dominant channel processes occurring along stream reaches. Streams across the region in 1993 and 1994, and in a smaller area in 2000, were similarly classified.

The comparison of the recent classification to those of the past has allowed the researchers in this study to describe how stream stabilization structures have impacted the streams and make predictions as to where future stream erosion will occur. The data has also been used to mathematically model channel evolution in the region.

The purpose of Hungry Canyons Alliance (HCA) is to focus attention on the problems and develop solutions related to stream channel degradation in 22 counties of western Iowa with deep loess soils. The HCA provides cost share to counties to build streambed stabilization structures, building about 20 structures per year. To date, HCA structures protect 195 bridges; numerous utility lines including electric, phone, gas, sewer, water lines; and an estimated 830 acres (336 ha) of farmland, equivalent to stopping 11.3 million tons (10.25 million metric tons) of sediment from being swept away into the Missouri River and Mississippi River systems.

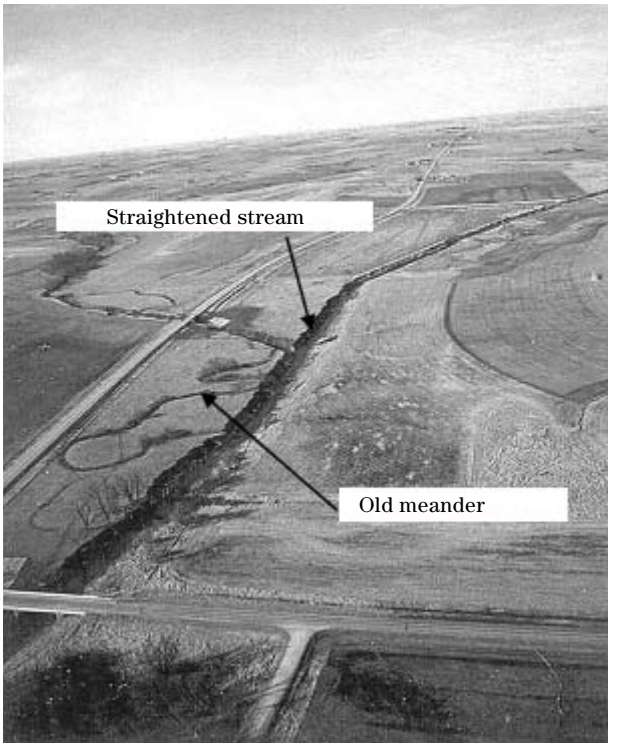
However, HCA members need to know where the areas of active stream erosion are to wisely locate structure sites. To do this, an aerial assessment of streams in western Iowa was performed, and streams were classified based on the channel evolution model.

Introduction

In western Iowa, 22 counties contain deposits of loess ranging from 13 feet (3.96 m) to more than 200 feet (60.96 m) deep. Loess is highly erodible and susceptible to stream channel erosion and degradation. It is estimated that stream channel downcutting and widening in western Iowa has caused $1.1 billion in damage to bridge and utility infrastructure (Hadish et al. 1994). Stream degradation has also eroded thousands of acres of valuable farmland, increased stream sediment loads, and decreased water quality.

Stream channel dredging, straightening, and land use changes since the early 1900s have caused many problems. After straightening, the streambed is steeper because the stream still had the same amount of fall per mile, yet there was now a shorter distance over which that fall occurred. The steeper slope increased water velocities. To reduce the water velocity, streams either had to meander or downcut. Due to the high erodibility of loess soils, the streams began downcutting, causing accelerated soil erosion in western Iowa (fig. TS3B–1).

Figure TS3B–1 Straightened versus meandering stream (Walnut Creek, Pottawattamie County, IA)



As the streambed downcuts, it destabilizes bridge pilings (fig. TS3B–2), and the vertical streambanks will slump to a more stable slope, in effect widening the stream and necessitating longer bridges.

Nickpoints, which are naturally occurring overfalls (fig. TS3B–3), will continue to erode and advance upstream, eventually affecting the entire watershed. Bed degradation of stream channels will force the channel's tributaries to also adjust to the lowered base level, often initiating gully formation where no channel had previously existed.

Streambed stabilization is the key to preventing further erosion and protecting infrastructure (fig. TS3B–4). After the stream downcut 6 feet in 7 years, the bank stabilization was left high and dry. Dams or weirs at regular intervals will help streams stabilize by changing their profile from a steep incline to a stable stair-step pattern. Structures normally have a raised weir section, like a low-head dam. Most structures use steel sheet pile driven into the streambed 20 to 25 feet (6.1–7.6 m) and riprap to protect the banks (fig. TS3B–5).

Figure TS3B–2 Bridge endangered by exhumation of pilings

Figure TS3B–3 Nickpoint eroding headward

Figure TS3B–4 An NRCS bank stabilization site that did not include streambed stabilization, (Page County, IA)

Figure TS3B–5 HCA steel sheet pile weir with riprap and cement grout that protects a large sewer main for the City of Denison, IA

Streambed stabilization structures have many benefits. They decrease the slope of the streambed, reducing water velocities. They prevent further downcutting, protecting farmland, bridge pilings, and utility lines from future erosion. They create an upstream backwater condition which allows sediment to settle out, reducing sediment loads and improving water quality. The backed-up water often helps to protect bridge pilings by submerging them.

Previous research

A six-stage channel evolution model (fig. TS3B–6, developed by Simon and Hupp (1986), was used to describe the dominant channel processes occurring along stream reaches.

An aerial reconnaissance of streams in 18 western Iowa counties was conducted in 1993 and 1994 by the HCA. They were flown during the spring to prevent tree cover and other vegetation and snow and ice from obscuring the view of the channels (Hadish 1997). The stream channels were recorded through the door of a low-flying helicopter by a hand-held video camera while a microphone recorded a narration onto video tapes. This narration described location information and other observations. The video focused on the streambed, streambanks, and the flood plain, along with other features like gullies, nickpoints, and grade control structures.

In conjunction with a set of manually transcribed notes, the videotape and narration was used to classify streams by Simon's six-stage channel evolution model. Classified stream reach locations were transferred to USGS 1:100,000 scale topographic base maps by hand. These hard copy maps were then converted to a digital data set using the Geographic Resources Analysis Support System (GRASS) geographic information system (GIS) software package. GRASS GIS is an open source free software GIS that is largely command line driven. In 2000, the NRCS reclassified streams in four counties and classified streams in two additional counties using the same process.

In the 1993 to 1994 study, 107 streams covering 1,540 miles (2,478 km) were videotaped. More than 90 percent of the study area at that time was unstable, in stages 2, 3, 4, and 5, with the greatest amount (55.9 percent) in stage 4 (Hadish et al. 1994). Stage 5 was concentrated in the downstream portions of larger drainages, whereas stage 3 reaches occurred in the upper reaches of main streams and on small tributaries. The data supported the notion of a recovery process, described by the channel evolution model, where degradation progresses from the lower to the upper stream reaches followed by aggradation (Hadish 1997).

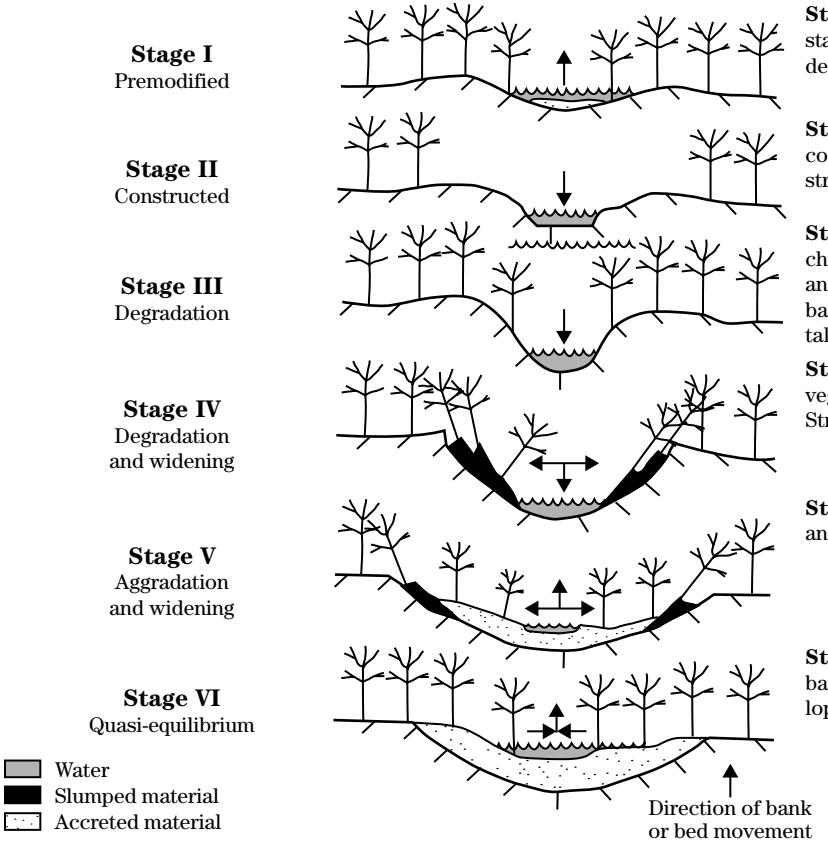
Methods

Many issues led the HCA to undertake another stream classification. First, the 1993 to 1994 data was largely outdated and not very helpful in streambed stabilization project planning. Second, it is still important to determine where streams are actively eroding and make predictions as to where future erosion will occur. Third, the HCA wanted to determine the impact of streambed stabilization structures on stream degradation. Fourth, with two sets of data, channel evolution in the region can be modeled by comparing the progression of erosion between flights. Finally, current stream videos and maps can be used by county engineers and NRCS district offices for other purposes such as county infrastructure inspections, land use planning and zoning, and conservation practice evaluations.

To update the stream classification, the decision was made to videotape those streams that were previously flown, had grade control structures on them, had drainage areas of more than 10 square miles, or were known to be experiencing streambed degradation. The stream classification began in the spring of 2002 and continued during the fall of 2002 and spring of 2003, when flying ended due to foliage or inclement weather.

The same basic process was again used to classify streams, but there were significant changes due to technological advances. Videotaping was done through the open door of a small helicopter with a hand-held digital video camera while a global positioning system (GPS) receiver recorded the position. The Video Mapping System (VMS), engineered by Red Hen Systems of Fort Collins, Colorado, integrates video and still images with simultaneous location information recorded by the GPS. The GPS signal is stored on one of the audio tracks of the videocassette. For added error

Figure TS3B–6 Six-stage channel evolution model



Stage 1: Includes streams that have not been modified and tend to be very stable. These streams tend to meander across their flood plain and have a dense vegetative cover on the banks down to the low-flow line of the channel.

Stage 2: Associated with streams recently modified or channeled by construction. Degradation at this stage depends upon such characteristics as streambed slope, angle of the banks, and cross-sectional area.

Stage 3: Degradation occurs most rapidly in the streambed. The increased channel slope increases the flow velocity, which causes the channel to deepen and its bank slopes to become steeper. As the channel downcuts, the toe of bank slopes are undercut, and bank failures will occur. Full-scale degradation takes place during this stage

Stage 4: Evident by channel widening. Mass erosion of bank soils and vegetation is predominant, creating a scalloped appearance in the banks. Streambed degradation slows during this stage

Stage 5: The bed of the channel begins to stabilize; bank failures decrease and re-vegetation occurs. The height and slope of the banks is reduced.

Stage 6: Channels show re-vegetation as it becomes increasingly stable and bank widening stops. Vegetation may extend in a dense cover up the sides-lopes

reduction, in case the GPS unit stopped working, notations on county maps and a journal were kept to note locations, dates visited, and keyed to the corresponding videocassettes on which that information had been recorded. The video was brought to the office where it was indexed by VMS, which associates a segment of videotape with the recorded GPS locations and translates this information between the GPS receiver, camcorder, and computer.

Once a tape was indexed, it was ready for classification, where attributes such as the six-stage channel evolution model were added to the information collected by GPS by examining the videotaped segment and applying a value to each stream reach. When that process is completed, the data are then exported out of the VMS software package as an ESRI formatted shapefile. The shapefiles were then brought into ESRIs ArcGIS, a menu and button driven software package, where further data manipulation such as overlaying the data with previously collected data sets and other spatially referenced information was performed.

One of the finished products that will be important to the overall impact of the project is the creation of DVDs for distribution to project partners. After the videotape was analyzed, a set of DVDs showing streams in their county was made available to county engineers and NRCS district offices to use in stream channel stabilization project planning.

Results

The stream reaches that were flown in the spring and fall of 2002 and the spring of 2003 are shown in figure TS3B–7. Also shown in figure TS3B–7 is the 22-county HCA region, loess thickness, and locations of known grade control structures (207 HCA, 300 EWP, 6 Iowa DOT, and 39 landowner-installed grade control structures).

To determine the progression of erosion through time and the impact streambed stabilization structures have had on controlling stream degradation, two segments of stream that were classified in 1994 and again in 2002 are shown for comparison in figure TS3B–8. In 1994, Graybill and Jordan Creeks in Pottawattamie County were predominately stage 4, with stage 3 farther upstream in the watersheds. Only two structures had been built on these streams at that time. The arrow points to a bridge where 23 feet (7 m) of degradation was recorded between 1972 and 1993. By 2002, 14 additional structures had been built on these two creeks. The streams have become less erosive and more stable, with the average stream at stage 5.

Degradation has migrated about 5 miles (8 km) upstream with stages 3 and 4 occurring only near the headwaters of the streams.

Conclusion

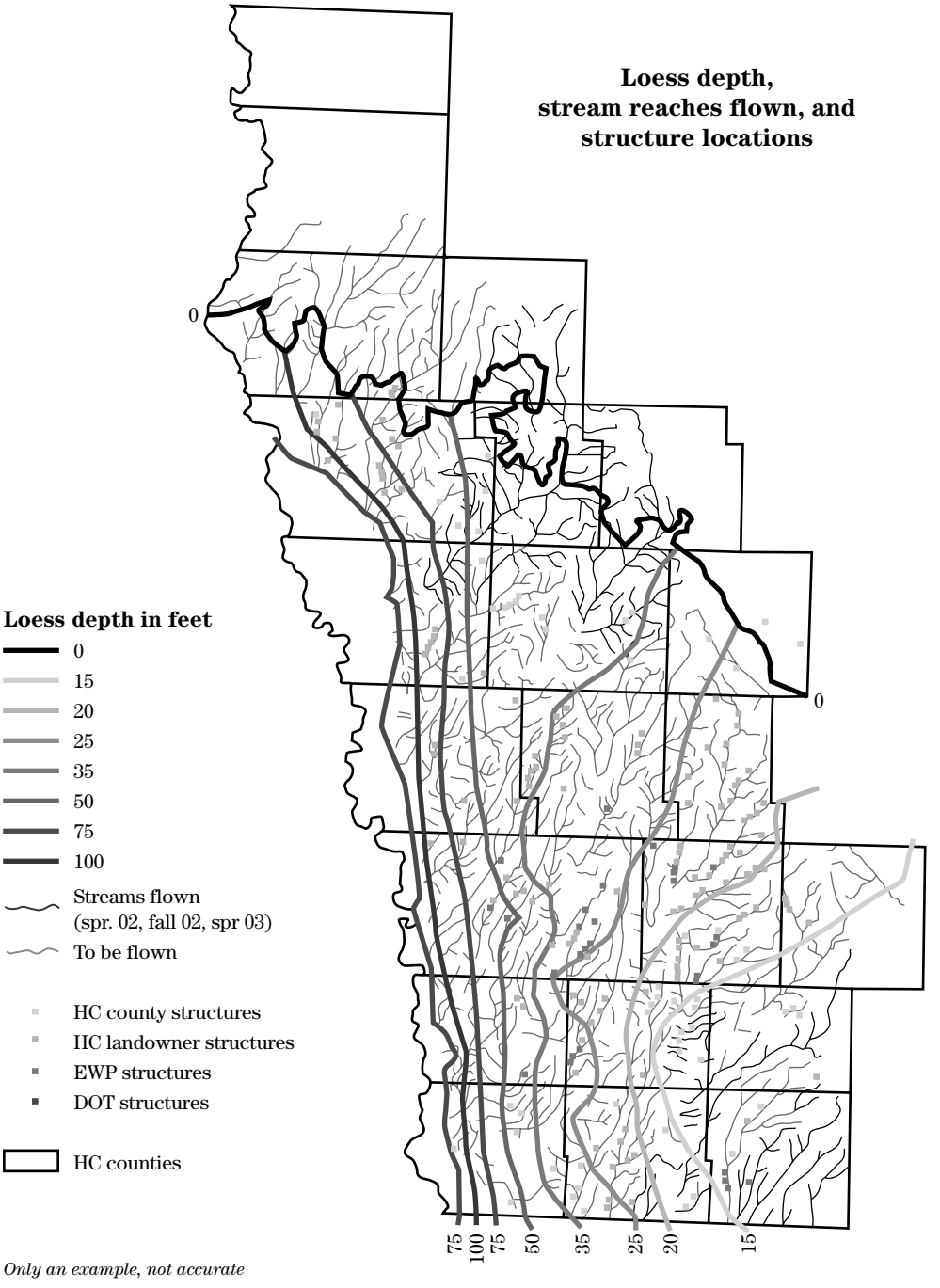
The use of aerial videography and GIS has proven to be a rapid and effective technique for rapidly assessing streams.

In the subject study, degradation does not appear to extend into areas where there is no loess soil cover, that is, beyond the loess-till boundary (line of 0 loess depth) in the northern half of western Iowa. Degradation, particularly gully erosion, becomes more pronounced in areas where loess soils are thickest.

Most large streams (those streams with a watershed greater than 70 square miles (181 km^2)) are no longer degrading, but aggrading. Many show evidence of deep incision in the past; however, they have filled with sediment eroded from upstream as degradation has moved upstream. This recovery process was also noted by Hadish (1997). The likelihood of active degradation becomes greater with distance north-northeast away from the Missouri River because many streams near the river experienced degradation first as degradation has migrated upstream. They have since become stabilized by the influx of sediment from upstream. Degradation only occurs where streams have been channelized downstream. The closer a stream reach is to the original channelization, the greater the degradation that has occurred on that stream reach.

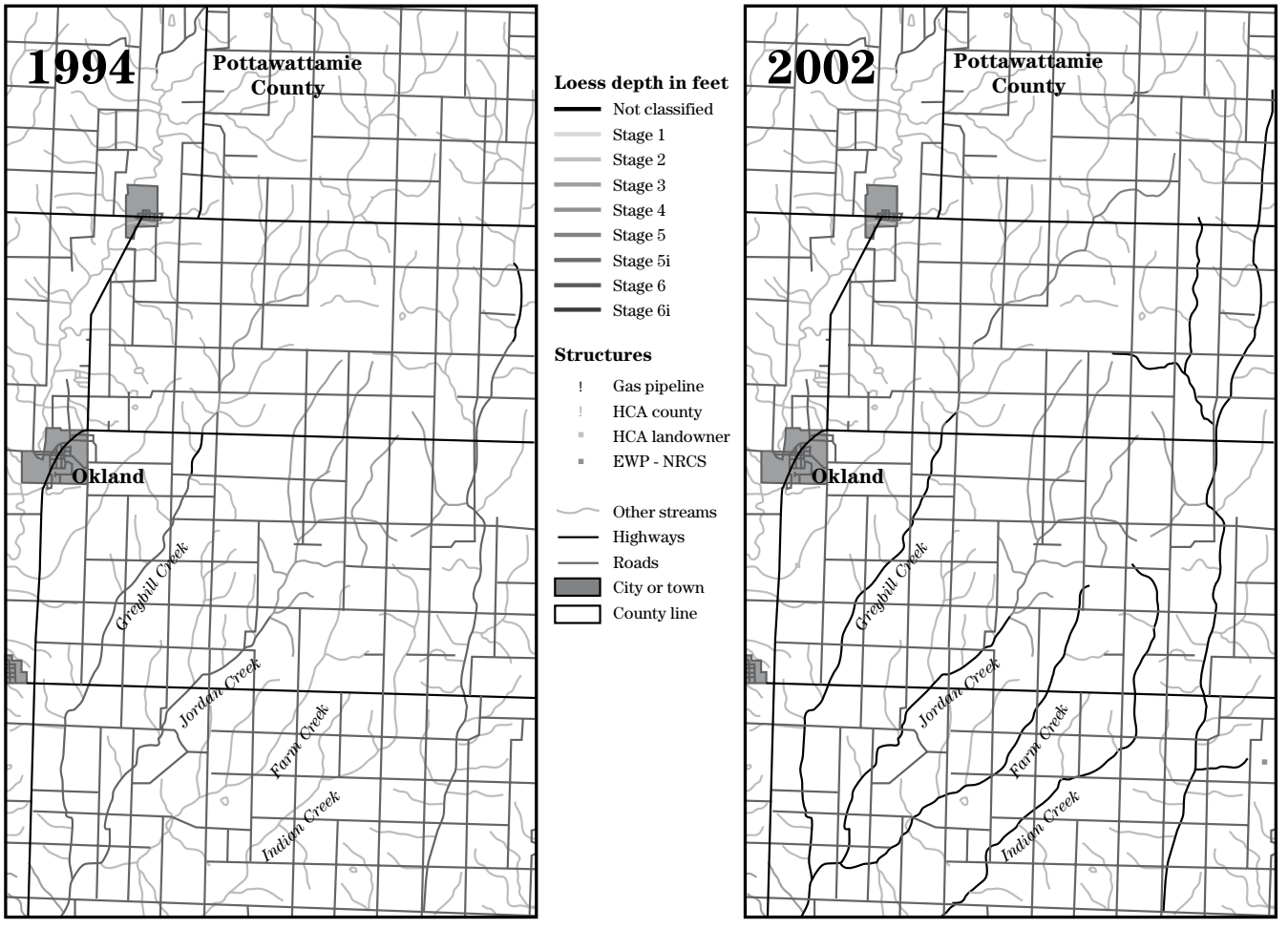
Grade control structures appear to have helped stabilize large stretches of stream. Streambeds up to 1 mile (1.6 km) upstream from the structures have been stabilized (fig. TS3B–8). For example, the rapid stabilization of Graybill and Jordan Creeks is largely due to the construction of grade control structures at regular intervals.

Figure TS3B–7 Stream reaches flown and classified in 2002 and 2003, grade control structure locations, loess depth, and 22 western Iowa counties in the HCA



Only an example, not accurate

Figure TS3B–8 Comparison of two classified stream reaches on Graybill and Jordan Creeks in 1994 and 2002



Technical Supplement 3C

Streambank Inventory and Evaluation

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Techniques for collecting data for design range from simple field measurements to complex modeling.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 3C

Streambank Inventory and Evaluation

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Technical Supplement 3C

Streambank Inventory and Evaluation

Abstract

Inventory and evaluation of stream stability requires an understanding of the cause of the perceived problems. Sometimes, causes of instability are visible onsite, but many times it is necessary to consider activities in other reaches of the stream or in the overall watershed. Also, the problem may not be anthropogenic at all, but rather a naturally occurring process that is incompatible with the existing riparian land use. This technical supplement introduces the concepts of stream stability and equilibrium along with a channel evolution model (CEM) as background material. It then presents a detailed procedure for data collection and analysis to facilitate the understanding of the dynamics of a subject stream. Published data and field-collected measurements are analyzed and compared; when all valid data match closely, the level of confidence in the analysis is high, and an assessment of the situation can proceed. The suggested procedure relies heavily on a spreadsheet tool developed by Illinois NRCS to collect and compare all available relevant data, but the same analysis can be successfully accomplished without this specific tool.

Problem identification and trend analysis

Causes of channel and bank instability can be broadly grouped into four areas of common causes: downstream, upstream, watershedwide factors, and local factors. Downstream factors involve lowering of the downstream base level, which can significantly impact upstream reaches. Upstream factors alter the incoming discharge of water and/or sediment by installation of features such as dams and diversion channels. Watershedwide factors are the result of major land use changes such as urbanization. Local factors result from geotechnical failures, sparse riparian vegetation, and unstable planform. These local causes may be exacerbated by upstream, downstream, or watershedwide factors or they may be the primary cause.

One common misconception often found is the assumption that a stable stream should not erode its banks. The fact is that stable streams are not static; they typically migrate more slowly than one that has been destabilized by anthropogenic forces. The difference between stable and unstable is not always a clear distinction as streams in dynamic equilibrium will continually migrate slowly across their flood plains. The distinction is in the rate of lateral migration being slow enough in stable streams that the riparian zone remains essentially intact through the entire process. Stable streams should, however, remain essentially static in relation to their overall profile; that is, they will not exhibit any large scale degradation or aggradation.

Watershedwide problems

Hundreds of years of human activity on the landscape have made significant changes in the major elements controlling stream balance. People have:

- cleared the timber
- plowed the prairie
- drained the wetlands
- straightened the streams
- levied the flood plains
- built cities with large areas of concrete, asphalt, and rooftops

Results of such activity on stream dynamics have generally had the effect of increasing runoff and stream slope and reducing flood plain width. In many watersheds, the land use changes are a significant factor in increased runoff. In rural areas, this may be due to more intense agricultural activities replacing woodland and grass land with cultivated land. In urban areas, the increase of impermeable surfaces within the watershed results in an increased volume of water. Additionally, the urban development of a watershed typically results in permanent land cover, either in impermeable surfaces or lawns, which produces little sediment to be delivered to the system.

Lane's Balance (fig. TS3C–1) is a tool for understanding the relationship between factors affecting channel configuration (Federal Interagency Stream Restoration Working Group (FISRWG) 1998). Stability is represented when the scale is balanced and the system has achieved an equilibrium condition. Both the increased runoff from impervious areas and the reduced sediment loads will tend to tip Lane's Balance to channel

degradation in the stream system, as illustrated with the arrow in figure TS3C–1. Increased runoff represents higher energy in the streamflow, and reduced sediment load means there is less work for that energy to do. The excess streamflow energy is dissipated by eroding the streambanks or scouring out the bed of the channel (degradation), providing more sediment and bringing the system to a new equilibrium.

Another aid in identifying the processes at work in a stream is the CEM (fig. TS3C–2 (Simon 1989)). This model describes a predictable series of changes that a channel may transition through following some disturbance. The CEM is addressed in more detail in NEH654.03.

Channel problems

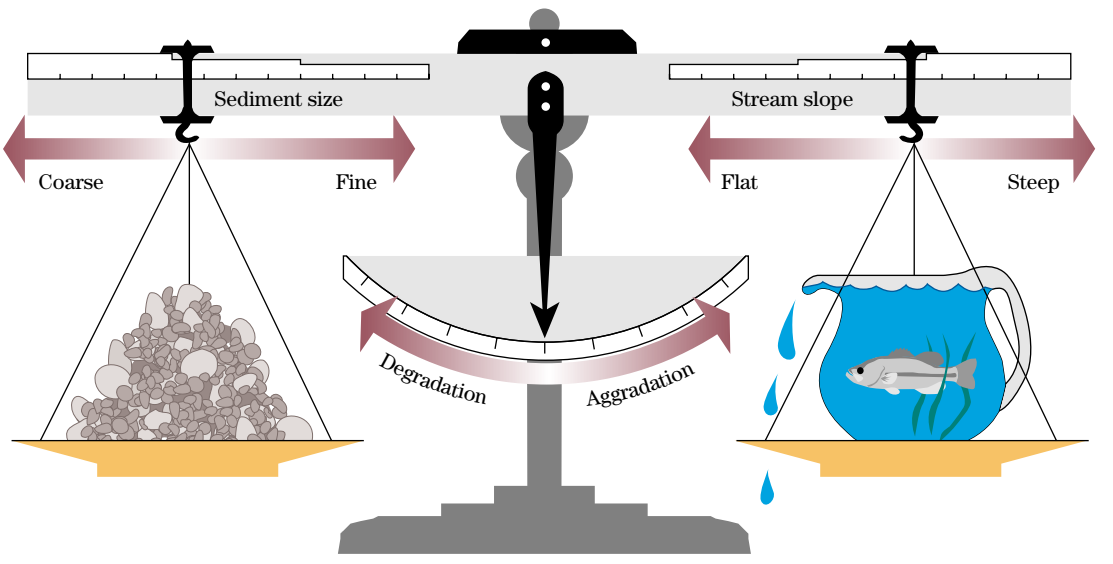
Channel modifications nearly always contribute to channel instability at some point. Some of the more obvious modifications are channelization, dam construction, and levees. Some less obvious, but still significant changes, include clearing and snagging, gravel mining, and channel lining or paving. The changes induced by these channel modifications can be dramatic, but more typically, they appear rather insignificant to the casual observer, especially in the short term. Time then becomes a significant element to consider in the problem identification phase, as the lag time between channel or watershed changes and the full effects of those changes can be decades. Because the impacts of channel modifications are cumulative over time, it is often difficult to identify a single modification that is responsible for an adverse condition.

The designer's most important task is to be aware of the overall condition of the stream and identify trends toward or away from the equilibrium or balanced condition. Only then can alternatives be considered.

Procedures for streambank investigations and analysis

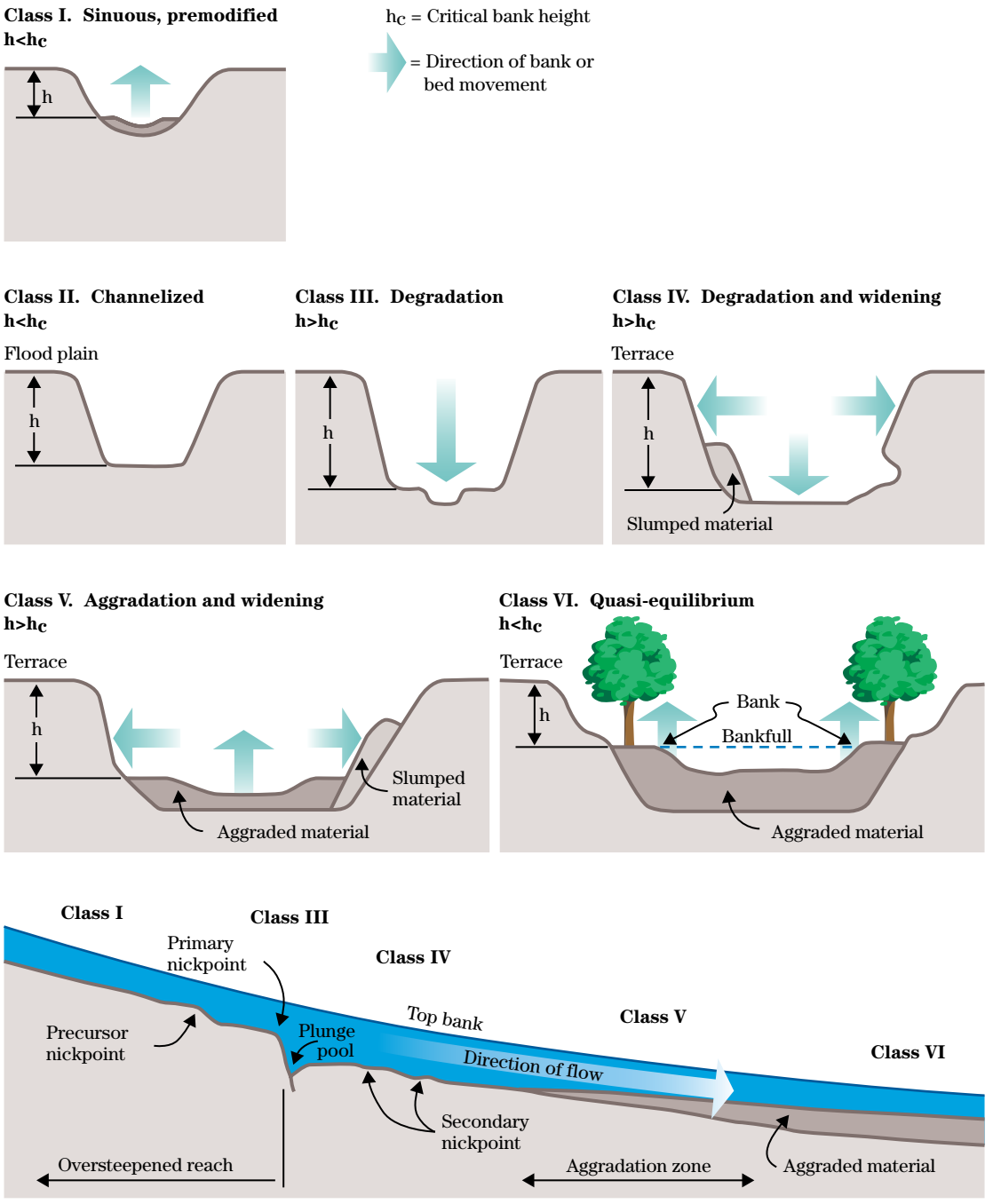
The underlying assumption to the designer's investigation and analysis is that every stream has a stable dimension, slope, and planform to safely carry the water and sediment generated from its watershed under the current climate and land use. That is not to

Figure TS3C–1 Lane's Balance for determining the effect of human activity on streams



$$Q_s \bullet D_{50} \propto Q_w \bullet S$$

Figure TS3C–2 Channel evolution model (CEM) (Simon 1989)



say that the stream is in a static condition, but rather that a stable stream maintains the same dimensions, slope, and planform while moving slowly within its flood plain position. The investigative procedure is a process of determining what the stable conditions of each unique stream segment should be and what the current conditions are, comparing the two conditions, and then attempting to understand the reasons for any differences. Only then can the designer analyze the condition of the stream and recommend action to improve an unsatisfactory condition and move the stream toward a stable state or, at a very minimum, prevent action that would further destabilize the stream.

The Illinois NRCS spreadsheet program, designed to assist in gathering and analyzing the data required for inventory and evaluation (I&E) of an Illinois stream segment, will be presented as a part of the suggested investigative procedure. Some of the data and analysis are very specific to Illinois, particularly gage data and regression curves. If the spreadsheet is used outside of Illinois, the reference stream gage section and the U.S. Geological Survey (USGS) flood-peak discharge prediction section will not apply. The collection form and its accompanying subroutines appear later in this supplement. The spreadsheet program can be found in its most current form on the Illinois NRCS Web site: *http://www.il.nrcs.usda.gov/technical/engineer/engsprdshts.html.*

Geomorphic values

There is a natural variability to hydraulic geometry relationships. It is important to recognize that this variability represents a valid range of stable channel dimensions due to such variables as geology, vegetation, land use, sediment load, sediment grain size, and runoff characteristics. The values suggested in the following procedure for bankfull discharge, width-to-depth ratio, sinuosity, radius of curvature-to-bankfull width ratio, and entrenchment ratio are based on measured observations from streams in Illinois, as well as published ranges from various research done elsewhere. Values for these relationships should not be assumed to be more accurate or precise than intended. These relationships can be used as a preliminary guide to stability in stream reaches, but other techniques and local data should be considered.

Background data collection (prior to field visit)

The first step in the investigation phase is to gather existing data for the project area. The information gathered will make the initial field visit much more productive and allow for some preliminary analysis to be done with less field time.

Step 1 On the I&E spreadsheet, enter the location and identification information including county, legal description, stream name, name(s) of decisionmakers or landowners, and UTM coordinates (if desired). These appear at the top of the spreadsheet I&E form.

Step 2 Aerial photography is the first data set to acquire. Using the most recent aerial photography available, compare with older aerial photos to determine:

- Channel alignment changes (straightening and shortening of the channel length)—Calculate channel sinuosity (old and new).
- Lateral migration rates—By measuring from discernible features such as known points, roads, and section lines, and determining the total migration rate for several years, a reasonable estimate can be made of average annual migration.
- Changes in the channel width over time—Has the channel top width gotten larger? Widening could be a sign of past downcutting, or excessive bed load causing aggradation.
- Changes in the bed features such as central bars and size of point bars—Increased bar size could be a sign of excessive bed load.
- Scour patterns in the flood plain
- Locations of any existing levees

Step 3 From USGS topographic maps (or other suitable maps), determine the watershed boundaries of the stream reach. Calculate drainage area (if available, nearby gage data can be used to help determine the drainage area), and enter in square miles on the spreadsheet.

Step 4 Regional curve bankfull dimensions are supplied by the spreadsheet program based on drainage area, based on work by Dunne and Leopold (1978) (fig. TS3C–3 (FISRWG 1998)). The data are based on typical relationships and may not be applicable to a specific watershed or area. For example, curve B bankfull widths and depths correlate reasonably well with observations of several hundred rural streams in Illinois ,but should be used cautiously (if at all) in an urban setting. Development of regional curve bankfull dimensions for streams in the subject hydro-physiographic area should be pursued for best results.

Step 5 Look for reference streamflow gaging data. USGS and some state and local governments may own or operate gaging equipment on the stream you are investigating. If not, look for the nearest gage data available in a watershed with similar soils, climate, and land use to the one you are investigating.

a. Gage data are available online at *http://www.usgs.org* for USGS-operated gages.

b. The Illinois NRCS stream stabilization spreadsheet has a pull-down menu of USGS gage data in and near the selected county. The 2-year return interval maximum discharge, Q2, calculated from the actual gage data will be displayed for the selected gage along with the station number and its drainage area. Results of the USGS regression analysis (USGS 1987) are also displayed, if available; they are not available for urban streams in Northeastern Illinois as the regression analysis does not represent urban hydrology. This feature is applicable only to Illinois streams. Further information on stream gage analysis is provided in NEH654.05.

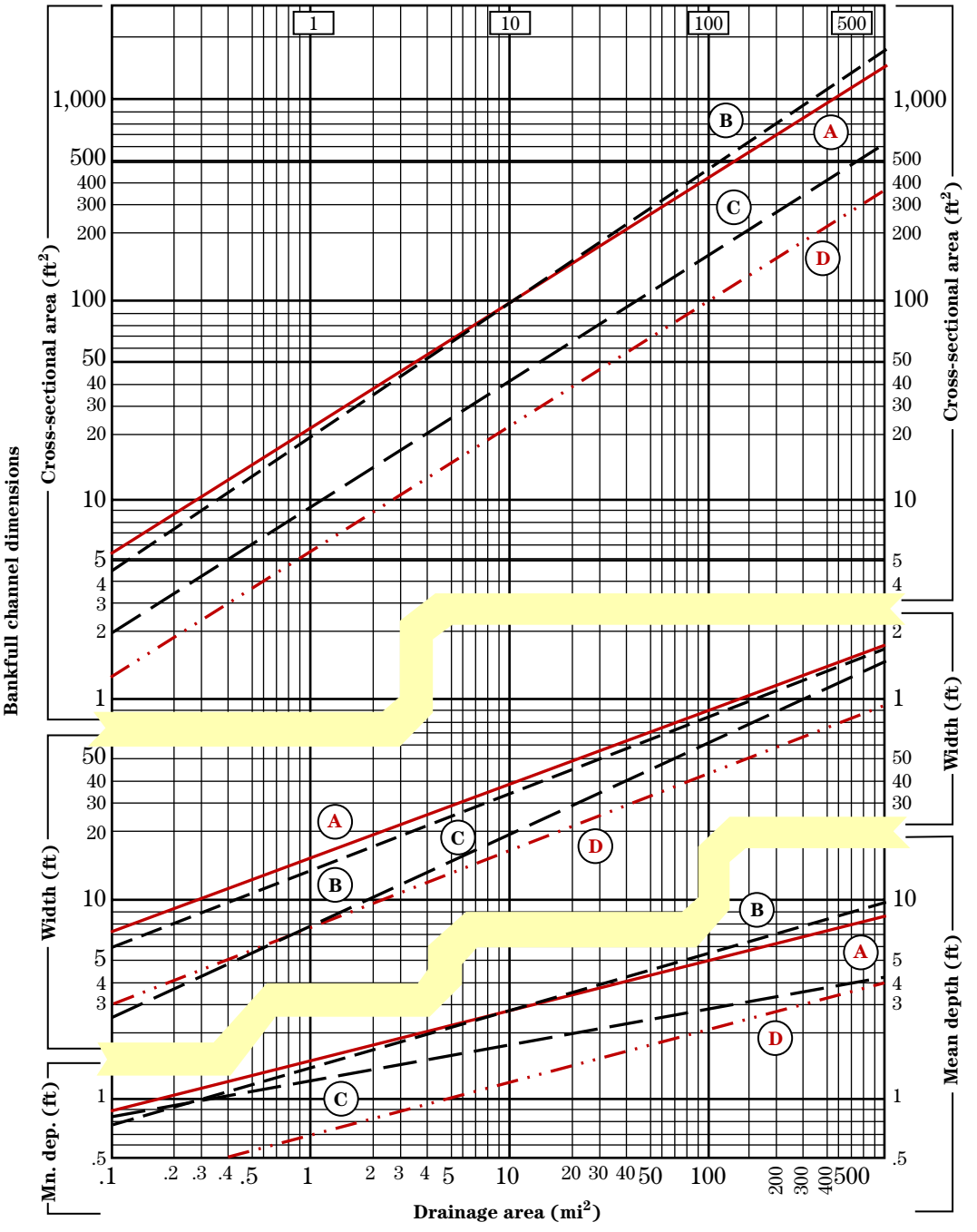
Step 6 To determine the USGS flood-peak discharge predictions for the subject stream, the spreadsheet needs a value for valley slope (USGS 1987). Rainfall and regional factor are automatically supplied based on the county selection, and the predicted Q2 discharge from the regression equation will be displayed. It will always display the typical range for bankfull, which is 40 percent to 80 percent of the Q2 discharge, corresponding to the approximate 1 to 1.5-year return interval storm event commonly representing bankfull flow in Illinois. If the subject stream is not in Illinois, use other data if available.

a. For the regression analysis, valley slope is defined as “the difference of elevations divided by distance between points 10 percent and 85 percent of the total distance measured along the low-water channel of the stream from the site to the basin divide” (USGS 1987). Divide the difference in elevation by total flowline distance between points, using the topographic map with delineated drainage area determined previously.

b. If desired, the spreadsheet valley slope subroutine (fig. TS3C–4) may be used. The subroutine prompts entries of topographic contour elevations and corresponding distances along the flow line of the channel. It automatically determines elevations at the critical points using linear interpolation and plots a profile of the channel to provide a visual model of the process.

Step 7 The sinuosity of the local stream site is best determined from a recent aerial photo. Identify the points where contour lines immediately upstream and downstream of the project site cross the stream channel. Measure the stream length along the channel between the two points, along with the valley length (a straight line measurement) between the same two points. Enter these distances on the spreadsheet, along with the contour interval, and the resulting sinuosity will automatically be determined.

Figure TS3C–3 Regional curves showing bankfull dimensions by drainage area



A San Francisco Bay region at 30 inches annual precipitation

B Eastern United States

C Upper Green River, Wyoming

D Upper Salmon River, Idaho (Emmett 1975)

Figure TS3C–4 Valley slope subroutine from stream stabilization spreadsheet

Valley Slope

USGS "Water Resources Investigations Report 87-4207" definition of valley slope:
"the difference of elevations divided by distance between points 10% and 85% of the total distance measured along the low-water channel of the stream from the site to the basin divide"

Project: John T
Assisted by: Wayne K
Date: 1/10/2007

back to I&E form

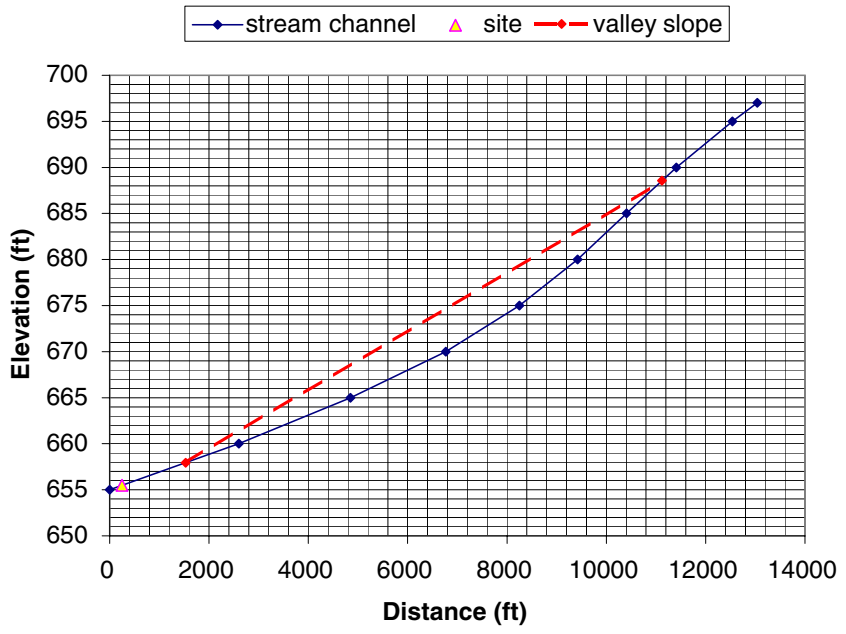
Valley Slope *(ft/mi)*
16.9

Clear Cells

Measured Data from Topographic Map:

Elevation at Downstream Contour *(ft)*	OR	*(m)*	Distance from Previous Contour *(ft)*
655			
660			2600
665			2250
670			1920
675			1480
680			1170
685			990
690			1000
695			1130
697			500

Stream Distance from Downstream Contour to Site *(ft)*	250



Field data collection

With the background data gathered and an understanding of the perceived problems and risks, the designer is ready to make a field visit to the site. Actual field measurements from the subject site are used to customize the analysis. The local stream morphology section of the spreadsheet is a way to record and interpret field observations of the bankfull condition.

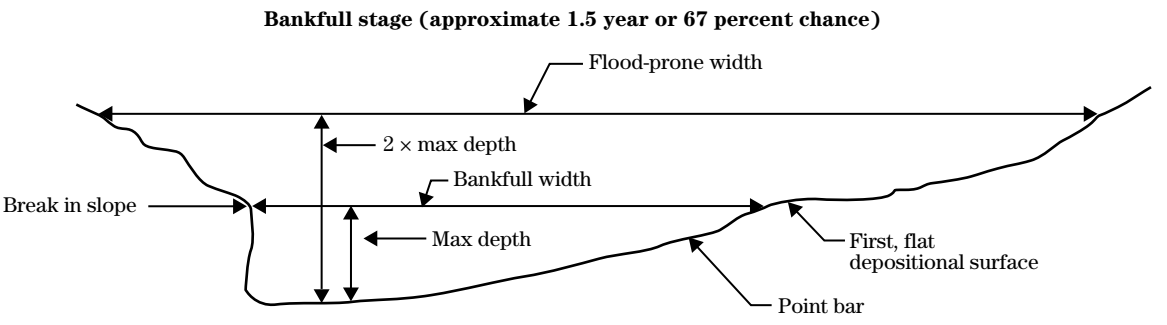
Step 1 Observe the roughness of the channel, which is affected by vegetation, obstructions, irregularities in cross section, and meandering. Select a value for Manning's *n* from the pull-down menu on the I&E spreadsheet, based on channel description.

Step 2 During the field visit, walk at least two meander lengths of the stream channel, identifying bankfull indicators. Mark the elevations of indicators with flags, and use a hand level or other survey instrument to determine the height above existing flowline. Best indicators are the first flat depositional surface, top of washed root zone, and a break in slope angle on the streambank.

Refer to figure TS3C–5 (Steffen, Roseboom, and Kinney 2000) for guidance on locating bankfull indicators. The regional curve predictions for channel dimensions (fig. TS3C–3) are mean depths. Bankfull indicators identified in the field will be measured at maximum bankfull depth, and maximum depth may be 0.5 to 2.0 times the mean bankfull depth predicted by the regional curve data. Therefore, during the field investigation, do not expect bankfull indicators to be found at the mean depth predictions unless the channel cross section is a flat bottomed rectangle. A further description on the identification of bankfull indicators is provided in NEH654.05.

Step 3 After measuring several bankfull indicator elevations, look for converging evidence to support your selection of indicators. When selected indicators are zeroed in to within a few tenths of a foot, take an average, and use the result as your field identified bankfull stage. Also, at a riffle location, measure the distance across the channel at the bankfull elevation. Note: If the channel is undergoing active downcutting (CEM stage 3 or 4 (fig. TS3C–2)), there will not be any reliable bankfull indicators.

Figure TS3C–5 Bankfull indicators used for field identification



Typical bankfull indicators may be:

- First, flat depositional surface
- Top of washed root zones
- Top of point bar or other deposits
- Change in size of substrate materials
- Lowest extent of woody vegetation
- Topographic break in slope
- Change in nature and amount of debris deposits
- Zone of washed rock

Step 4 Survey a cross section at the nearest riffle (fig. TS3C–6), extending out on each side at least to the flood plain elevation. The survey data will be used to calculate the cross-sectional area at the field identified bankfull stage. To determine a representative channel slope, survey at least several hundred feet along the streamflow line, at riffle locations. Since channel slopes are often quite flat, it is critical to take accurate measurements at a minimum of three or more riffles to determine channel slope.

Step 5 Measure the radius of curvature, Rc, (fig. TS3C–7 (FISRWG 1998)) of the channel bend(s) in the project area. Alternatively, this can be done using a recent aerial photo, if desired.

Step 6 During the field visit, measure the characteristics of the bed load. Larger cobbles indicate higher velocity flow. Sieve a bed load sample and do a pebble count, or estimate the D_{90} bed-load size (the size mesh through which 90 percent of the bed load would pass). Do the same for the D_{50} bed-load size. More information on sediment sampling is provided in NEH654 TS13A.

Data analysis and assessment

Analysis of the field data involves first determining the value of several standard parameters used to describe stream morphology: width-to-depth ratio, entrenchment ratio, sinuosity, and the ratio of radius of curvature to bankfull width. These parameters will be used to assess the condition of the stream and the potential for stabilization. Bankfull discharge and flow velocity are determined in several ways from the field data. The ultimate goal is to develop confidence in the analysis by matching discharge and velocity measurements from as many sources as possible.

Step 1 Plot the riffle cross section on the cross-sectional spreadsheet subroutine (fig. TS3C–6) and enter a flow depth equal to the maximum bankfull depth as determined from the field bankfull indicators. Cross-sectional area, velocity, discharge, and hydraulic radius will be computed using Manning's equation and displayed on the subroutine page. If the actual channel slope data is absent on the I&E sheet, the cross-sectional subroutine will use a slope estimate based on entries from the sinuosity determination.

Step 2 Width-to-depth ratio is determined from the bankfull width and the mean bankfull depth.

Step 3 Bankfull width can be entered directly from the field measurement, or measured from the plotted cross section.

Step 4 Mean bankfull depth can be determined by dividing the cross-sectional area at the field-determined maximum bankfull elevation by the stream width at the maximum bankfull elevation.

Step 5 The entrenchment ratio compares the bankfull width to the width of flow when the stream reaches twice the maximum bankfull depth for the bankfull discharge. On the I&E spreadsheet, enter maximum bankfull depth (from the cross section taken at the riffle) and the width of the channel or flood plain at twice the depth; the entrenchment ratio will be automatically determined.

Step 6 Enter the measured radius of curvature; its ratio to bankfull width is automatically calculated by the spreadsheet.

Step 7 Enter the discharge calculated by the cross-sectional subroutine at maximum bankfull depth as the selected Q on the I&E spreadsheet, or select your own best estimation of bankfull discharge based on all of the foregoing data (including the regression analysis and other background investigation).

Step 8 Enter the field-determined bed-load sizes on the spreadsheet.

Step 9 The spreadsheet will display a series of four bankfull velocity checks:

- velocity required to move D_{90} bed load
- velocity from cross-sectional subroutine (using Manning's equation on actual surveyed cross section and slope)
- velocity calculated from basic field data (using a modified Manning's equation with mean depth in place of hydraulic radius)
- velocity from the selected Q entry, using V=Q/A and a cross-sectional area determined from the basic field data section

Figure TS3C–6 Cross-sectional subroutine from stream stabilization spreadsheet

Natural Open Channel Flow

Project:	John T	
Assisted by:	Wayne K	
Date:	1/10/2007	
Channel Slope (***S***):	0.004580	*ft/ft*
Manning's ***n***:	0.035	
Flow Depth:	3.3	*ft*

$$Q = \frac{1.486}{n} A R^{\frac{2}{3}} S^{\frac{1}{2}}$$

assuming uniform, steady flow

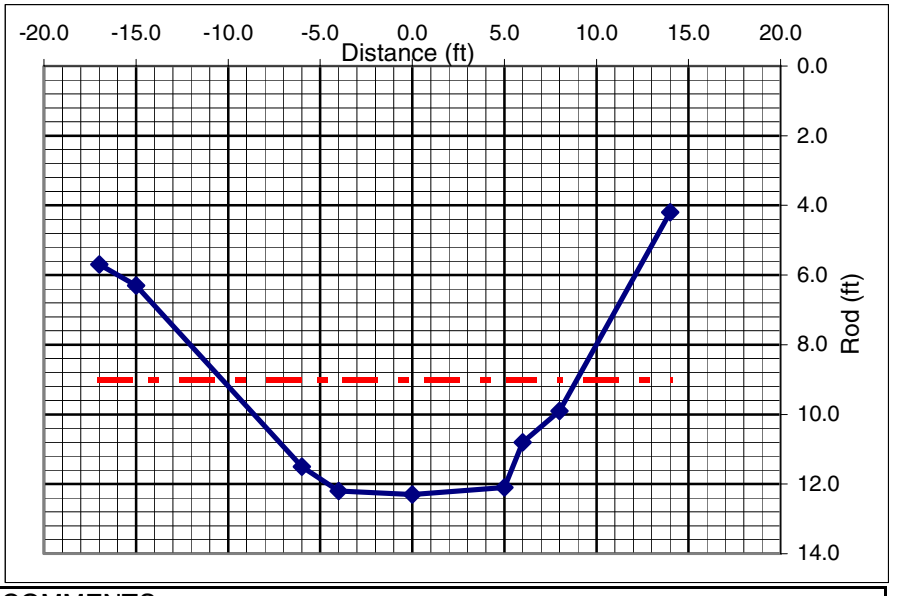
back to I&E form

Clear Cells

		Trial Depth 2	*Trial Depth 3*
Selected Flow Depth:	3.3 *ft*	4.0	5.0
Channel Flow (***Q***):	218.6 *cfs*	319.2	497.3
Channel Velocity:	4.8 *ft/sec*	5.3	6.0
Cross-Sectional Area (***A***):	45.7 *sq.ft.*	59.9	82.5
Hydraulic Radius (***R***):	2.1 *ft*	2.5	3.0

Survey Data:

Rod *(ft)*	Distance *(ft)*
5.7	-17.0
6.3	-15.0
11.5	-6.0
12.2	-4.0
12.3	0.0
12.1	5.0
10.8	6.0
9.9	8.0
4.2	14.0

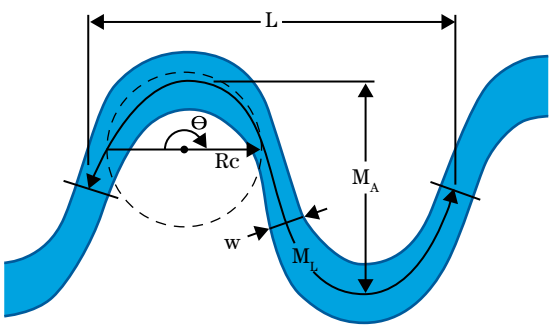


COMMENTS:

Step 10 Velocities from all four calculations should be very close and should be sufficient to move the D_{90} bed load. If more than 1.0 feet per second difference is observed between these four values, review to see if there is a mistake in data entry. If not, the bankfull indicators may be in error and need to be rechecked.

Step 11 After all the velocities compare well, compare the bankfull dimensions with those predicted by the regional curves, and compare the selected Q with the discharge predicted by the gage data and/or the regression equation. Modify entries as needed to develop confidence that the stream condition is understood. The field indicators should be the main guide, not the regional curve data or the regression equation predictions, as the field indicators are specific to the stream being investigated. Also, if the stream segment is in channel evolution stage 3 or 4, there will be no reliable bankfull indicators, and the designer will be forced to rely on flow relationships developed from other similar watersheds and experience gained from previous comparisons.

Figure TS3C–7 Typical stream morphology illustrating radius of curvature



L Meander wavelength
M_L Meander arc length
w Average width at bankfull discharge
M_A Meander amplitude
Rc Radius of curvature
Θ Arc angle

Departure analysis

Now that the designer has determined the bankfull or channel forming discharge in the stream segment, some analysis of the stream condition compared to stable streams can begin.

Condition 1: Is the flood plain elevation at or near the elevation of maximum bankfull depth?

Yes. The channel is connected to the flood plain. Discharges larger than bankfull begin to spread out over the flood plain, slowing velocities and dissipating energy. The channel has not experienced significant downcutting. CEM stage 1 or 6 would apply: a stable configuration. The entrenchment ratio (width at twice maximum bankfull depth/ bankfull width) will be greater than 2.5.

No. The channel is not connected to the flood plain. Discharges larger than bankfull will remain inside the channel with little or no opportunity to spread out onto the flood plain. This is evidence of current or past downcutting. The channel evolution process is active and its morphology is adjusting to regain equilibrium with flow characteristics. Incised channels such as this are likely to continue to erode laterally to build a flood plain. CEM stage could be 2, 3, 4, or 5. The entrenchment ratio will be less than 2.5. Entrenchment ratio will be smallest in stage 2 or 3 channels and then increase to about 2.5 or more as channel nears a new equilibrium in stage 6. The exception to this condition will be low-gradient, channelized streams with insufficient energy to erode the channel boundary, even when entrenched.

Condition 2: Is the channel bed in riffle locations comprised of bed-load material or is it residual (hard) silt, clay, or bedrock?

Bed-load material. The channel is probably not actively downcutting. Bed-load material is not being swept away by streamflow. If the entrenchment ratio is low (less than 2.5), the channel is most likely in the widening phase of the CEM, stage 4 or 5.

Residual (hard) silt, clay, or bedrock. Bed-load material is being swept out of this reach of channel, leaving the residual material exposed at the riffle locations. The channel is actively downcutting (CEM stage 3). If the streambed is not stabilized, this reach of stream will go through all six CEM stages and the degradation will advance upstream until it meets resistance in the form of bedrock, bridge floor, and culvert. Channels can be downcutting even when the entrenchment ratio is over 2.5. Streams are not considered entrenched until they degrade to twice the maximum bankfull depth, but degradation begins as soon as the bottom begins to be eroded.

Condition 3: Is the width-to-depth ratio less than 10 with an entrenchment ratio less than 1.4 (a deep, narrow channel)?

Yes. Width-to-depth ratios can be small (less than 10) in low gradient, fine-grained, or sinuous channels. However, these channel types are always connected to the flood plain in stable situations. Therefore, width-to-depth ratios less than 10, combined with entrenchment, are good indicators that downcutting has occurred in the past or is actively occurring at present (CEM stage 2, 3, 4 or 5). If, in addition, the sinuosity is low (less than 1.2), it is likely that the stream has been channelized to create the entrenched condition.

No. If width-to-depth is greater than 20, suspect an overwidened stream segment and sediment transport problems (CEM stage 5). This condition could indicate an aggrading stream segment.

Condition 4: Is the velocity calculated from the cross-sectional subroutine of the I&E spreadsheet much faster or much slower than that required to move the D_{90} bed-load material?

Much faster—Excessive velocities indicate that bed-load material is too small to resist existing velocities. Therefore, downcutting is probably occurring (CEM stage 3). Check the status of condition 2. Streams with only very fine-grained bed-load material will have excessive velocities compared to D_{90} material size. Vertical stability of these streams cannot be assessed using bed-load material size estimates.

Much slower—Slow velocity could indicate an aggrading system where the heavy bed load generated upstream cannot be transported through the system. These conditions often occur in delta areas above impoundments or at confluences with larger streams. They also occur when channel velocities change due to slope changes (at the downstream end of a channelized reach), when width-to-depth ratios increase dramatically or when there is an exceptionally large contribution of bed load just upstream.

Condition 5: Is the radius of curvature-to-bankfull width (Rc/W) ratio less than 1.8?

Yes. The situation is outside of the normal range of planform stability. It may be necessary to realign the channel or walk away from the project. Natural, stable channel radius of curvature-to-bankfull width ratios vary widely, but most commonly range from 2.3 to 2.7 or higher. With a radius of curvature-to-bankfull width ratio less than 1.8, the possibility of a channel cutoff at this point increases dramatically.

I&E spreadsheet details

The inventory and evaluation function of the stream I&E spreadsheet includes the following introduced in the discussion of suggested I&E procedure (figs. TS3C–4, TS3C–6, and TS3C–8) in this technical supplement:

- streambank I&E form
- cross-sectional subroutine
- valley slope subroutine

In addition to the above, the spreadsheet also includes design sheets to determine dimensions and material quantities for certain standard stream stabilization practices, and automatically fills out the applicable Illinois standard drawings:

- rock riffles
- stone toe protection
- stream barbs

Figure TS3C–8 Stream stabilization I&E form from spreadsheet

ILLINOIS NRCS - Version 2.06- modified 6/06 R.Book

County Jefferson T. 4S R. 1E Sec. 22

Date 1/10/2007 By Wayne K

Stream Name Happy Creek UTM Coord.

Landowner Name John T

Drainage Area 2.67 sq. mi. Clear Cells

Regional Curve Predictions:

Bankfull dimensions	Width	22 ft.	Cross Sectional Area	44 sq. ft.
	Depth	2.0 ft.		

Reference Stream Gage:

Sevenmile Creek near Mt. Vernon	Station No.	05595800	Gage Q_2 1030 cfs
	Drainage Area	21 sq.mi	Regression Q_2 1410 cfs
Jefferson County, IL		**REFERENCE STREAM DATA ONLY**	

USGS Flood-Peak Discharge Predictions:

Valley Slope:		*ft./mi. (user-entered)*			Regression Q_2 295 cfs
	16.9	*ft/mi (from worksheet)*	Rainfall	3.40 in *(2 yr, 24 hr)*	Adjusted Q_2 216 cfs
	0.0032	*ft./ft.*	Regional Factor	0.983	Typical Range for Bankfull Discharge:
					80 to 180 cfs

Local Stream Morphology:

Channel Description: (b) Same as (a), but more tones and weeds

Manning's "n" 0.035

			Stream Length	1000	*ft.*
Basic Field Data:			Valley Length	1000	*ft.*
Bankfull Width	13	*ft.*	Contour Interval	5	feet
Mean Bankfull Depth	3.2	*ft.*	Estimated Sinuosity	1.00	
Width/Depth Ratio	4.06				
			Channel Slope:		Bankfull Q from:
Max. Bankfull Depth	4.2	*ft.*	Surveyed:	0.00458 *ft./ft.*	Cross-Section 219 *cfs*
Width at twice max. depth	300	*ft.*	Estimated:	0.00500 *ft./ft.*	Basic field data 260 *cfs*
(8.4 ft.)					Selected Q 224 *cfs*
Entrenchment Ratio	23.08		Radius of Curvature (Rc)		ft.
			Rc/Bankfull width:	0.00	

Bankfull Velocity Check: *(typical Illinois streams will have average bankfull velocity between 3 and 5 ft/sec.)*

Bedload:	D_{90}	1 *in.*	Velocity required to move D_{90}:	2.1	*ft./sec.*
	D_{50}	*in.*	Velocity from Cross-Section data:	4.78	*ft./sec.*
GOAL: Develop confidence by matching			Velocity from basic field data:	6.26	*ft./sec.*
velocities from different sources.			Velocity from selected Q:	5.4	*ft./sec.*

Channel Evolution Stage III Stream Type (Rosgen)

Notes

Technical Supplement 3D

Overview of United States Bats

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Bats contribute to the overall health of riparian ecosystems.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 3D

Overview of United States Bats

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Technical Supplement 3D

Overview of United States Bats

Introduction

In the past, bats (order *Chiroptera*) have been one of the most feared and misunderstood creatures (fig. TS3D–1). Today, as researchers are beginning to unravel the secrets of the world's only flying mammals, bats are declining worldwide. Misconceptions and folklore concerning bats have been passed along for generations, and unfortunately, this has led to the senseless killing of colonies of millions of bats, sometimes in a single destructive act. Most bats only produce one pup a year; therefore, recolonizing a decimated population and replenishing a once occupied cave may take decades. With thanks to extensive conservation and education, bats are making a comeback both in the United States and worldwide.

The diversity of bat species, the habitats they occupy, and their behavioral and social features are impressive (Fenton 1997). Occurring worldwide except for the Polar regions and a few isolated islands, there are more than a thousand known bat species, and this number continues to increase (Mickleburgh, Hutson, and Racey 2002; Engstrom and Reid 2003). Some species, such as Mexican free-tailed bats (*Tadarida brasiliensis*), undertake extensive seasonal migrations. Other species migrate for food, such as the long-nosed bats (*Leptonycteris* spp.) and the Mexican long-tongued bat (*Choeronycteris mexicana*), which are thought to travel along nectar corridors. Species such as endangered gray bats (*Myotis grisescens*) and Indiana bats (*M. sodalis*) migrate more locally between summer roosts and winter hibernacula. Elevational migration routes may also occur. They are thought to be a strategy of the spotted bat (*Euderma maculatum*), which lives in southwestern regions.

Figure TS3D–1 Southeastern bat (*Myotis austroriparius*)

Bats have an extraordinary dietary diversity. Different species feed specifically on insects, fish, frogs, blood, fruit, or nectar. In the United States, bats are voracious feeders on night-flying insects, and three species of nectar/pollen feeding bats live in the extreme southern regions bordering Mexico. Insectivorous species typically consume more than 50 percent of their body weight in bugs nightly (Harvey, Altenbach, and Best 1999). Consider the 20 million Mexican free-tail bats from Bracken Cave, Texas, the largest concentration of mammals in the world. They devour approximately 200 tons of pests a night. Only 150 big brown bats (*Eptesicus fuscus*) are required to protect farmers from 33 million corn rootworms (*Diabrotica* spp.) each summer Bat Conservation International (BCI) 2000. The pollinating species of bats in the Southwest is important for the survival of agaves and columnar cacti. The beneficial and economical role bats play for humans and ecosystems is clear.

Habitat

An increased availability of roost diversity usually corresponds to an increased population and diversity of bat species (Findley 1995). Cave bats or forest-dwelling species are generally colonial species that roost in caves, crevices, hollow trees, or under loose bark. The Indiana bat, for example, uses caves for hibernation during the winter months and tree cavities or beneath exfoliating bark of various trees in the summer (Kurta et al. 1992). Some cavity dwelling species will also use manmade structures such as mines, bridges, culverts, and bat houses (fig. TS3D–2). Several suitable roosts

in an area may provide a colony of bats with the necessary thermal variation required throughout the day and allow escape from parasites or predators (Lewis 1995).

Tree bat species typically roost solitarily or in small groups in foliage or moss at different canopy levels. Young red bats (*Lasiurus borealis*), for example, have been observed roosting higher in trees than adults (Constantine 1966). Studies have suggested that female and male bats use different habitats and roosting sites (Brigham 1991; Cryan, Bogan, and Altenbach 2000). Findley (1995) suggests that differences in morphology, flight maneuverability, and echolocation proficiency affect partitioning of foraging and roosting habitat between species. Thus, species, age, sex, reproductive condition, or migratory status may account for differences in roost and habitat selection among bats. Overall, a diverse landscape and forest stratification with a multifaceted arrangement of potential roosting and foraging areas, even in suburban areas, appears to be important for healthy bat populations (Evelyn, Stiles, and Young 2004).

Since fresh water is critical to their survival, bats are closely associated with riparian environments (Martin 2001) (fig. TS3D–3). Riparian areas are of particular importance to bats, possibly due to their high resource of flying insects (Barclay 1991), especially for species in arid regions (Bell 1980).

Riparian areas also offer an abundance of snags, which are important roosting sites for many species of bats. Tuttle (1976) suggests that roost selection may be determined by proximity to required resources such as water, forage areas, and hibernation sites, thus, reducing energy expenditure by reducing travel distance. For example, there is a decreased growth rate and higher mortality rate among juvenile gray bats where greater distance is traveled from roosting sites (caves) to their preferred foraging habitat, which is over water. Research also suggests that riparian zones act as important travel corridors, space for open flight, and forest edges which are frequently used for feeding and migration by some bat species (Wunder and Carey 1994).

Status and impacts

Of the 45 species of bats that occur in the continental United States and Hawaii, 6 species are considered to be federally endangered and 20 are species of concern (Harvey, Altenbach, and Best 1999). The International Union for Conservation of Nature and Natural Resources lists 10 bat species on the Red List of Threatened Species (IUCN 2002) (table TS3D–1). Species of concern, former category 2 candidates, are those sensitive species in which data pertaining to biological vulnerability and threat is not yet available to justify a

Figure TS3D–2 Maternal colony of Rafinesque's big-eared bats (*Corynorhinus rafinesquii*) roosting beneath a concrete bridge

Figure TS3D–3 Healthy riparian areas are important to bats for roosting and foraging

threatened or endangered status. Due to their nocturnal behavior, capability of flight, and the frequent remote location of their roosts, bats are one of the most difficult groups to research and to monitor. Although many species of bats in the United States appear to be declining, little is known about the populations and ecology for many species (Arnett 2003; O'Shea, Bogan, and Ellison 2003).

Intentional killing, vandalism, cave exploration and commercialization, and closure of abandoned mine entrances have greatly reduced roosting habitat for many bat species in the United States (Harvey, Altenbach, and Best 1999). Disturbance to hibernating bats can be detrimental due to the potential loss of needed energy reserves, which must last until summer emergence. Endangered gray bats, for example, are especially vulnerable since 95 percent of the population hibernate in only 11 caves in the Southeast (Harvey, Altenbach, and Best 2001). Disturbance to maternal colonies when newborn are present may also be injurious to bats since frightened mothers may drop their young or abandon the roost (Harvey et al. 1999). Natural disasters such as flooding can also effect populations, but human disturbance is the primary cause of their decline.

Loss of healthy riparian systems, especially in the southwest where permanent water sources are in decline, also negatively impact bat populations. Use of pesticides and other toxicants may contaminate water and food sources, and the loss of mature trees and snags may limit roost availability. Other causes of decline may include clearcutting, strip mining, and human encroachment into dwindling habitats (Martin 2000).

Conservation

Fortunately, a greater understanding of these beneficial creatures has led to great strides in their conservation. For example, protection of caves and mines through properly designed gates has shown considerable success for endangered and sensitive species (Tuttle 1977; Tuttle and Taylor 1998) (fig. TS3D–4). Placement of artificial roosts in roost-deficient areas or where colonies have been evicted from homes or buildings has also been valuable to various bat species. Burke (1999), Brittingham and Williams (2000), and Arnett and Hayes (2000) attracted bats to flat-bottomed bridges by installing specially constructed bat boxes. Continued research, innovation, management, and education have been, and will be, critical for the future of this unique group of animals.

Nonprofit organizations such as BCI have been educating people, advancing research efforts, and establishing collaboration efforts around the world for 20 years. The North American Bat Conservation Partnership (NABCP) was established in 1999 to support continentwide conservation efforts. They formed an alliance of working groups, researchers, nongovernmental organizations, and state and Federal agencies from Canada, the United States, and Mexico. This partnership has identified conservation priorities through a strategic plan, which will guide the future direction of research, education, and management (Keeley, Fenton, and Arnett 2003).

Another collaboration, the Program for the Conservation of Migratory Bats between Mexico and the United States (PCMM), was developed in 1995 due to declining bat populations in Mexico. Its objectives are to protect and conserve migratory species and to sustain their ecological roles and evolutionary processes (Medellin 2003). Partnerships and collaboration efforts such as these have demonstrated their important role in curtailing the rapid decline of bats in the United States and worldwide.

Figure TS3D–4 Sauta Cave National Wildlife Refuge serves as critical protected habitat for gray and Indiana bats.

Table TS3D–1 General habitat, distribution, and status of federally protected bat species in the United States

Species	Status		U.S. distribution	General habitat
	USFW	IUCN		
Phyllostomatidae				
California leaf-nosed bat (*Macrotus californicus*)	SOC	VU	Southern CA, AZ extending into the southern tip of NV, and the extreme western portion of NM	Lowland desert habitat; abandoned mine tunnels may be used as day roosts and night roosts may include buildings, bridges, porches, or rock shelters
Mexican long-tongued bat (*Choeronycteris mexicana*)	SOC	LR/nt	Southern portion of CA, AZ, and southern tip of NM	Occupies a range of habitats from arid thorn shrub to tropical deciduous forest and mixed oak conifer; inhabits caves, buildings, and abandoned mines
Lesser long-nosed bat (*Leptonycteris curasoae yerbabuenae*)	FE	VU	South central and south eastern part of AZ and the extreme southern region of Mexico	Desert-scrub habitat; occupies abandoned mines and caves in areas consisting of agaves, yuccas, saguaros, and organ pipe cacti
Greater long-nosed bat (*Leptonycteris nivalis*)	FE	EN	Big Bend region of TX	Occupies a range of habitats from sparsely vegetated deserts to pine-oak woodlands; generally inhabits deep caverns, but will use hollow trees, mines, culverts, and buildings
Vespertilionidae				
Spotted bat (*Euderma maculatum*)	SOC	—	West central U.S.	Mostly occupies rocky arid to semiarid terrain such as desert, scrub areas, or ponderosa pine forest
Allen's big-eared bat (*Idionycteris phyllotis*)	SOC	—	Extreme southern NV, southern third of UT, throughout AZ, and the southwestern quarter of NM	Riparian habitats above 3,000 feet; common in coniferous forests and pine-oak forest canyons; maternity colonies of 30 to 150 individuals have been found in mine shafts, boulder piles, lava beds, and under bark of large ponderosa pine snags
Hawaiian hoary bat (*Lasiurus cinereus semotus*)	FE	—	Hawaiian Islands: Kauai, Oahu, Maui, and Hawaii	Coastal and lowland forested areas; on Kauai, occurs primarily in open wet areas near forests; roosts in trees or rock crevices
Southeastern bat (*Myotis austroriparius*)	SOC	—	Wide spread distribution in the Southeast	Roost primarily in caves in the North and in the South will utilize buildings, bridges, hollow trees; maternity colonies have been located mainly in caves and hardwood swamp areas
Western small-footed bat (*Myotis ciliolabrum*)	SOC	—	Western U.S.	Arid habitats associated with cliffs, talus fields, prairies; roosts in crevices, clay banks, beneath rocks in the ground, and under bark in barns; hibernates in caves and mines
Western long-eared bat (*Myotis evotis*)	SOC	—	Western U.S.	Coniferous forests, typically only at higher elevations in southern areas (between 7,000 and 8,500 feet) and semiarid shrublands, sage, chaparral, and agricultural areas; roost in tree cavities, beneath exfoliating bark of both living trees and dead snags., buildings, cliffs, and sink holes; pregnant bats often roost at ground level in rock crevices, fallen logs, and in the crevices of sawed-off stumps

Table TS3D–1 General habitat, distribution, and status of federally protected bat species in the United States—Continued

Species	Status		U.S. distribution	General habitat
	USFW	IUCN		
Gray bat (*Myotis grisescens*)	FE	EN	Cave regions of AR, MO, KY, TN, and AL	Year-round cave residents although different caves are utilized for winter and summer; hibernation sites are typically deep caves with large rooms capable of trapping cold air; maternal caves usually contain streams with configurations capable of trapping heat
Eastern small-footed bat (*Myotis leibii*)	SOC	—	Eastern U.S.	Hibernate in caves and mines often near entrances, cracks in the floor, and under rock slabs in quarries; use caves, buildings, and barns in summer
Arizona bat (*Myotis lucifugus occultus*)	SOC	—	Southwestern U.S.	Ponderosa pine, oak woodlands, riparian forest in desert areas; roosts buildings, crevices, bridges, rarely in mines
Indiana bat (*Myotis sodalis*)	FE	EN	Cave regions in the Eastern U.S.	Caves used in winter and maternal colonies inhabit hollow trees or under exfoliating bark usually in floodplain deciduous forests or upland stands adjacent to riparian or floodplain forests; generally several suitable trees are required for roost switching
Fringed bat (*Myotis thysanodes*)	SOC	—	Western U.S.	Oak and pinion woodlands most common, also ranges from fir-pine areas to desert-scrub; roosts in caves, mines, and buildings
Cave bat (*Myotis velifer*)	SOC	—	Southern KS, western OK, and the southwestern states	Cave regions from south central Kansas to central Texas, rocky canyons, and desert flood plains; summer roosting sites include caves, mines, and sometimes buildings and under bridges
Long-legged bat (*Myotis volans*)	SOC	—	Western U.S.	Forested mountainous regions usually at elevations of 4,000 to 9,000 feet most common, also stream arid and streamside environments; caves and mine tunnels used in winter, summer roosts are tree cavities, crevices and under bark, rock and stream bank crevices, and buildings
Yuma bat (*Myotis yumanensis*)	SOC	—	Western U.S.	Variety of habitats with a nearby permanent water source; most often roost in buildings or bridges, sometimes in mines or caves, bachelors sometimes use abandoned cliff swallow nests
Rafinesque's big-eared bat (*Corynorhinus rafinesquii*)	SOC	VU	Southeastern U.S.	Historical distribution similar to that of cypress swamps; northern populations hibernate in caves and mines and more southern populations use cisterns or wells, maternal colonies utilize large hollow trees, abandoned homes and buildings, and bridges
Virginia big-eared bat (*Corynorhinus townsendii virginianus*)	FE	VU*	KY, NC, VA, WV	Limestone karst regions associated with mature hardwood forests; uses caves and abandoned mines as both summer maternity roosts and winter hibernacula, rock shelters
Ozark big-eared bat (*Corynorhinus townsendii ingens*)	FE	VU*	AR, OK, possibly MO	Limestone karst regions associated with mature hardwood forests; uses caves and abandoned mines as both summer maternity roosts and winter hibernacula

Table TS3D–1 General habitat, distribution, and status of federally protected bat species in the United States—Continued

Species	Status		U.S. distribution	General habitat
	USFW	IUCN		
Western big-eared bat (*Corynorhinus townsendii pallescens*)	SOC	VU*	Along the west coast	Roosts include abandoned buildings, bridges, and tunnels
Townsend's (Pacific) big-eared bat (*Corynorhinus townsendii*)	SOC	VU*	Western U.S.	Roosts include abandoned buildings, bridges, and tunnels
Molossidae				
Florida mastiff bat (*Eumops glaucinus floridanus*)	SOC	—	Southern tip of FL	Hardwood hammocks
Western mastiff bat (*Eumops perotis californicus*)	SOC	—	Southwest U.S.	Areas with natural springs; roosts in crevices high in cliffs
Underwood's mastiff bat (*Eumops underwoodi*)	SOC	LRnt	South central AZ	Organ Pipe Cactus National Monument, Baboquivari Mountains; roosts in woodpecker cavities within saguaro cacti
Big free-tailed bat (*Nyctinomops macrotis*)	SOC	—	Southwest U.S.	Rocky habitats; roosts in crevices in cliffs, known to use buildings
Mexican free-tailed bat (*Tadarida brasiliensis*)	—	LR/nt	Southern U.S. (largest populations in the West)	Occupy various habitats ranging from desert to pine-oak forests; utilize limestone caves and abandoned mines in the Southwest, manmade structures such as bridges and buildings in the Southeast, colonies also found in hollow trees

FE = Federally endangered
SOC = Species of concern
EN = Endangered
VU = Vulnerable
LR/nt = Lower risk/near threatened

Notes:
* Subspecies are not distinguished

Technical Supplement 3E

Rosgen Stream Classification Technique—Supplemental Materials

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: The Rosgen stream classification system uses morphometric data to characterize streams.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 3E

Rosgen Stream Classification Technique—Supplemental Materials

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Technical Supplement 3E

Rosgen Stream Classification Technique—Supplemental Materials

Purpose

Rivers are complex natural systems. A classification system is often used to stratify river reaches into groups that share common physical characteristics. A stream classification system provides better communication among those studying river systems and promotes a better understanding of river processes. The river classification system presented in this technical supplement is based on measurable physical parameters.

The stream classification presented in this technical supplement is condensed from the more detailed version by Rosgen (1994). It is intended for planning purposes, but is not sufficient for design. Appropriate data for use in river classification systems can be obtained from simple measurements and estimates.

The objectives of the Rosgen stream classification are to:

- assimilate a relatively complex mix of mathematical relationships that describes a type of river and simplifies it into a system that can be understood
- provide a consistent and reproducible frame of reference for those working with river systems and communicating stream morphology among a variety of disciplines and interested parties and enable people to talk in common terms about streams
- predict a river's behavior from its appearance and better understand cause and effect relationships (the specific measured stages in channel evolution models)
- provide a mechanism to extrapolate site-specific data to stream reaches having similar characteristics
- encourage thinking about stream processes relative to channel evolutionary changes and trends
- provide a tool to define a target such as the stable reference reach or desired form to aid in departure analysis and for setting objectives for restoration or rehabilitation

Data requirements

The Rosgen stream classification can be met through a hierarchical assessment of channel morphology measured based on bankfull dimensions. In order, the hierarchical attributes are:

- single-threaded or multiple-threaded channels
- entrenchment ratio
- width-to-depth ratio
- sinuosity
- slope
- material size D_{50} median particle size bed material

Stream reach

The classification applies to segments or reaches of the stream as defined by the user. However, any given stream reach should be comparatively uniform in its physical and biological characteristics.

Stream reaches can be defined from U.S. Geological Survey (USGS) 7.5-minute quadrangles, aerial photographs of appropriate resolution, and confirmed by field reconnaissance. Soil and geologic maps may also provide helpful information for delineating stream reaches.

Plan view (planform) type and level I classification

Level I classification is related to basin relief, landform, and valley morphology. This broadest characterization level is used only where general classification is required. The dimensions, patterns, and profiles are based on information from topographic and/or landform maps and aerial photography. The intent of level I classification is for a broad characterization that integrates landform and fluvial features of valley morphology with channel relief pattern, shape, and dimension (table TS3E–1).

A geomorphic characterization describing A through G stream types completes the level I classification. The stream type is based upon the measures of the stream in plan view from topographical or ortho-digital maps (fig. TS3E–1 (Rosgen 1996)). The plan view is classified as straight, sinuous (meandering), sinuous with active point bars, or braided (numerous intertwining channels separated by longitudinal and/or transverse bars). Some reaches may have actively eroding banks, anastomosing (multiple narrow and deep channels with extensive well-vegetated bars and flood plains), tortuous (extremely contorted meanders), or highly sinuous low width-to-depth bankfull channels.

Aerial photographs of sufficient resolution to show the plan view of the channel bed are good tools depending on the channel size and age of air photos. Judgments should be verified by field reconnaissance, since channels are dynamic and can change their plan view character over time. Photographs and field observations are best obtained at times of low flow and optimum visibility. Flood stages or vegetation can mask important features from observation and could result in misidentification of the plan view type.

A general longitudinal profile, which can be inferred from topographic maps, serves as the basis for breaking the stream reaches into broad slope categories that reflect profile morphology.

The shape of the cross section indicating a narrow and deep stream or a wide and shallow stream can be inferred at the broad level I characterization. The manner in which the channel is incised into the valley can be also be deduced at this level. For example, A and G stream types are narrow, deep, confined, and entrenched. F stream types are wide and shallow and are entrenched. Stream types D and C are wider and shallow with well-developed flood plains. E stream types are narrow and deep with well-developed flood plains. B stream types typically have moderately developed flood-prone areas in narrower but steeper valleys than D, C, and E types. Table TS3E–2 includes aerial and ground photos of the eight major stream types, A through G.

Valley types

Stream type, width-to-depth ratio, sinuosity, entrenchment, and other morphological features are dependent upon the valley development. For an initial broad level association of stream types, valley types are invaluable to level I classification and as a general indication of morphological pattern. Table TS3E–3 provides more detailed description of the 11 valley types used in the Rosgen stream classification system.

Channel slope—level I

Over substantial distances, elevations, and channel lengths can sometimes be determined from a combination of USGS 7.5-minute quadrangles and aerial photos. Aerial photographs can be used to calculate sinuosity. Thus, slope can often be calculated by first multiplying sinuosity times stream length from the quad maps, then second, by dividing the difference in elevation by the total sinuous stream length.

Table TS3E–1 Plan view characteristics of Rosgen stream type

Stream type	Plan view
Aa+, A	Relatively straight
B	Slightly sinuous
F, G	Moderately sinuous, F or Bs may have active point bars
C	Sinuous with active point bars
D	Multiple thread, braided
DA	Multiple thread, anastomosed
E	Tortuous and/or highly sinuous

Figure TS3E–1 Major stream types

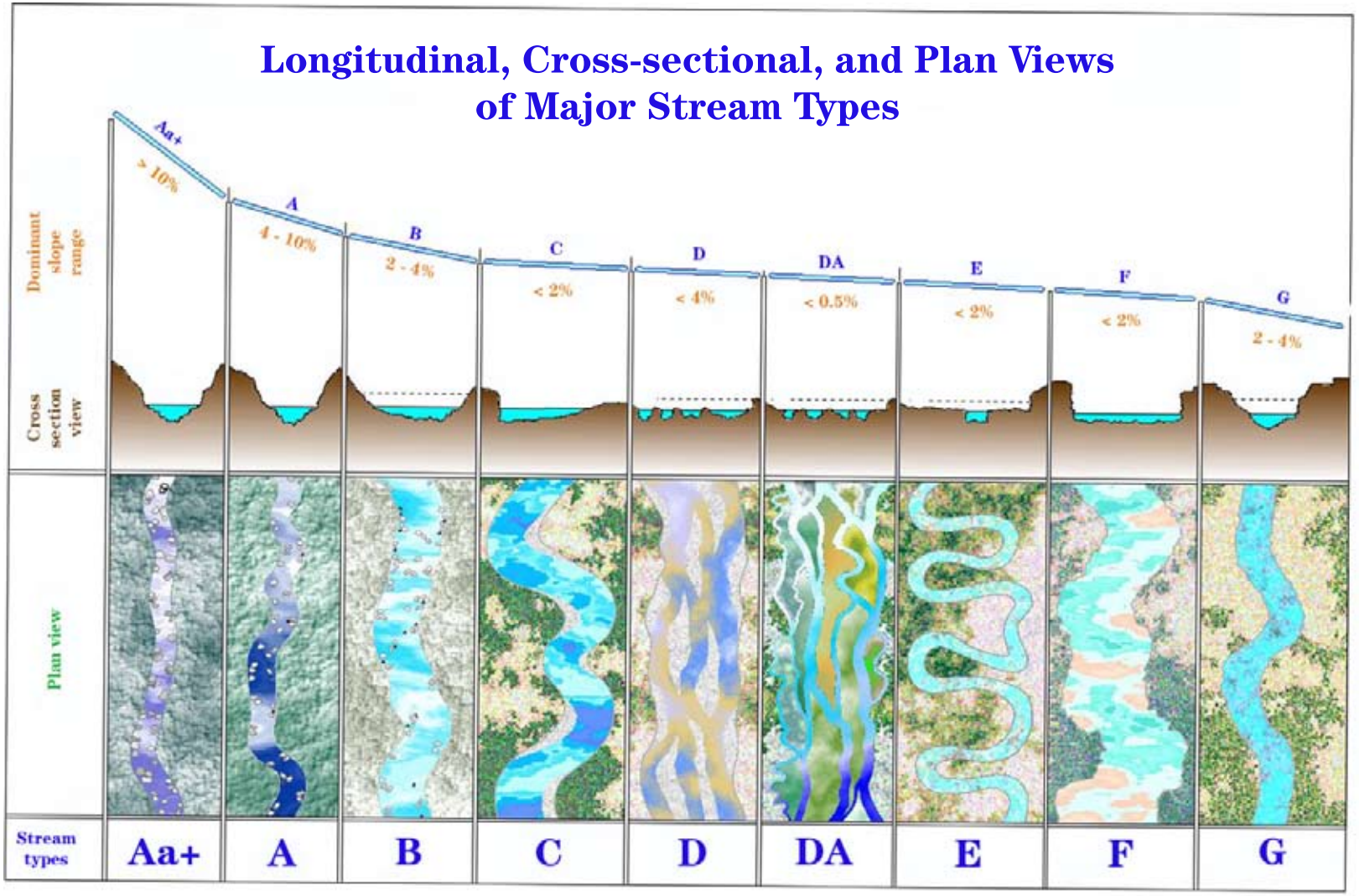
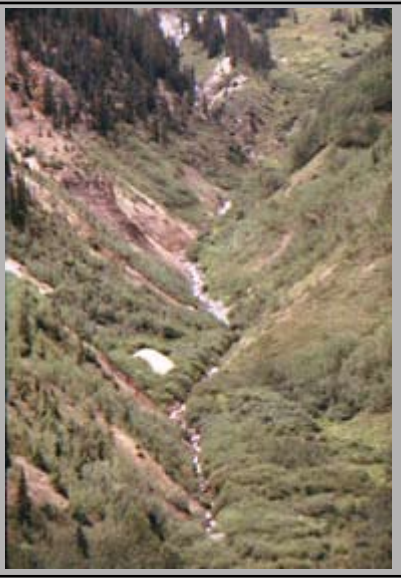


Table TS3E–2 Aerial and surface views of major stream types

Stream type and bedforms	Aerial view	Surface view
A Step-pool and/or cascade and/or chute bed		

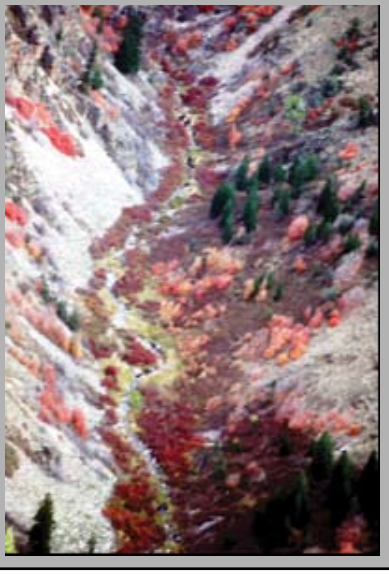
Stream type and bedforms	Aerial view	Surface view
B Step-pool and/or plane-bed and/or pool-riffle		

Table TS3E–2 Aerial and surface views of major stream types—Continued

Stream type and bedforms	Aerial view	Surface view
C **Pool-riffle and/or plane-bed and/or ripple-dune**		

Stream type and bedforms	Aerial view	Surface view
D **Braided** **Some pool-riffles develop on patterns of convergence and divergence**		

Table TS3E–2 Aerial and surface views of major stream types—Continued

Stream type and bedforms	Aerial view	Surface view
DA **Anastomose** **Braided pool-riffle or ripple-dune**		

Stream type and bedforms	Aerial view	Surface view
E **Pool-riffle or ripple-dune** **Depositonal bars sometimes not present**		

Table TS3E–2 Aerial and surface views of major stream types—Continued

Stream type and bedforms	Aerial view	Surface view
F **Pool-riffle or ripple-dune** **Depositonal bars sometimes not present**		

Stream type and bedforms	Aerial view	Surface view
G **Step-pool with some instances of plane-bed forms**		

Table TS3E–3 Valley type, morphological description, and stream type association

Valley type	Description	Stream type association
I	Steep V-shaped confined, highly dissected fluvial slopes greater than 2 percent	A and Aa+
II	Moderate relief gentle sloping side slopes with a parabolic valley bottom form often in colluvial valleys	B
III	Primarily depositional, usually steep, greater than 2 percent valley slope with debris-colluvium or alluvial fan landform	A, B, G, and D
IV	Gentle gradient canyons, gorges and confined alluvial valleys such as the Grand Canyon. Valley floors are typically less than 2 percent	F
V	U-shaped glacial-fluvial troughs with slopes generally less than 4 percent. Landforms typically include lateral or terminal moraines, alluvial terraces and flood plains. Trough is typically the result of glacial scouring process	C, D, G
VI	Fault control valleys, structurally controlled and dominated by colluvial slope building processes. Moderately steep with slopes less than 4 percent. G stream types observed under fault disequilibrium	Mostly B with C and F; some G
VII	Steep highly dissected fluvial slopes typically in either colluvium, alluvium or in residual soil. Active lateral and vertical accretion (Badlands of SD)	A and G
VIII	Mature wide gently valley slopes with well developed flood plain features adjacent to river terraces. Alluvial terraces and flood plains are predominate landforms. Depending on local streambed and riparian conditions D, F, and G stream types can be found. Gentle slopes with the alluvial valley fills	C and E D, F, and G
IX	Glacial outwash and/or eolian sand dunes. Moderate to gentle slopes. High sediment supply either single- or multiple-threaded channels	C and D
X	Very broad and very gentle slopes with extensive flood plain development. Often associated with lacustrine and gentle alluvial slopes. G and F streams are common when local base grades have been changed	E or C and DA, G, and F
XI	Large river deltas and tidal flats constructed of fine alluvial materials originating from riverine and estuarine depositional processes. Extremely gentle slopes with base grade controlled by sea or lake levels. Most often distributary channels, wave, or tide dominated	DA C and E

Morphological description (level II classification)

The level II classification process provides a more detailed morphological description of the stream based on field collected data. It includes assessments of channel entrenchment, dimensions, patterns, profiles, and bed materials. It uses a more finely resolved hierarchal criterion to stratify types and address general characteristics such as sediment supply, stream sensitivity to disturbance, potential for natural recovery, channel responses to flow regime change, and fish habitat potential. The morphological description level requires the computation of the entrenchment ratio, width-to-depth ratio, sinuosity, slope, and D_{50} or dominant particle size determination (fig. TS3E–2 (Rosgen 1996)).

Channel slope—level II

The channel slope should be determined for each stream reach being classified. It consists of the difference in elevation of the water surface or bed through the reach divided by the length of the channel reach. The elevations are usually obtained at the upper and lower ends of each bed feature. When water surface slope is field measured it is preferable to measure through at least two meander wavelengths. For example, the average slope is calculated from the top of the riffle to the top of the next downstream riffle (fig. TS3E–3). The channel length used in the calculation is the centerline length of the channel between the two points used for elevations.

In practice, the average low-flow water surface slope is the same as the average bankfull stage slope and is an accurate representation of slope needed for classification. The average slope of the water surface is generally measured through 20 to 30 channel widths. The higher water surface profile can be obtained from standard hydraulic methods.

Bankfull discharge validations

Dimensions measured and characterized in Rosgen's geomorphic stream classification system are based on bankfull discharge. A complete description of bankfull discharge is provided in NEH654.05. It is advised that a field validation of bankfull discharge and associated return intervals be completed at USGS or similar gages. This validation is used to develop and/or check regional curves for specific hydro-physiographic areas.

Although a bankfull discharge of 1.5Q is generally considered to be the typical return interval (Williams 1978), it is not at all uncommon to find ranges between 1.1 to 2.0 or higher. Data show that a return interval difference from a 1.1- to a 1.5-year event can have as much as 68 percent more flow (Southerland 2003). The bankfull dimensions associated with this difference in flow can likely lead to incorrect and/or inconsistent classification.

Entrenchment

Entrenchment is a measure of the extent of vertical containment of a channel relative to its adjacent flood plain. Entrenchment is defined as the ratio of the width of the flood-prone area to the bankfull width of the channel (fig. TS3E–4). The flood-prone width is measured at an elevation of two times the maximum depth at the bankfull stage. In figure TS3E–4, the flood-prone width is 305 feet, and the bankfull width is 25 feet. The entrenchment ratio is 12.2.

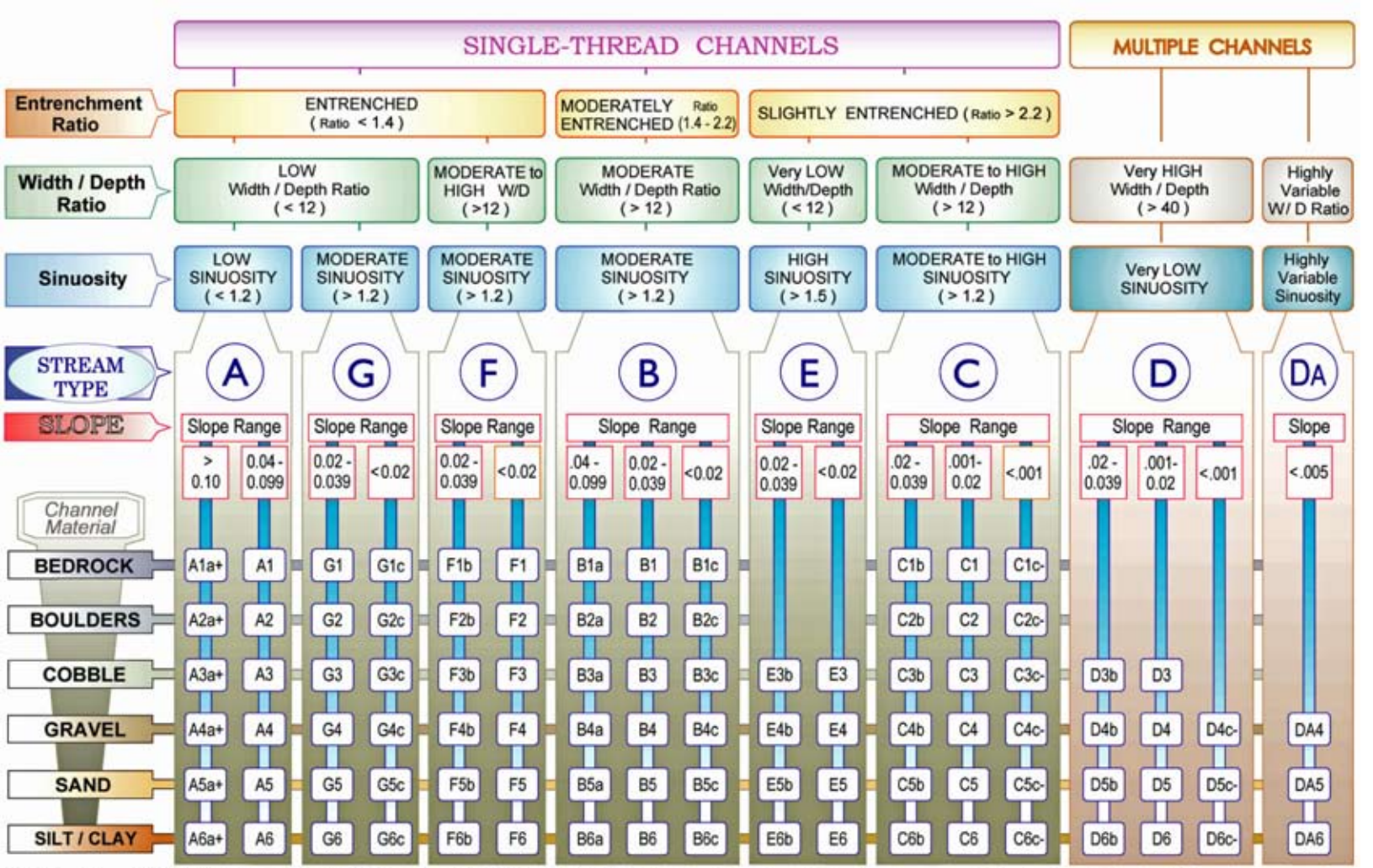
The top of banks does not always indicate bankfull stage. In deeply entrenched channels, the flood-prone area may be contained entirely within the banks. Entrenchment ratios for various stream types are shown on figure TS3E–5 (Rosgen 1996). The flood-prone area is measured at the riffle facet of the profile at level II classification. The entrenchment ratio may vary by 0.2 units without necessarily changing the classification.

Usually, field measurements will be necessary. In some cases, widths of flood-prone areas and bankfull stages can be made from aerial photos and topographic maps at the level I characterization. It is recommended that a typical cross section of each stream reach and its associated flood plain be obtained.

Channel material

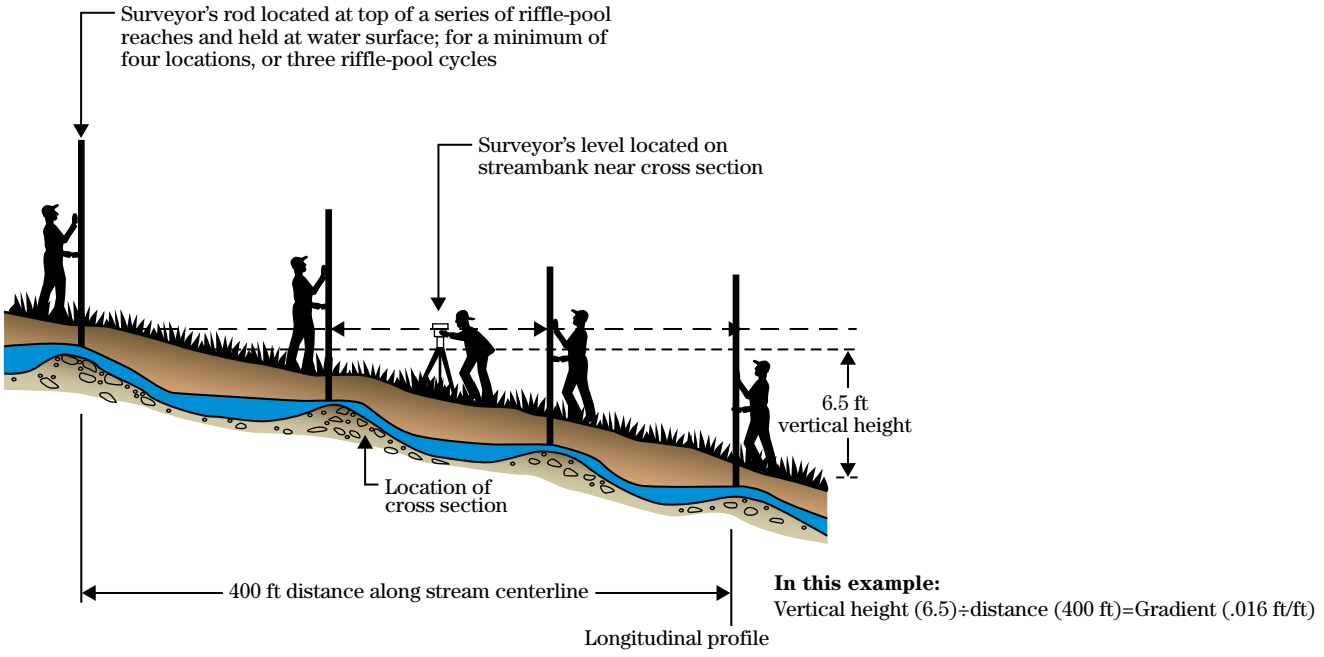
The channel material consists of the soil, rock, and vegetation that occur in the bed and banks of the channel. For classification, the dominant bed material is of primary interest. This consists of the sediment or rock

Figure TS3E–2 Hierarchical level II key



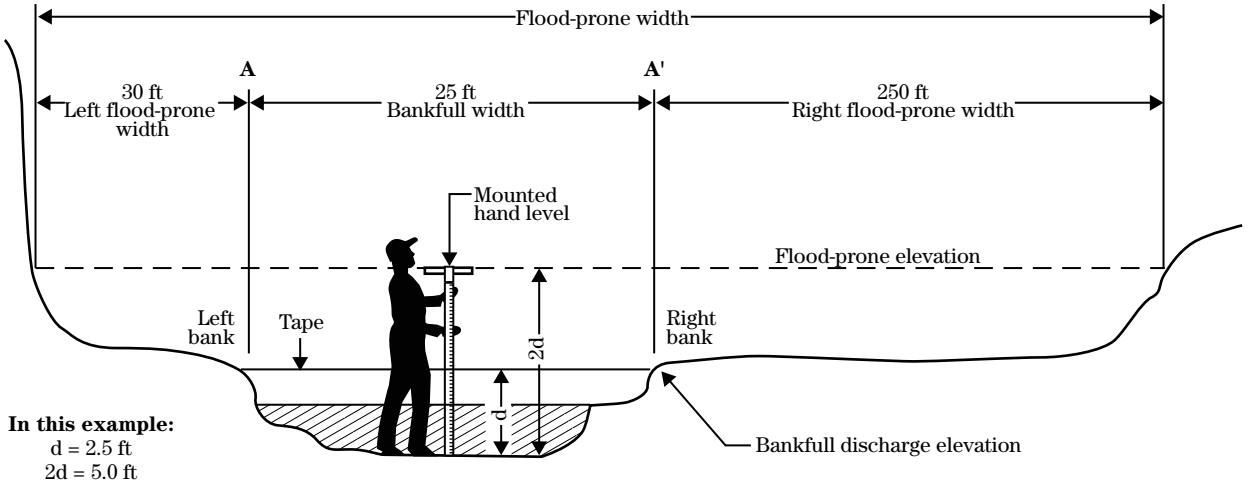
KEY to the ROSGEN CLASSIFICATION of NATURAL RIVERS. As a function of the *"continuum of physical variables"* within stream reaches, values of ***Entrenchment*** and ***Sinuosity*** ratios can vary by +/- 0.2 units; while values for ***Width / Depth*** ratios can vary by +/- 2.0 units.

Figure TS3E–3 Measuring stream gradient



Note: Riffle to riffle gradient approximates the average water surface slope

Figure TS3E–4 Field measurement of entrenchment ratio



Cross sections

Figures are not to scale

exposed in the bed and on about the lowermost third of the banks of the stream reach. The measure used is the median grain size, or D_{50}, of the bed material. Several sampling traverses may be necessary to represent the channel material in a given stream reach (fig. TS3E–6). Plant material, leaves, and so forth are not counted as bed-load material. When organic material is found, bed load at the particle size sampling interval (usually 1-ft intervals) should be pulled from beneath. If the debris is too large for bed load extrication below it, consistently draw particles at the same 1-foot interval from the side of the woody debris, while remaining on the same path. A detailed description of sediment sampling is provided in NEH654 TS13A.

Based on the D_{50} size, the channel material is classified into one of six particle size categories:

1 – bedrock (>2048 mm)

2 – boulder (256 mm to 2047.9 mm)

3 – cobble (64 mm to 255.9 mm)

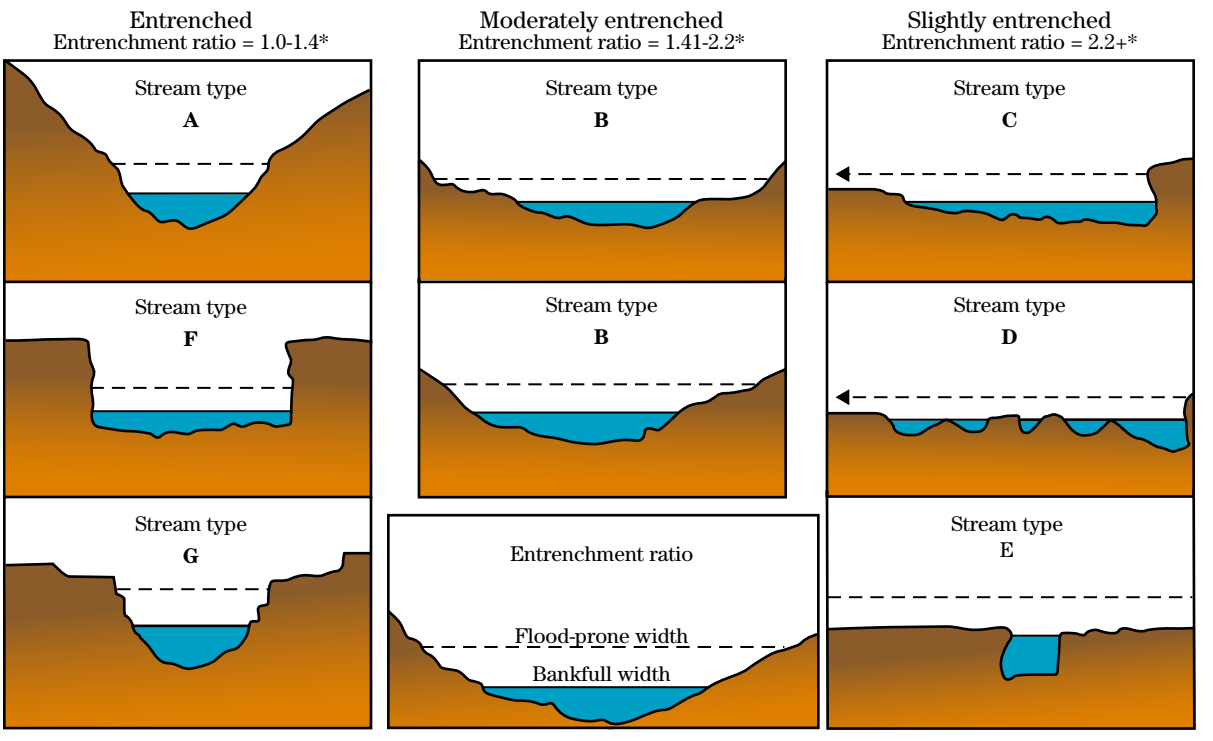
4 – gravel (2 mm to 63.9 mm)

5 – sand (0.062 mm to 1.99 mm)

6 – silt/clay (<0.062 mm)

Plot material sizes showing cumulative and percent distributions on log normal scaled paper. Estimate D_{50}. In figure TS3E–7, D_{50} is 34 millimeters (gravel size).

Figure TS3E–5 Entrenchment ratios of major stream types

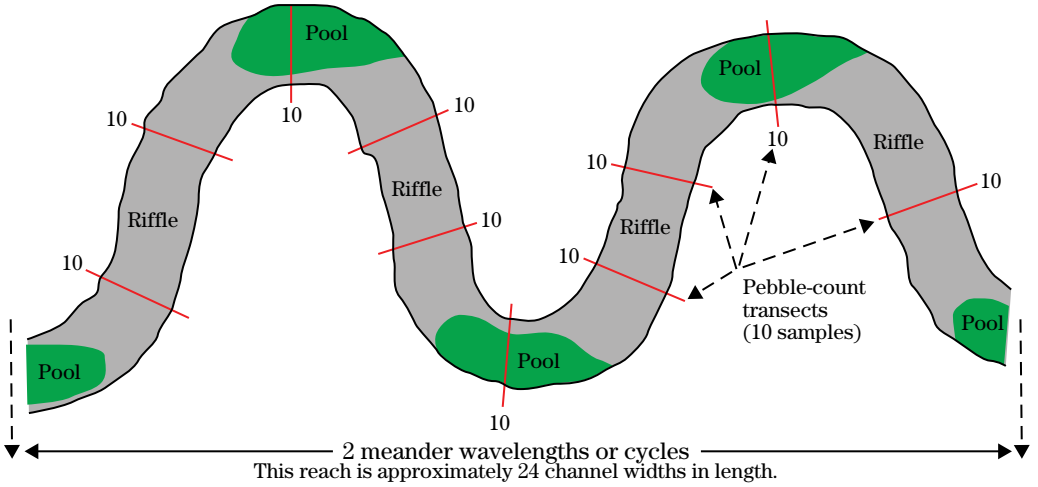


$$\text{Entrenchment ratio} = \frac{\text{Flood-prone width}}{\text{Bankfull width}}$$

*Entrenchment ratio may vary by ± 0.2 units

Flood-prone width = water level @ 2 × max/ depth

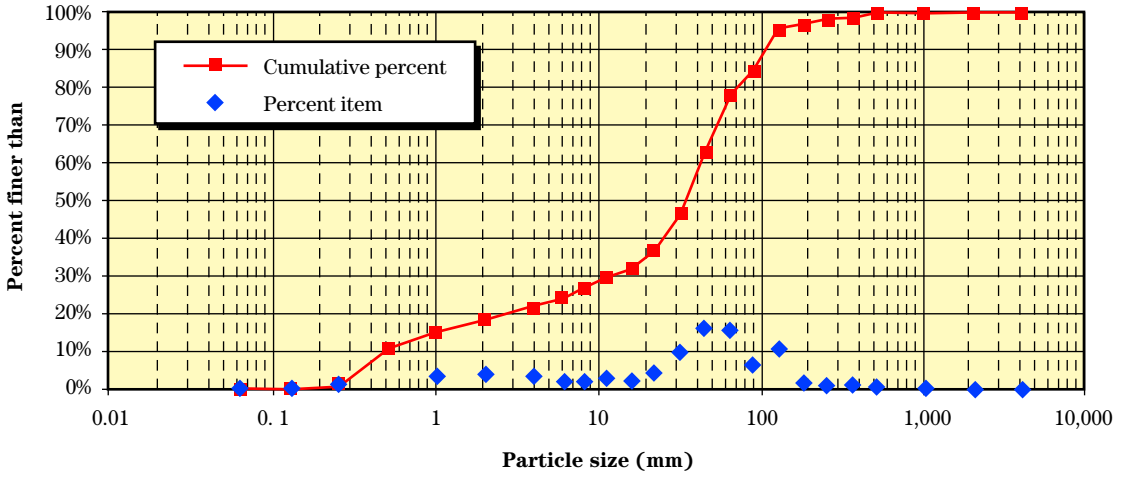
Figure TS3E–6 Pebble count



This reach is approximately 24 channel widths in length.

Example: Reach configured as:	Pool	Riffle	
	75 ft	175 ft	Total length: ± 250 ft
Percentage:	30%	70%	Samples for pebble count: 30% in pools, 70% in riffles

Figure TS3E–7 Plotted particle size distribution



Width-to-depth ratio

This consists of the ratio of the top width of the channel at bankfull stage to the average depth at bankfull. The average depth is computed by dividing the cross-sectional area of the channel by the width. The width divided by the depth (width-to-depth ratio) is typically the most sensitive indicator in level II classification. However, the width-to-depth ratio can vary by 2.0 units without necessarily changing the classification (Rosgen 1996).

It is recommended that a typical cross section of each stream reach be surveyed and appropriate measurements obtained from a graphical plot of the cross section. The bankfull stage can be estimated from field evidence or predicted using standard hydrologic techniques.

Sinuosity

This measure indicates the degree of meandering and channel migration within a valley that the channel exhibits in plan view. It consists of the ratio of channel length to valley length. The channel length of the stream reach is measured along the thalweg of the channel. The valley length is the length of the reach measured along a line paralleling the local trend of the stream valley.

Aerial photographs of appropriate resolution, soil maps, geologic maps, and USGS 7.5 quadrangles provide convenient means for measuring sinuosity.

Procedure

The simplified version of Rosgen's stream classification is implemented by applying the following procedure to each stream reach of interest.

Identify a reach of at least 20 bankfull widths. Define drainage area and use relative USGS gage data, if available.

Step 1 Identify plan view type and determine whether the channel type is multiple-threaded (three or more channels) at bankfull.

Step 2 Determine the entrenchment ratio.

Step 3 Find the width and average depth of bankfull event, and compute the width-to-depth ratio.

Step 4 Determine the channel slope (water surface).

Step 5 Using the data from steps 1 and 2, figure TS3E–1, and table TS3E–1, assign a capital letter designation representing stream type. If results are ambiguous, give more weight to plan view type than channel slope or entrenchment. This corresponds to the level I classification of Rosgen (1994).

Step 6 Determine D_{50} size of the dominant bed material, and classify in terms of size (sand, gravel).

Step 7 Assign a number from 1 to 6, depending on results of step 6. This corresponds to the level II classification of Rosgen (1994).

Step 8 Determine the channel and valley lengths of the stream reach and compute sinuosity.

Step 9 Check the values of channel slope, entrenchment ratio, width-to-depth ratio, and sinuosity against typical ranges for those values associated with the channel type (refer to fig. TS3E–2).

Step 10 If steps 4 and 9 are completed with satisfactory results, proceed with interpretations.

Step 11 If the stream reach fails to fit into a category in step 4 or if one or more values in step 9 lie outside the indicated ranges, additional studies are necessary. Anthropogenic alterations to some streams are so recent that the form may be in a transitory state and difficult to classify. However, with additional analysis a classification trend possibly may be identified. Table TS3E–4 (Harrelson, Rawlins, and Potyondy 1994) provides both the stream morphometry and landform features of the major stream types. Information such as this may aid in this further analysis

Interpretations and uses of the Rosgen stream classification system

The Rosgen stream classification system is intended as an evaluation tool. It conveys important information about the stability of the stream reach and about the degree of compatibility that certain types of stream

Table TS3E–4 Stream morphometry and landform

Stream type	General description	Entrench ratio	Width-to-depth ratio	Sinuosity	Slope	Landform/soils/features
Aa+	Very steep, deeply entrenched, debris transport streams	<1.4	<12	1.0 – 1.2	>.10	Very high relief. Erosional, bedrock, boulder, or depositional features; debris flow potential. Deeply entrenched streams. Vertical steps with deep scour pools; waterfalls
A	Steep, entrenched, cascading, step-pool streams. High energy/debris transport with depositional soils. Very stable if bedrock or boulder-dominated channel	<1.4	<12	1.0 – 1.2	.04–.10	High relief. Erosional bedrock forms. Entrenched and confined streams with cascading reaches. Frequently spaced, deep pools in associated step-pool bed morphology
B	Moderately entrenched, moderate gradient dominated channel, with infrequently spaced pools. Very stable plan and profile. Stable banks	1.4 – 2.2	>12	>1.2	.02–.039	Moderate relief, colluvial riffle deposition, and/or residual soils. Moderate entrenchment and width-to-depth ratio. Narrow, moderately sloping valleys. Rapids predominate with occasional pools
C	Low gradient, meandering point-bar, riffle-pool, alluvial channels with broad, well-defined flood plains	>2.2	>12	>1.4	<.02	Broad valleys w/terraces, in association with flood plains, alluvial soils. Slightly entrenched with well-defined meandering channel. Riffle-pool bed morphology
D	Braided channel with longitudinal and transverse bars. Very wide channel with eroding banks	N/a	>40	N/A	<.04	Broad valleys with alluvial and colluvial fans. Glacial debris and depositional features. Active lateral adjustment with abundance of sediment supply
DA	Anastomosing (multiple channels) narrow and deep with expansive well-vegetated flood plain and associated wetlands. Very gentle relief with highly variable sinuosity's, stable streambanks	>4.0	<40	Variable	<.005	Broad, low-gradient valleys with fine alluvium and/or lacustrine soils. Anastomosed (multiple channel) geologic control creating fine deposition with well-vegetated bars that are laterally stable with broad wetland flood plains. Stream type common in estuaries
E	Low gradient, meandering riffle-pool stream with low width-to-depth ratio and little deposition. Very efficient and stable. High meander width ratio	>2.2	<12	>1.5	<.02	Broad valley/meadows. Alluvial materials with flood plain and/or lacustrine soil. Highly sinuous with stable well-vegetated banks. Riffle-pool morphology with very low width-to-depth ratio
F	Entrenched meandering riffle-pool channel on low gradients with high width-to-depth ratio	<1.4	>12	>1.4	<.04	Entrenched in highly weathered material. Gentle gradients usually less than .02 ft/ft, but may range up to .04 ft/ft with a high width-to-depth ratio. Meandering, laterally unstable with high bank erosion rates. Riffle-pool morphology.
G	Entrenched gully step-pool and low width-to-depth ratio on moderate gradients	<1.4	<12	>1.2	.02–.039	Gully, step-pool morphology with moderate slopes and low width-to-depth ratio. Narrow valleys, or deeply incised in alluvial or colluvial materials (fans or deltas). Unstable with grade control problems and high bank erosion rates

management measures will have with the channel. The following section on stream management presents a few examples of how the classification can be used in stream assessment and restoration work.

Stream management

Each of the stream types delineated by level II classification has certain characteristics that indicate its sensitivity to changes. Table TS3E–5 (Rosgen 1996) summarizes the expected degree of sensitivity each stream type exhibits.

For example, type C streams have a meandering plan view with active point bars (fig. TS3E–1). A type C2 (table TS3E–5) exhibits low sensitivities to disturbance and erosion because of the coarse, boulder channel material. Type C5, however, is extremely sensitive to disturbance and erosion (table TS3E–5, col. 2 and 5) because the sandy channel materials are extremely susceptible to erosion. Column 6 indicates vegetation exerts a very high level of controlling influence on stability of C5 channels.

Channel evolution

If a stable channel is subjected to significant changes in its alignment, bank vegetation, or watershed land use, it is likely to become unstable. The channel system readjusts to a new level of equilibrium. The sequence of changes can be documented by applying the stream classification presented previously. In some cases, the type and magnitude of the changes can be predicted and management measures planned to prevent adverse responses. The sequence of changes may occur rapidly over a few years or more slowly, depending on the sensitivity of the stream and the magnitude of the imposed changes.

For example, figure TS3E–8 illustrates the sequence of changes in a particular stream (Rosgen 1996). Initially, the stream reach was a stable type E4. Extensive land use changes reduced the bank vegetation and increased the supply of sediment from the watershed. The channel responded to the imposed changes by increasing its width and gradient and decreasing sinuosity to form a C4 channel. As the gradient increased, the stream was able to attack its bed with more energy, eventually initiating a gully in the streambed (type G4). As the slope decreased within the tall confining banks, the channel migrated laterally which led to a degraded F4 type entrenched well below its original flood plain. The channel eventually reestablished a sinuous course at the lower elevation, returning to its initial E4 geomorphic stream type.

Planning stream restoration measures

Certain stream reaches have undesirable characteristics from an ecological point of view. These characteristics were often initiated by past land use and stream management practices. To restore the stream reach to a more desirable condition, it is necessary to know what suite of characteristics will be compatible with its new condition. The stream classification approach provides useful insight into this matter. If structural approaches to restoration are considered to be a viable alternative, understanding past, current, and future stream types will aid the user in developing the appropriate stable stream form and its respective bankfull dimensions.

Communication

Streams and rivers are complicated systems which are governed by complex and interdependent energy, form, and shape relationships. Classifying things into groups is a mechanism for creating order out of chaos (Goodwin 1999). The Rosgen stream classification provides such a needed communication tool for the existing condition of a stream. At level II, it stratifies data for the pattern, dimension, profile, bed materials, and entrenchment of the stream. It provides a shorthand description of morphological variables which are influenced or influence the energy use, behavior, and sensitivity of a stream. Channel classification and channel typing is particularly of use when stratifying data to develop hydraulic geometry relations and in the selection of a hydraulic geometry relations.

Prediction

The Rosgen stream classification at level II classifies the form of the stream. This classification system by itself only provides information about the existing pattern, dimension, profile, and bed materials. However, if it can be assumed that streams with the same general form also tend to have the same geomorphic processes, the classification can be used to predict typical

Table TS3E–5 Summary of delineative criteria for broad level classification

Stream type	Sensitivity to disturbance 1/	Recovery potential 2/	Sediment supply 3/	Streambank erosion potential influence 4/	Vegetation controlling
A1	Very low	Excellent	Very low	Very low	Negligible
A2	Very low	Excellent	Very low	Very low	Negligible
A3	Very high	Very poor	Very high	High	Negligible
A4	Extreme	Very poor	Very high	Very high	Negligible
A5	Extreme	Very poor	Very high	Very high	Negligible
A6	High	Poor	High	High	Negligible
B1	Very low	Excellent	Very low	Very low	Negligible
B2	Very low	Excellent	Very low	Very low	Negligible
B3	Low	Excellent	Low	Low	Moderate
B4	Moderate	Excellent	Moderate	Low	Moderate
B5	Moderate	Excellent	Moderate	Moderate	Moderate
B6	Moderate	Excellent	Moderate	Low	Moderate
C1	Low	Very good	Very low	Low	Moderate
C2	Low	Very good	Low	Low	Moderate
C3	Moderate	Good	Moderate	Moderate	Very high
C4	Very high	Good	High	Very high	Very high
C5	Very high	Fair	Very high	Very high	Very high
C6	Very high	Good	High	High	Very high
D3	Very high	Poor	Very high	Very high	Moderate
D4	Very high	Poor	Very high	Very high	Moderate
D5	Very high	Poor	Very high	Very high	Moderate
D6	High	Poor	High	High	Moderate
DA4	Moderate	Good	Very low	Low	Very high
DA5	Moderate	Good	Low	Low	Very high
DA6	Moderate	Good	Very low	Very low	Very high
E3	High	Good	Low	Moderate	Very high
E4	Very high	Good	Moderate	High	Very high
E5	Very high	Good	Moderate	High	Very high
E6	Very high	Good	Low	Moderate	Very high
F1	Low	Fair	Low	Moderate	Low
F2	Low	Fair	Moderate	Moderate	Low
F3	Moderate	Poor	Very high	Very high	Moderate
F4	Extreme	Poor	Very high	Very high	Moderate
F5	Very high	Poor	Very high	Very high	Moderate
F6	Very high	Fair	High	Very high	Moderate
G1	Low	Good	Low	Low	Low
G2	Moderate	Fair	Moderate	Moderate	Low
G3	Very high	Poor	Very high	Very high	High
G4	Extreme	Very poor	Very high	Very high	High
G5	Extreme	Very poor	Very high	Very high	High
G6	Very high	Poor	High	High	High

1/ Includes increases in streamflow magnitude and timing and/or sediment increases
2/ Assumes natural recovery once cause of instability is corrected
3/ Includes suspended and bed load from channel derived sources and/or from stream adjacent slopes
4/ Vegetation that influences width-to-depth ratio stability

stream processes, sensitivity, and behavior. However, this sort of assessment needs to be made within the context of the topographic setting, as well as the channel evolution and watershed history.

The Rosgen stream classification system, as with many classification systems, describes a static condition that is not necessarily related to a specific process or change and, therefore, does not provide a direct mechanism for predicting a new stable channel form in disturbed watersheds (Gillian 1996, Cherry, Wilcock, and Wolman 1996). In addition, due to the dependence of the classification upon the present morphological characteristics, the approach does not have the ability to take into account previous or anticipated hydrologic changes. The classification of a stream to a particular type does not, by itself, imply that a stream is stable or unstable. It only indicates that the stream pattern, dimension, profile, and bed material are within the specified limits and variances of the classification system.

Trends and dominant processes

The Rosgen stream classification can be used to assess general trends in stream behavior and also to provide a guide to the dominant processes that a stream system can experience. Table TS3E–6 summarizes the characteristics of the Rosgen stream types by watershed conditions.

It is important to recognize that the science of fluvial geomorphology is based primarily on observation. As a result, predicted trends and changes tend to represent average conditions. Assessment and design for a specific project area requires the use of physically based calculations (Goodwin 1999).

Figure TS3E–8 Evolutionary stages of channel adjustment

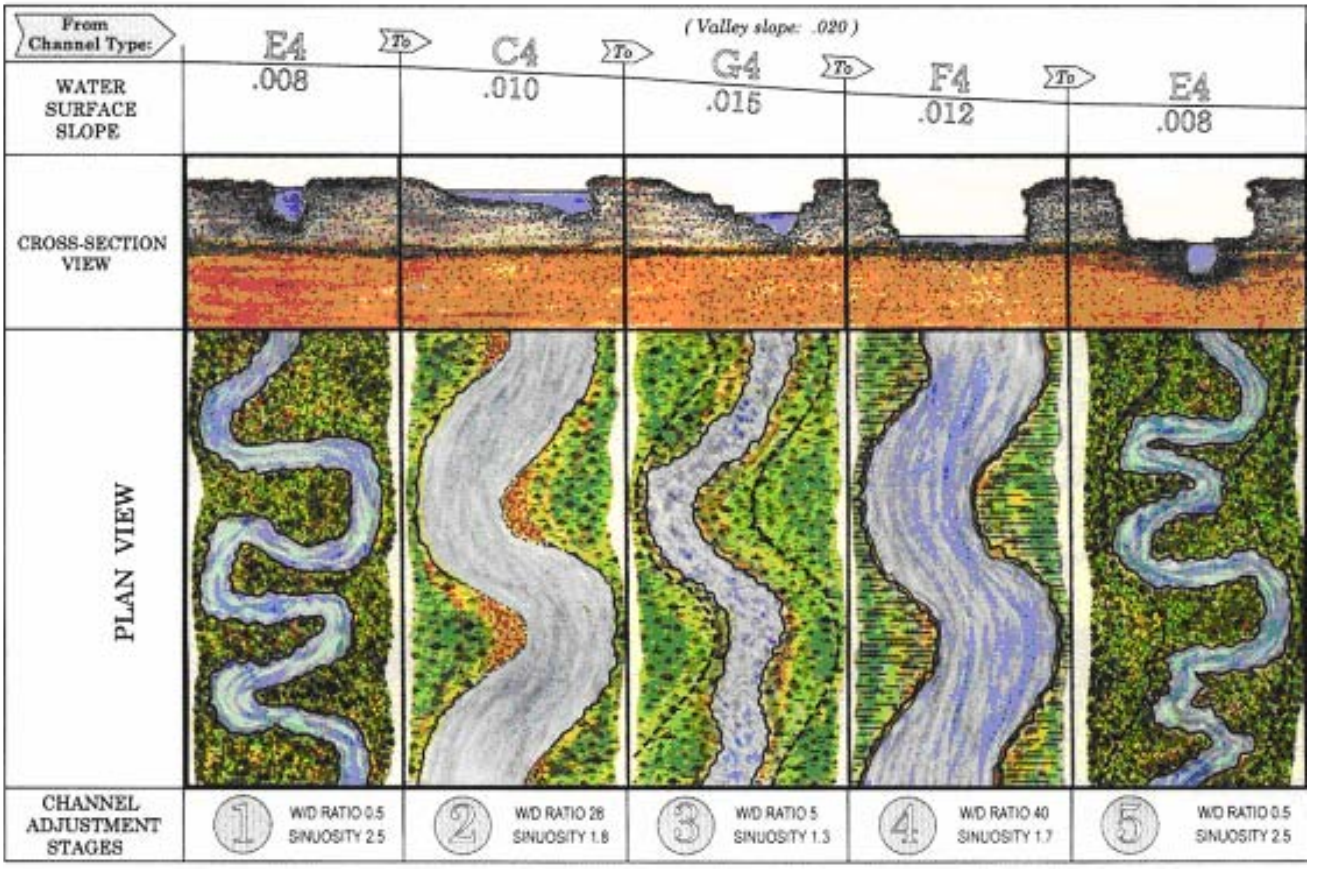


Table TS3E–6 Summary of characteristics of Rosgen stream types by watershed conditions

Rosgen stream type	Watershed type	Sediment load	Energy of stream	Energy dissipation in stream is typically by:	May be appropriate for design in:
A	Typically associated with steep, narrow mountain valleys. Bank vegetation is typically a low component of stability	High	High	Step pool	• Upper order urban streams (A2 and A3) • Grade control (A2)
B	Associated with narrow, gently sloping valleys. Bank vegetation is a moderate component of stability	Low to moderate	High	On banks and bed materials	• Urban streams (B2 and B3) • Grade control (B2 and B3) • Transition from flood plain to incised streams (B2, B3, B2c, and B3c) • Limited flood plain width (B and Bc) • Bottom incised streams (B and Bc)
C	Associated with broad, valleys with terraces and alluvial soils. Bank vegetation will typically have a high component of stability	High	Moderate	Through meanders, bedforms, and vegetation	Rural and urban streams with broad flood plains. However, these typically require bank protection and grade control during establishment of vegetation
D	Associated with broad valleys, glacial debris, and alluvial fans. Active lateral adjustment with abundant sediment supply. Vegetation will typically have limited influence on stability	High	Low to moderate	Banks and sediment	Normally not recommended
E	Often associated with broad valley meadows and well vegetated flood plains. Vegetation is typically a high component of stability	Very efficient at carrying sediment	Low	Through meanders, bedforms, and vegetation	Rural and urban streams with broad flood plains. However, these types may be difficult to construct due to low width-to-depth ratio and need for vegetation for stability especially on larger streams
F	Associated with modified channels and unstable channels	Low to very high	Low to moderate	Banks, vegetation, and sediment	Normally not recommended. These stream types can be laterally unstable with high bank erosion rates
G	Associated with narrow valleys or deeply incised in alluvial or colluvial materials such as fans or deltas	Low to very high	Moderate to high	Banks, vegetation, and sediment	Normally not recommended. These stream types can be laterally unstable with grade control problems and high bank erosion rates

Conclusion

Fluvial geomorphology techniques provide insight relative to general responses of a river system to a variety of imposed changes. These techniques are useful in analyzing the stability of the existing stream system and in identifying the source of instabilities.

The Rosgen stream classification system is based on the systematic collection and organization of field data by measuring combinations of morphological features. This system requires multiple measurements and calculations related to the pattern, dimension, profile, bed material, and entrenchment of a stream. It requires the assessment and characterization of valley types.

Some of the advantages of the Rosgen stream classification system (Rosgen 1996) are:

- communication—provides a common language for describing streams and their attributes
- standardization—encourages practitioners to measure things in a standard manner
- encourages thinking about stream processes
- provides a basis for generalizing and extrapolating data, knowledge, treatment strategies, and testing hypotheses about stream systems
- prediction—used to predict a river's behavior from its dimension, pattern and profile
- extrapolation—used to extrapolate data from a few sites or channels to a much larger number of channels over a broader geographic area
- defining a target—used to define the stable or desired form and to set targets or objectives for restoration or rehabilitation
- defining the scope of a problem—provides a means for quantifying the size of the problem and the type and size of the responses needed to address the major issues

While not all of these advantages are universally applicable or accepted by all practitioners, the Rosgen stream classification has been used as a tool to help understand how the stream form and processes are related, and it can be used to assist with stream evaluation, management, and design.

As stated by Craig Goodwin in Fluvial Classification: Neanderthal Necessity or Needless Normalcy (Goodwin 1999):

> *Classification should be considered only one part of a much larger scientific puzzle that also incorporates observation, laws, hypothesis, theories, and models.*

Since every stream system is unique, trends should only be considered to be general guidelines and a designer should note that there will always be exceptions.

Technical Supplement 5

Developing Regional Relationships for Bankfull Discharge Using Bankfull Indices

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Bankfull or channel-forming flow can be determined by field measurements, using indicators. Systematic measurement of bankfull conditions in a region can result in the development of regional curves that relate bankfull stream discharge and other geomorphic characteristics to watershed drainage area.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 5

Developing Regional Relationships for Bankfull Discharge Using Bankfull Indices

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Technical Supplement 5

Developing Regional Relationships for Bankfull Discharge Using Bankfull Indices

Purpose

This technical supplement presents a basic approach for the development of regional relationships for bankfull discharge using bankfull indices. This technical supplement provides guidelines to identifying bankfull stages along riparian stream corridors and procedures to determine the bankfull discharge associated with the bankfull stage. The bankfull discharge is used as a surrogate for the channel-forming discharge. While this technical supplement is primarily focused on the development of curves that are used in the Rosgen geomorphic channel design approach (NEH654.11), they are applicable to other assessment and design tools, as well.

Regional curves

Regional curves are constructed from observations and measurements of stable riffle cross sections on gaged rivers and streams. They are empirical by nature. The measured bankfull data are plotted versus the contributing drainage area flowing through the measured cross section(s). The regression equations express mathematical relationships between the bankfull channel dimensions: cross-sectional area, top width, mean depth, and the contributing drainage area.

Regional curves are a useful planning tool for natural stream design, stream restoration/stream enhancement, and fish habitat improvement or enhancement projects. They may provide estimations of the bankfull channel dimensions and bankfull discharge for any ungaged river or stream within the same physiographic area, given its drainage area.

However, discharge, not drainage area, is the driving force that moves, shapes, and maintains channels. Watershed shape, drainage pattern, slope, vegetal cover, land use, and management practices all affect the timing and magnitudes of runoff and, therefore, affect the size of the bankfull channels. Mathematically, better correlations exist between the bankfull hydraulic geometry and bankfull discharge. In watersheds with similar drainage areas, magnitude and duration of bankfull discharges can vary and, hence, hydraulic geometries at bankfull stage will vary due to the shape and cover of the watershed. Regime curves (hydraulic geometry vs. effective discharge) improve correlation over regional curves. Hydraulic geometry is described in further detail in NEH654.07 and NEH654.09.

USGS gage station selection criteria

Regional curves are constructed from stream survey measurements at U.S. Geological Survey (USGS) gaging stations. The period of record should accurately reflect the expected land uses and reported drainage size of the watershed. Gage data should be collected to represent a wide array of drainage areas with similar selection criteria.

Selection criteria for USGS gages to be used for regional curves are based on several factors: length of record, channel and drainage network stability, land use (rural vs. urban), drainage area size and shape, and the degree of (flood) control within the watershed. These criteria should match for the gages used to develop the regional curves. A combination of factors must be avoided. In addition, the criteria for the gages used to develop the curve should also match the intended project or evaluation area. Additional issues are described.

Stability of channel and drainage area network

The period of record should accurately reflect the expected land uses and reported drainage size of the watershed for the entire period of record. It may be difficult to recognize a natural drainage system that has undergone some degree of land use change or instability in its past. When researching gages, look for clues which may indicate that the watershed's hydrology and sediment production has changed over the length of record. Such clues may be obtained from a series of historic aerial photos, land use maps, mining records, road development, grazing or farm practices, development patterns, or fire records. These practices can affect the timing and volume of runoff, as well as the sediment production in a given watershed.

Gages on streams with flood control or diversions

Bankfull channel dimensions normally correspond to the uncontrolled drainage area. The assumption behind taking measurements of streams at known drainage areas and discharges is that a link can be made between the stream's geometric parameters and peak rates of discharge with uncontrolled drainage areas. Dams, detention basins, and diversions affect the watershed hydrology by storing part or all of the peak discharges and releasing an attenuated flow regime. This will also affect sediment transport, which is critical to channel formation and maintenance. Dams and storage basins may store all incoming bed-material load and only a small portion of the finer wash load. Flood control affects the timing of peak discharges and may prevent the normal bankfull event from occurring downstream of the impoundment. Therefore, it is generally recommended to avoid gaging stations in watersheds that have flood control and water diversions.

Urban versus rural land use drainage areas

Urbanization generally increases the amount of impervious surface in a watershed. Additionally, stormwater and sewer conveyance systems combine to increase the volume of runoff and magnitude of runoff for a given storm event. Urbanization of a drainage area tends to reduce the recurrence interval for the bankfull event; from say, a 1.5-year recurrence interval to a 1.2-year recurrence interval. Urbanization may also increase flow velocities, increasing the forces and stresses imposed on the beds and banks of the channels. This is brought about in a number of ways such as changes in alignment (meander patterns, cutoffs, and increased stream gradient), encroachment on the flood plain, reduction in the boundary roughness, and changes in the median bed-material particle sizes. Urban areas are unique and should be separated from natural drainage areas to account for these changes in hydrology and sediment transport regimes. For example, regional curves may be constructed separately to represent natural forested and/or rangeland areas, rural farmed areas, or urban areas.

Equipment and human resources

Creating regional curves requires a team for collecting, analyzing, extracting, and transforming data into information. The size of the survey crew varies, depending on the site and intended use of the curves.

Surveys can be conducted with any standard survey instruments including a theodolite, total station, automatic level, or a laser level. The following equipment is usually needed to conduct these surveys:

- stable tripod
- telescoping rods, prisms
- two-way radios
- field notebook(s)
- compass
- measuring tape
- camera
- waders
- flagging
- station pins and nails
- orange vests
- personal flotation devices
- ruler (in millimeters, for rocks)
- data collection sheets

Other items may include a Global Positioning System (GPS) unit for precise locations, range finder to expedite the surveying process, buckets and shovel for sampling bed and bank materials, set of sieves for determining grain size, and scale for weighing samples.

In-office data collection

Gage data

The USGS Web site (*http://waterdata.usgs.gov/nwis/rt*) contains information such as station name and number, latitude and longitude of the gage site, drainage area, period of record, number of years of record,

and peak annual discharges and corresponding peak gage heights for each year in the period of record (n years). An estimate of the 1.5-year discharge is derived using techniques described in NEH654.05 and is used as a surrogate for bankfull discharge recurrence interval.

For the gages of interest, the practitioner should contact the local USGS data chief and request station descriptions, current rating tables, and summaries of discharge measurement notes. However, it is important to note that while each gage station may have a unique rating curve, the relationship between gage height and discharge is not necessarily unique. The rating curve may shift over the long term, as the cross-sectional shape and/or elevation changes, and it may shift over the course of a hydrograph due to the unsteady loop effect or due to changing bedforms.

The user should collect an existing gage analysis, or information sufficient to conduct such an analysis, following the procedures described in NEH654.05. By cross-referencing the estimated flows with the rating table, the user can define specific gage heights as the elevations for specific return intervals or specific chances of exceedance.

Discharge measurement notes are useful in that they provide specific cross-sectional flow areas, top widths of flow, and velocities for specific gage heights and discharge measurements. Plotting measurements below and above the bankfull discharge allows the user to estimate flow area, top width, and velocity at the bankfull or channel-forming flow.

Aerial photographs, topographic maps, and geology maps of the watershed of interest should be examined. These maps and photos can reveal details of the watershed and land use patterns that indicate the conditions that help shape the drainage network.

The topographic maps will also provide information required for stream characterization that is not easily obtained from the ground survey. Reach slope, sinuosity, and meander belt width can be estimated from these maps. Average reach slope can be estimated from the topographic maps by measuring the planimetric distance between contour intervals. It is recommended that the practitioner identify two to four consecutive contour intervals both downstream and upstream from the gage and measure the streamwise distance between the contour intervals.

A cross-reference to the USGS rating curve will provide the gage height for the 1.5-year discharge. When at the site, the user should locate the staff gage and identify a relatively flat depositional feature above or below this gage height corresponding to the bankfull discharge. If the staff gage has been removed, the user should locate an existing reference mark (from the station description) that refers back to a gage elevation. With a measuring tape, measure up or down to the gage height corresponding to the 1.5-year discharge, and again, look for the first flat depositional feature around this elevation. The practitioner should study this feature and the corresponding material size. Of particular interest are moss lines, debris lines, changes in slope and other distinguishing features. The elevation of these features relative to the water surface may be useful in identifying bankfull stages away from the gage site.

Note that the bankfull discharge elevation may vary significantly from the 1.5-year recurrence interval that is a normal surrogate for bankfull discharge in natural streams. As stated previously, the recurrence interval for the bankfull flow may be more frequent in developed watersheds.

Use of discharge notes

Discharge measurement notes can also provide insight into the hydraulic characteristics of the stream. Discharge measurement notes are a summary of discharge measurements taken throughout the period of record. They include date of measurement, gage height, discharge, top width of water in the cross section, cross-sectional area of flow, and mean velocity in the measurement cross section. The location where measurements take place is usually described in the station description. It is common to have two cross-sectional locations—one on the control feature of the stream for low-flow measurements and one across the bridge for high flows.

Energy slope and Manning's n are not included in the measurement notes. After calculating an average reach slope from topographic maps, Manning's n can be calculated using Manning's equation by approximating the hydraulic radius by the hydraulic depth d = flow

area/top width from the discharge measurement notes for a given discharge. Manning's n can vary considerably with depth of flow. Streams characteristically have high roughness at low flows and become hydraulically smoother as depth of flow increases. It is also important to note that this is a normal depth assumption and may not represent the flow levels due to any backwater effects that may occur. More information on the normal depth assumptions and computer modeling approaches is provided in NEH654.06.

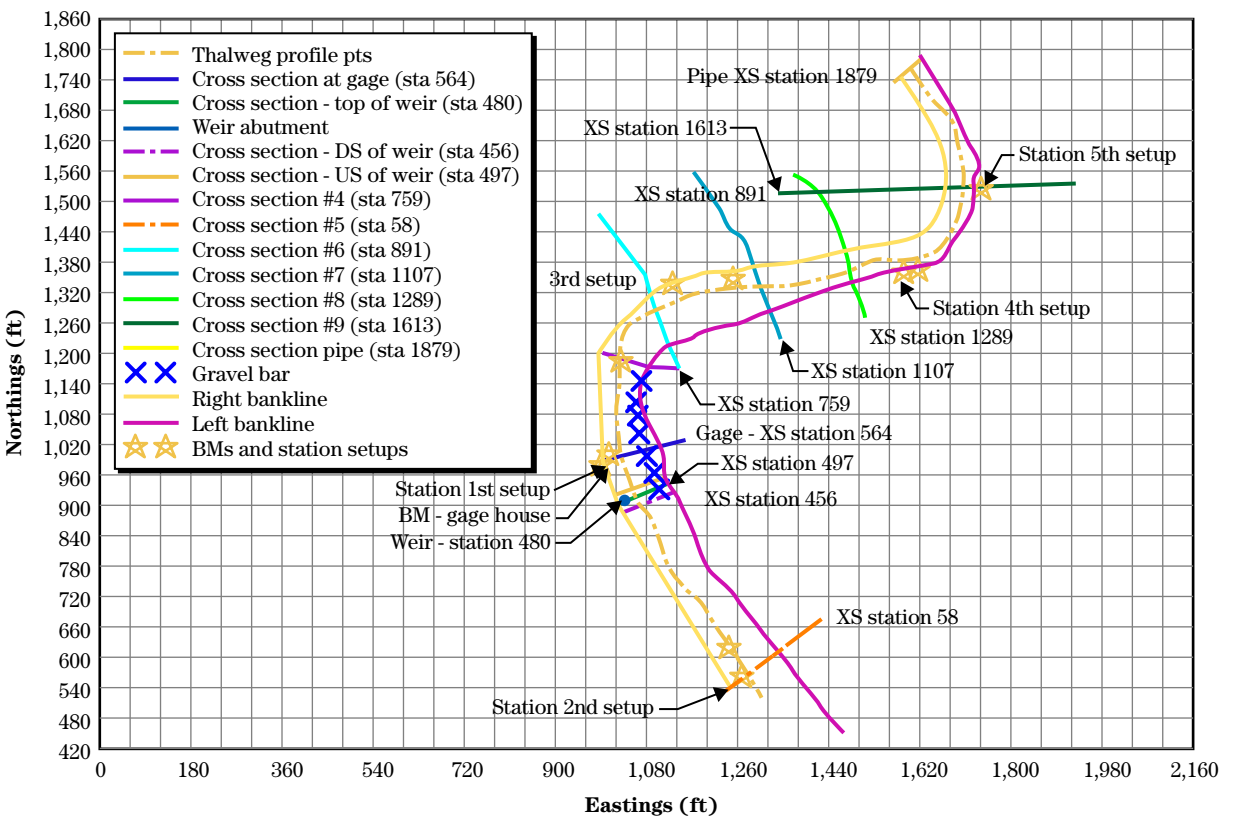
Site reconnaissance

Before setting up the surveying equipment, a reconnaissance along the reach is prudent to select optimum station setups and minimize the overall number of setups and turns. During this reconnaissance, team members should assess the reach to determine if it is a stable form of the river, as it would have developed under natural conditions. The team can make this assessment by asking the following questions:

- Is there a low water ford or cattle access present that changes the channel geometry?
- Is accelerated bank erosion occurring?
- Are there undercut banks and trees falling in?
- Has bank vegetation been grazed, removed, sprayed, or cleared away?
- Is there one long continuous pool upstream from the gage?

The team should also assess the location of sections to be surveyed. Identification of riffle locations or the heads of glides, selection of cross-sectional locations, flagging bankfull indicators, and deciding the length of the reach to survey prior to setup may actually save field time. Figure TS5–1 provides an example of a sur-

Figure TS5–1 Example plan view of a site



vey using a total station survey instrument and shows station setups, benchmarks, thalweg profile, bank lines, cross sections, instream weir, and pipe crossing.

Station setups, benchmarks, and reference marks

The first step in beginning a survey is to tie survey elevations into the gage datum using the USGS reference marks. From the USGS station gaging description, the team should find all existing reference marks. These are published elevations with respect to the gage datum and allow the survey to be tied to an official datum. These marks may be chiseled Xs or chiseled squares on bridge abutments, gage houses, elevations of check bars on an outside wire weight gage, USGS or U.S. Department of Transportation (DOT) brass caps, staff gages, or bolts in trees or telephone poles. A shovel may be needed to scrape away dirt and overgrown weeds over concrete surfaces. The team should assure that at least two reference marks are visible from the initial station setup.

When using a total station instrument, the resection method for determining the station location requires coordinates for two known elevations. A measuring tape, compass, and calculator will be required to determine these coordinates in northings and eastings. The coordinate system may be arbitrary on setup, but afterwards, it must remain consistent, or the true alignment will be lost. Figure TS5–2 shows a station setup just upstream of a gage house along a riffle section where USGS discharge measurements are conducted.

Cross sections

Estimating a bankfull discharge may be accomplished by surveying a single section that is upstream of the gage and correlating it to the gage rating curve. However, for regional curve development, several cross sections for two to three full meander wavelengths for a detailed HEC–RAS model is recommended. Since the profile of the river reach will vary between the relatively steep riffle sections and the long relatively flat pool sections, the use of the HEC–RAS model will allow the practitioner to reconstruct the bankfull water surface elevations along the survey reach back to the gage site and, ultimately, prepare the rating table to determine discharges.

The survey data are to be used to develop a HEC–RAS model, so a cross section that represents the rating table is required. This cross section will be very important in calibrating the model. The station description usually describes where in the reach (in relation to the gage) low-flow discharges are measured. More than likely, this is on a riffle or upstream in a pool from a manmade control point, such as a cross-channel weir. Surveying a cross section over the end of the pressure transducer pipe is also wise, for this may be the section that represents the USGS rating table. Several cross sections should be surveyed downstream from the gage. The furthest one from the gage must be sufficiently far enough downstream that any erroneous assumptions of starting the flow conditions at normal depth are negligible at the gaging cross section. A good location for the first cross section may be in the next downstream riffle section, usually six to eight bankfull widths downstream.

The practitioner should survey several cross sections in the middle of three to four riffle sections above the gage cross section. This will help assure that the average reach geometry is not dependent on just one or two cross sections. All cross sections should start at or above the 100-year flood plain, or high on the valley wall, and extend across the valley to the opposite valley wall, or end above the 100-year flood plain. It

Figure TS5–2 Station setup is just upstream of gage house

is normal protocol to define a cross section looking downstream with the stationing (in the cross section) increasing from left to right.

Figure TS5–3 shows a cross-sectional view near the gage house shown in figure TS5–2. The HEC–RAS computed rating curve at this cross section was compared to the USGS rating curve to complete calibration. Note in the cross-sectional view that the bankfull elevation corresponds to the top of a gravel bar feature near the left bank.

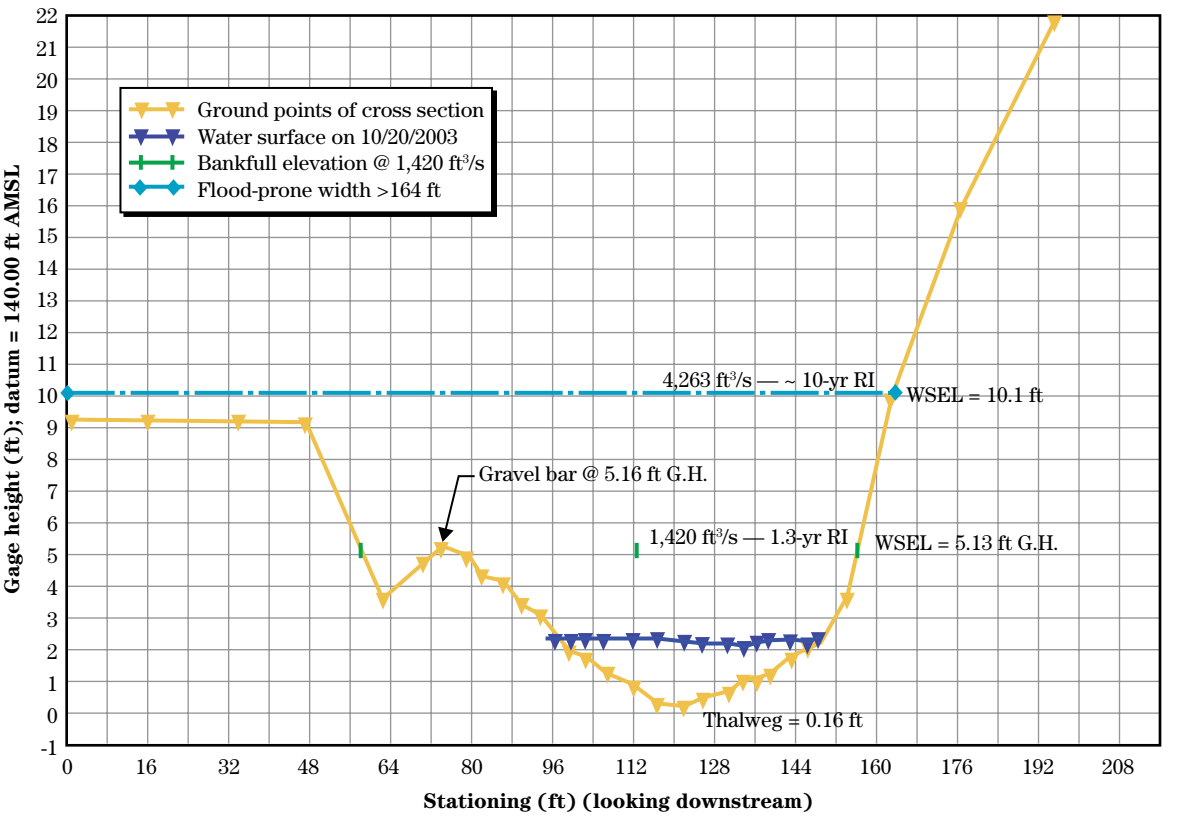
Profile—channel bed, water surface, bankline, flood plain, and terraces

Between cross sections, the survey should locate the thalweg profile, water depth, bankline profiles, and flat depositional features adjacent to the stream, known as the active flood plain. With a four-person team, one person operates the instrument with three people each with a survey rod; one along the right bank, one along the thalweg, and one along the left bank. This technique lends itself well to defining bankfull elevations because there will be at least two opinions on bankfull features. Every shot of the survey should include a recorded description of the particle size of the bed material that is found under the survey rod.

Bankfull indicators

Bankfull flow elevations and discharges are associated with sediment transport and, therefore, are closely tied to particle sizes moved and deposited in gravel and cobble dominated bed streams. In sand-bed streams, there may not be a differentiation of particle sizes from the channel and the active flood plain, but there

Figure TS5–3 Cross-sectional survey



should be a break in slope. Flat depositional features, breaks in slope, height of point bars, and vegetation features are other bankfull indicators that should be used. One of many bankfull indicators is a change in particle size distribution from gravels to fine grained sands. More information on bankfull indicators is provided in NEH654.05.

Characterization of bed material

The typical technique used for sampling the bed material is the Wolman pebble count. Wolman pebble counts are conducted in the riffle sections for several purposes and are described in more detail in NEH654 TS13A.

Data processing and analysis

HEC–RAS model input

The cross-sectional data are used to build a conventional HEC–RAS hydraulic model. It is recommended to use the thalweg stationing to set the channel distances between cross sections (required input to HEC–RAS model). All water surface elevations generated by the model will be in reference to the channel distances, which may be different from the bankline distances.

Calibrating to USGS rating curves

After the initial input of cross-sectional data, a HEC–RAS computational model run can be made to determine if the model has sufficient cross-sectional data to compute the actual water surface elevations recorded along the reach measured during the day of survey. Plotting computed water surface elevations along with channel bed and measured water surface elevations is helpful in pointing out areas along the profile that could use refinement or more definition. Depending on the level of agreement, additional refinement may be done by either returning to the field to take more measurements or by adding in interpolated cross sections based on the thalweg profile. It should be noted that this approach may be problematic in streams where the flows were very low at the time of the survey.

When the model definition is robust enough to match measured low-water surface elevations, calibration of the model by changing Manning's coefficients and contraction/expansion coefficients can proceed to match the USGS rating curve at the gaging cross section. Figure TS5–4 shows a comparison of rating curves between the calibrated model results at the gage cross section. As shown in this figure, the model calibration is good up to discharges of 4,000 cubic feet per second, which is well beyond the bankfull discharge of 1,420 cubic feet per second.

Selecting the channel-forming discharge

Once the model is calibrated to the USGS rating curve, a selection of the channel-forming discharge can be made. This will entail running a range of discharges in the HEC–RAS model and comparing computed water surface elevations along the longitudinal profile to measured bankfull indicators and associated bankfull elevations. The criterion for consistency is that the profile of bankfull-stage elevations should plot approximately parallel to the longitudinal profile of the water surface at some given discharge through the reach (Kilpatrick and Barnes 1964). The channel-forming discharge is the discharge that comes closest to the surveyed bankfull indicators: flood plains, benches, breaks in slope, change in particle sizes, and vegetation indicators along the reach.

Hydraulic geometry relationships at bankfull

Once the channel-forming discharge or bankfull discharge is known and the corresponding water surface elevations computed, the hydraulic geometry in the stable riffle cross sections can be estimated. Cross-sectional flow area, hydraulic radius, hydraulic depth, and top width can be selected as output variables from the HEC–RAS Profile Output Table. The hydraulic geometry for the reach is best represented by an average of three or four stable riffle cross sections. The hydraulic geometry relationships at bankfull should then be plotted with respect to drainage area on the regional curve (fig. TS5–5). These relationships are useful in a variety of channel assessment and design applications.

Figure TS5–4 Comparison of USGS rating curve with HEC–RAS

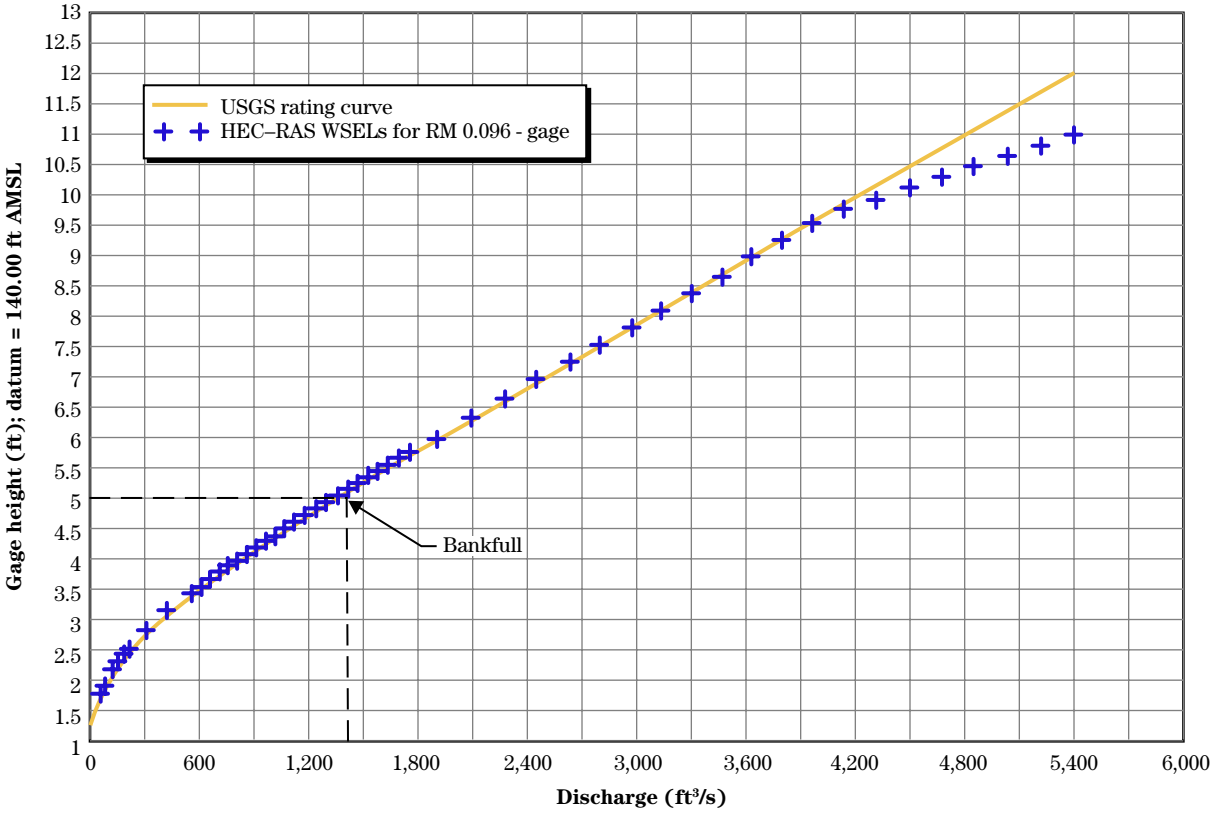
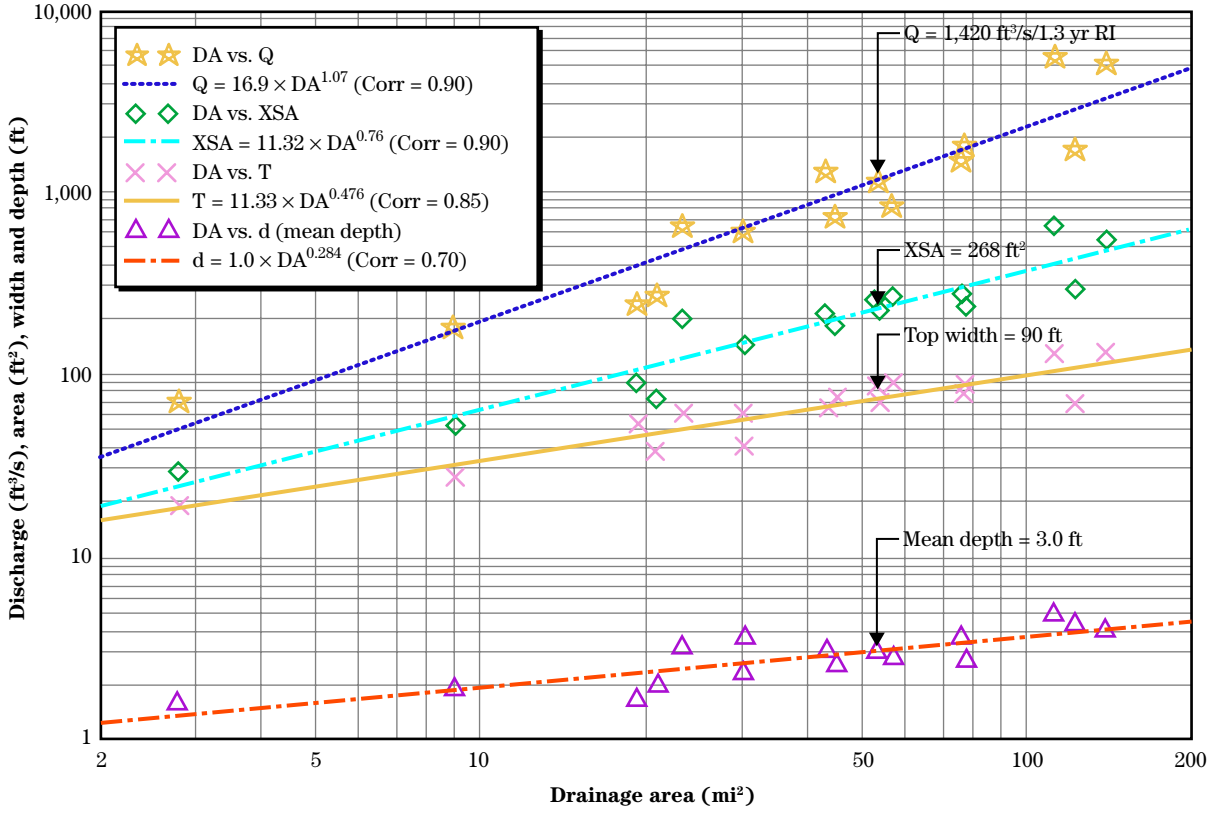


Figure TS5–5 Regional curves for hydraulic geometry



Technical Supplement 13A

Guidelines for Sampling Bed Material

(210–VI–NEH, August 2007)

Issued August 2007

Cover photos: *Top*—Bed-load sediment may be poorly graded or well graded. Armoring layers may also be present.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 13A

Guidelines for Sampling Bed Material

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Technical Supplement 13A

Guidelines for Sampling Bed Material

Purpose of bed-material sampling

The characteristics of a given stream are linked to the composition of the material that comprises its channel bed, bank, and sediment flow. Knowledge of streambed material is necessary for a variety of engineering and environmental purposes. The size and gradation of the streambed material may affect the source, transport, and fate of pollutants; fish habitat; resource management; morphological trends; and stream restorations.

Bed-material sampling programs must be carefully designed to meet the particular needs of a specific study. Studies may include objectives related to the following:

- Contaminants—Typically attach to cohesive sediment and, therefore, are distributed over a wide area, especially in areas where flow velocity is low. Sampling for a contaminant concentrates on depositional zones in the stream and overbank.
- Aquatic habitat—Fish habitat studies may focus on the suitability of the streambed for spawning. Sampling for this type of study is often extensive, identifying lateral, longitudinal, and temporal variations in the surface layer over a wide area of the stream. An assessment of vertical variations may also be of critical importance, as the composition of the material immediately below the surface, especially the fines content, may be of importance in the evaluation of spawning habitats for some species.
- Gravel mining—Resource management studies are frequently concerned with the need or feasibility of sand and gravel mining. Core or substrate sampling that identifies vertical variation of the streambed is essential for this type of study.
- Stream assessment and design—Morphologic and engineering studies are concerned with changes in the character of the river over time. These studies require knowledge of the grain size distribution of both the bed surface material and subsurface material for sediment transport calculations, critical shear stress determinations, determining potential for particle sorting and armoring, and determining hydraulic roughness.

Complex studies may need to secure data to meet a combination of objectives and purposes. However, sediment data collected for one purpose will not necessarily be applicable for another. While the issues and recommendations presented here are generally applicable, the focus is on bed sampling for stream assessment and design.

Site selection for representative sampling

Sufficient sampling of the streambed should be conducted to determine the spatial variability, size, and gradation of the bed material. No simple rule exists for locating representative sampling sites or reaches. The general rule is to carefully select sampling locations and avoid anomalies that would bias either the calculated sediment discharge or the calculated bed stability. Sampling locations must be representative of the hydraulic and sedimentation processes that occur in that reach of the river. The site should be morphologically stable. To ensure data reflect reach-averaged river conditions, there should be no tributary inflow in the proximity of the site, as it may interfere with the homogeneity of the section by supplying sediment for deposition. The site should not be located adjacent to a zone of active bank erosion, as the material deposited in the channel near the eroding area may not be representative of the reach. Although bridges provide good access, bridge crossings are typically not appropriate sampling sites because either they are located at natural river constrictions or their abutments and piers create constrictions and local scour. Dead-water areas behind sand bars or other obstructions should be avoided, as these are not representative of average flow conditions.

The location of the bed sample should be chosen with the target analysis in mind. Table TS13A–1 provides guidance for where a bed-material sample might be taken as a function of the type of geomorphologic or engineering analysis to be conducted. This list is not inclusive, exhaustive, or absolute. Ideally, bed-material

samples should be taken at different times during the year to account for seasonal variations.

Sand-bed streams

Sand-bed streams have relatively homogeneous bed-material gradation. Vertical and temporal variability are normally insignificant in stable sand-bed streams. Longitudinal variability typically occurs over distances of many kilometers. However, lateral variability, especially in bends, can be significant. In sand-bed rivers, sampling of bed material is most frequently done in the low-flow channel. The sampling equipment and methodology used depend on the river depth and velocity. The task can be accomplished in flowing streams either by wading or from a boat or in ephemeral and intermittent streams in the dry. Vertical variations in the bed material are usually insignificant in flowing water, and samples are collected from the surface. However, in standing water or on dry beds, a layer of fine material is sometimes found deposited on the bed surface during the recessional part of a flood hydrograph. It is standard practice to remove this fine surface layer before collecting a bed-material sample in this location.

Einstein (1950) recommended using only the coarsest 90 percent of the sampled bed gradation for computations of bed-material load. He reasoned that the finest 10 percent of sediment on the bed was either material trapped in the interstices of the deposit or a lag deposit from the recession of the hydrograph and should not be included in bed-material load computations.

Representative bed-material sampling in sand-bed streams may be accomplished by one of two methods. Employing the cross-sectional approach requires selecting a site and time for sampling where and when the bed characteristics are typical. This method requires considerable experience. Unanimity of opinion about where and when the typical condition occurs cannot be expected, even among experienced river scientists. Frequently, judgment is influenced by the type of streams the sampler has experienced and by the intended use of the data. Employing the reach approach, where samples from several systematically selected cross sections are averaged to obtain a representative sample, may eliminate some uncertainty associated with the cross-sectional approach.

Cross-sectional approach

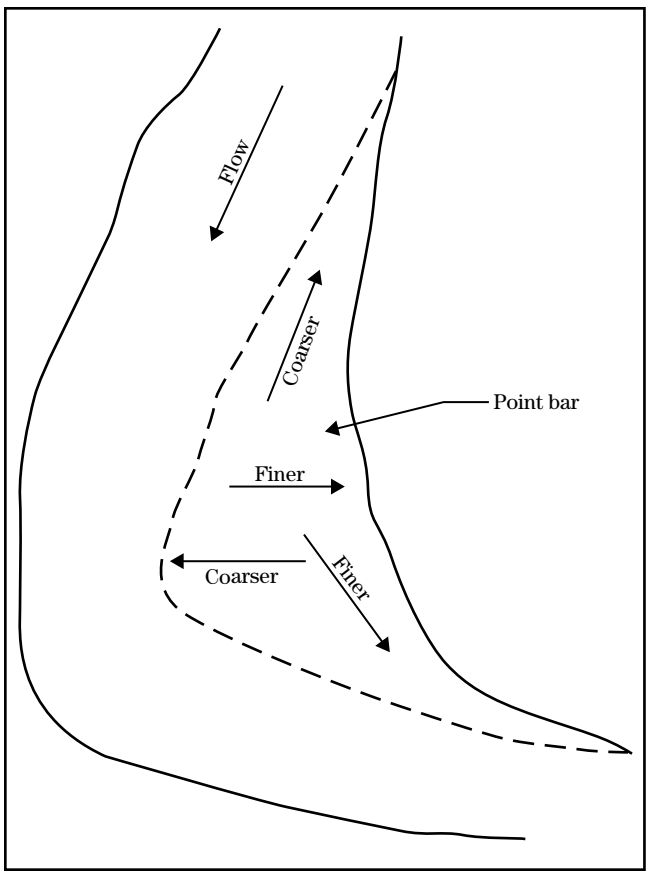
This approach requires the selection of a representative cross section for a reach. In streams with relatively uniform depths, between three and five samples should be taken across the section to account for lateral variations. In streams with variable depths, more samples are required. Twenty verticals are commonly taken along the cross section in braided streams. Taking bed-material samples at crossings where flow distribution is more uniform reduces the lateral variation in the samples. However, at low flow, crossings may develop a surface layer gradation that reflects sediment transport conditions at the lower discharge, which may be coarser or finer than the bed gradation at bankfull discharge. Also, crossings are typically submerged, and more elaborate sampling equipment is required than at exposed bars, where a

Table TS13A–1 Bed-material sampling sites

Purpose of analysis	Sample location
To estimate the maximum permissible velocity in a threshold stream	Riffle
To estimate the minimum permissible velocity in a threshold stream	Areas of local deposition
To estimate sediment yield for an alluvial stream	Crossing or middle bar
To quantify general physical habitat substrate condition	Bars, riffles, and pools

shovel is usually a sufficient sampling tool. However, samples collected on a point bar or alternate bar may exhibit considerable variation. Figure TS13A–1 illustrates typical bed-material gradation patterns on a point bar. Note that although the typical grain sizes found on the bar surface form a pattern from coarse to fine, no single sampling location always captures the precise distribution that represents the entire range of sedimentation processes.

Figure TS13A–1 Gradation pattern on a point bar



Reach approach

An alternative to the cross-sectional approach is the reach approach. A reach is defined as a portion of the stream with similar morphology (identified by its homogeneity). Generally, five cross sections are laid out in the homogeneous reach. If there is a gage in the reach, locating the center cross section near the gage is preferred. This facilitates relating the sediment data to measured hydrologic and hydraulic data. If the stream reach is straight, the spacing of the cross sections should be approximately two to five stream widths, and if the reach is meandering, the spacing should occur within one meander length (fig. TS13A–2). The same criteria used in the cross-sectional approach to determine the number of verticals to take along each section are applied here. The reach approach applies best to rivers with meanders of different wavelengths and amplitudes.

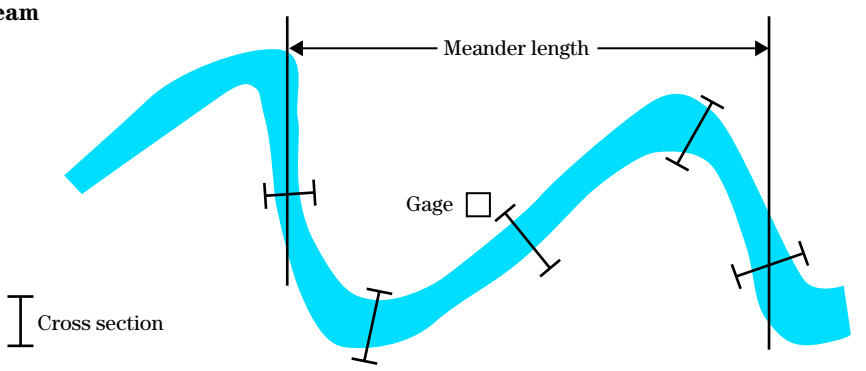
Gravel-bed streams

Coarse beds (gravel, cobble, and boulder) are characterized by significant vertical, spatial, and temporal bed-material variability. A vertical stratification in the bed material can be formed as the finer material is winnowed from the surface. A sketch of the resulting sediment profile is provided in figure TS13A–3. Another distinctive characteristic of gravel-bed streams is a coarse surface layer that may form in both the low-flow channel and on bars. Frequently, the low-flow channels of coarse bed streams are armored with large cobbles and boulders, while bars consist primarily of sand and gravel.

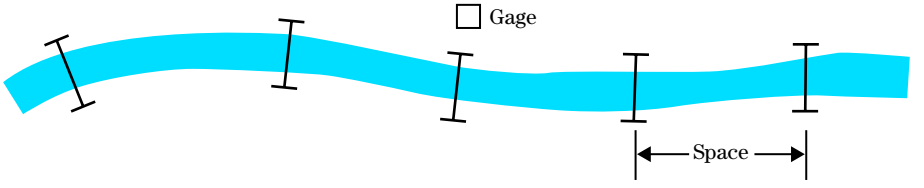
Since the spatial variability in most coarse bed streams is high, securing representative samples is difficult. River bars are frequently chosen as sampling sites because they are considered the most representative of the sediment moving in the stream, and they are usually dry during sampling. Specific bar types have been determined to be more representative than others. A bar type hierarchy established to aid site selection (Bray 1972; Yuzyk 1986) is shown in figure TS13A–4. Mid-channel and diagonal bars are most ideal sampling sites because they are exposed to the highest velocities, which transport the largest materials. Point bars are not as ideal because velocities are highly variable,

Figure TS13A–2 Bed sampling locations for sand-bed streams

Meandering stream



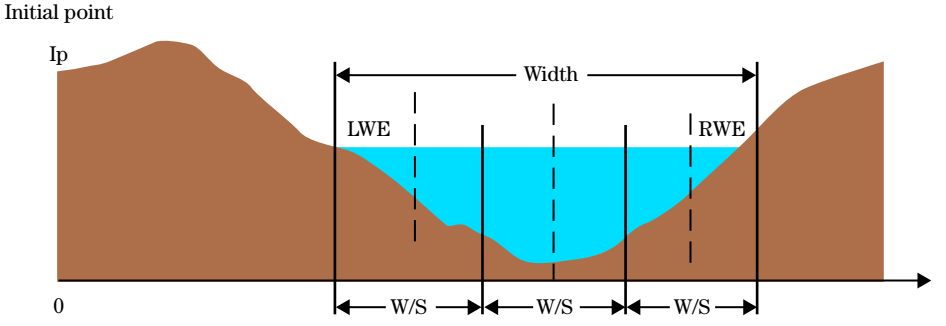
Straight channel



Space = 2 to 10 channel widths

Cross section

Panel widths=1/3 to 1/5 width of channel (W)
Sampling locations—centroid of each panel



decreasing toward the inside bank. Channel side or lateral bars are least desirable because they exist in zones of low velocities due to boundary and bank effects. In small streams with no bars and a pool-riffle sequence, the riffles may be sampled to characterize bed-material size. However, the bed material in a riffle is normally much coarser at low flow, when sediment transport is typically negligible, than at bankfull flow when sediment transport is active.

Based on the assumption that the coarsest materials in the bed exert the predominant effect on channel behavior and flow resistance, some practitioners recommend that samples be collected at the upstream end of a bar (Bray 1972; Church and Kellerhalls 1978; Yuzyk 1986). Sediments at this location are indicative of the sediments in the main channel, are readily identifiable, and generally exposed. The upstream end of a bar usually consists of the coarsest material in the channel and not the average size in the reach. This is because the upstream end of a bar is the location most frequently exposed to the highest stream velocities.

Finally, it is helpful if the bed-material sampling location is near a stream gaging station to better relate the sampled sediment data to measured hydrologic and hydraulic data.

Figure TS13A–3 Gravel-bed sediment profile showing vertical variation

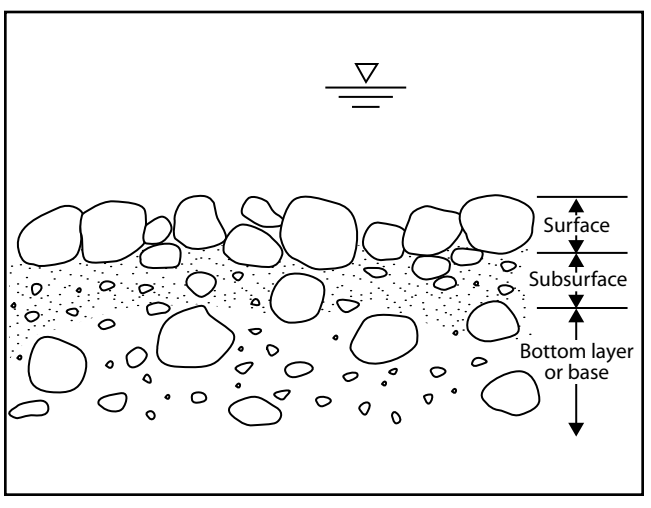
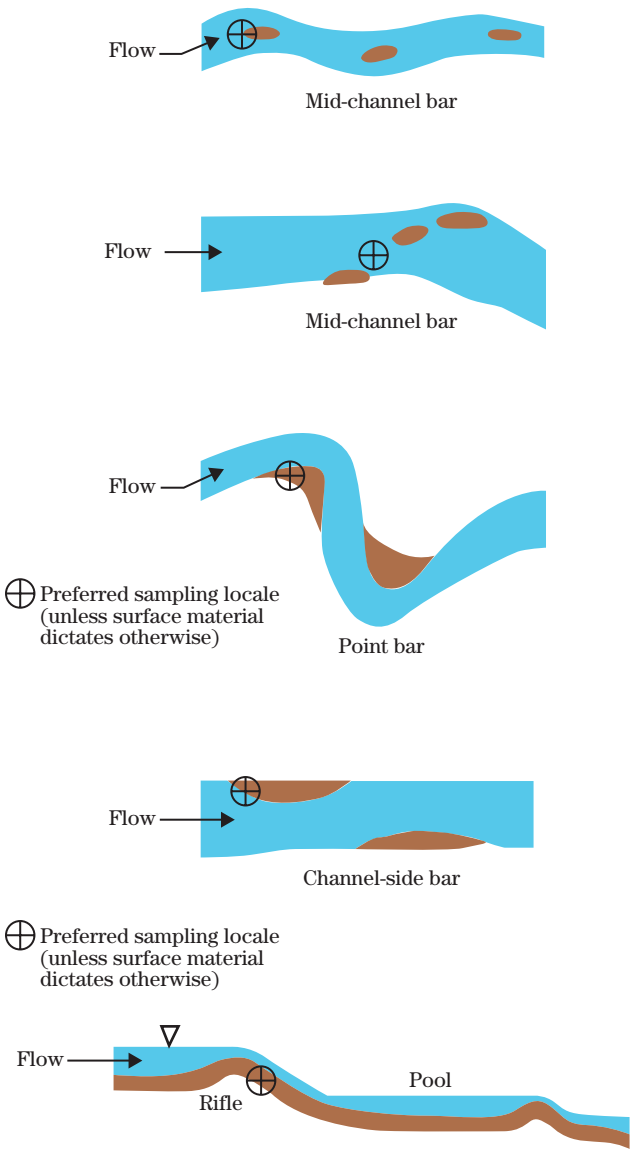


Figure TS13A–4 Coarse bed stream sampling hierarchy



Surface sediment sampling

Bulk or volumetric sampling is generally considered to be the standard sampling procedure. It involves the removal of a predetermined volume of material large enough to be independent of the maximum particle size. In general, the minimum depth of a volumetric sample should be at least twice the diameter of the maximum particle size, and the minimum weight should be 200 times the weight of the largest particle of interest (Diplas and Fripp 1992). This can lead to unrealistically large samples for many gravel-bed streams, and extrapolation may be necessary. The sample is then sieved, and the analysis is interpreted as a grain size frequency distribution by weight.

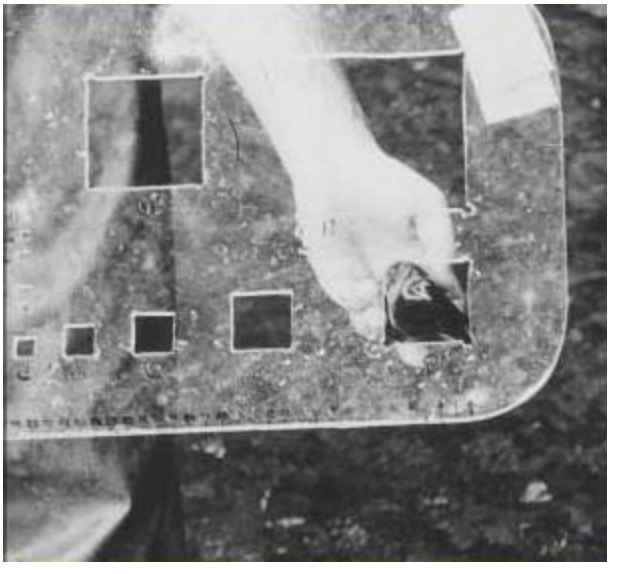
As previously noted, in coarse or gravel-bed streams, the top layers may be stratified by size due to armoring effects. Typically, bulk sampling is employed to characterize the subsurface or base layers. However, to quantify the particle size of the surface, a surface sampling technique is typically used.

Surface or areal-surface sampling is used to characterize the surface of a gravel bed. This coarse surface layer correlates to such important characteristics as hydraulic roughness, critical shear stresses, armoring, and sediment transport. A common methodology for surface sampling is a pebble count (Wolman 1954), where individual particles are collected at random by hand, and the intermediate axis is measured. The random walk method devised by Wolman can easily be employed on a dry bed or in wadeable flow, and with more difficultly by divers in deeper water. To obtain a sample, a team member paces along a selected path, stopping to collect a pebble with each step. The pebble is selected with closed or averted eyes. Other forms of this sampling include laying out a linear tape and selecting the pebble at a designated interval, laying out a preconstructed rectangular grid, and selecting the pebble at grid point intersections. The spacing of the sampling points must be at least two times the diameter of the largest particle in the sampling area. This reduces the influence of nearby particles.

At least 100 particles should be included in the hand-collected surface sample. However, to be very precise or to accurately measure small percentiles, the number of sampled particles should be increased. For example, if the D_{10} and D_{90} size fractions are of importance, the sample size should consist of at least 200 stones (Fripp and Diplas 1993). The gradation curve developed from these data is based on the number of particles in each size class, not their weights or projected surface areas. However, the resulting gradation curves are identical to those developed using sieve analysis because the selected particles all represent the same surface volume, and therefore, the same weight. The measuring process may be streamlined in the field by using a gravelometer or template (fig. TS13A–5) with standard sieve sizes to measure the sieve diameter of each particle immediately after the particle is selected. The sieve diameter for each particle is recorded as the maximum size of the opening on the template that the stone will not fit through.

Studies have shown that particles smaller than 2 millimeters are typically missed, and particles below 8 millimeters are underrepresented with Wolman or hand-based surface sampling (Fripp and Diplas 1993). This truncation is especially prevalent if the bed surface is submerged. When a sizable fraction is missing or underrepresented, the percentage of the remaining size fractions is increased, and the distribution

Figure TS13A–5 Gravelometer held above stream



becomes biased towards the larger sizes. Even gross measurements, such as median grain size, can be affected. As a result, the use of the sampled distribution can result in erroneous results. Typically, adhesive-based areal sampling is required to accurately sample surface particles.

Adhesive surface sampling uses clay, tape, or wax to remove the surface particles. Clay is generally preferable for underwater sampling. The plans for a typical clay sampling device are shown in figure TS13A–6. The clay is placed on the piston and pressed firmly onto the gravel bed. It is then drawn up into the cylinder so that the sample is protected from the stream flow as it is brought to the surface. The clay and sample material are then removed, washed to free the clay, and the sample is then sieved. The analysis is interpreted as a grain-size frequency distribution by weight.

In general, the minimum areal sample should be 100 times the area of the maximum particle of interest (Diplas and Fripp 1992). It is important to note that areal samples, which are interpreted by weight, are not directly comparable to volumetric samples, as they are biased in favor of the coarser sized material (Kellerhals and Bray 1971). The equation for converting a clay-based areal sample to its volumetric equivalent is provided below:

$$P(V-W)_i = Cp(S)_i D_i^{-1} \qquad \text{(eq. TS13A–1)}$$

where:

$P(V–W)_i$ = percentage of the frequency distribution by weight obtained for volumetric sampling

$p(S)_i$ = percentage obtained from the areal/surface sampling technique

D_i = mean diameter between size interval i and i+1

C = a proportionality constant that is unique for each sample and is calculated as

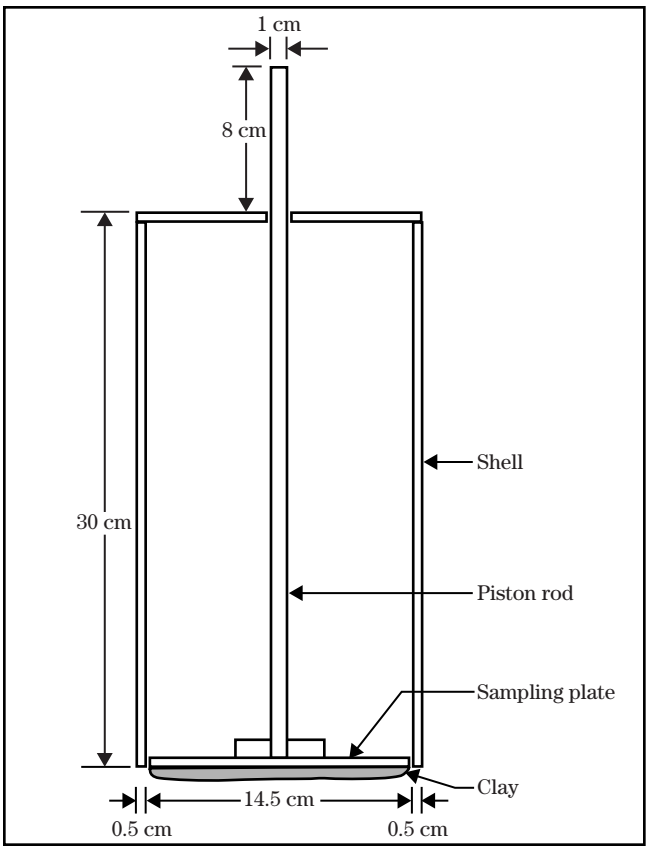
$$C = \frac{1}{\sum p(S)_i D_i^{-1}} \qquad \text{(eq. TS13A–2)}$$

Techniques for converting the material from various types of areal samples into equivalent volumetric samples are described further in Proffitt (1980); Diplas and Sutherland (1988); and Diplas and Fripp (1992).

Adhesive sampling using clay is typically limited to particles which are smaller than 40 millimeters in size (Diplas and Fripp 1992). If clay areal sampling is applied to samples containing larger material, the clay will not consistently attach to the larger size fraction, and the sample will be biased towards the smaller size fractions. Truncation can limit the obtained information and also bias the distribution.

The problem with truncation of either the smaller sizes (resulting from hand-based techniques) or the truncation of the larger sizes (as occurs with adhesive techniques) can be overcome with a combination of the two approaches. Results from an adhesive areal sample can be combined with the results of a pebble count, where the bed gradation influences significant

Figure TS13A–6 Piston sampler



amounts of both coarse and fine size fractions of material. This is done by matching the percentages where the two samples overlap. This is typically between 15 and 40 millimeters. More detailed information on this approach can be found in Fripp and Diplas (1993).

Sediment intrusion into spawning gravels

Sediment intrusion into the bed of gravel streams is an important ecological issue, as it can adversely affect fish reproduction. Sands, silts, clays, and organic matter that are deposited in gravel spawning beds, referred to as redds for salmonids, can adversely affect egg survival. The clogging of gravel beds by sands, fines, and organic matter reduces the availability of dissolved oxygen needed by salmonid embryos and fry. These deposits also restrict intergravel flows that are necessary to remove toxic metabolic wastes produced by incubating salmonid eggs. As a result, there is a need to quantify the degree of fine sediment and organic matter intrusion in gravel-bed streams.

One way to assess sediment intrusion into spawning gravels is to conduct freeze-core sampling over time (Rechendorf and Van Liew 1988, 1989). This sampling technique can be conducted for salmonids in an artificial redd built into the streambed prior to salmonid spawning. The artificial redd is constructed by excavating a depression 12 to 18 inches into the stream bed. The bottom of the depression is then lined with colored rocks or marbles. It may also be advisable to place a 2- to 3-inch piece of lead in the bottom of the hole so that a metal detector can be used to locate the site. A weighted piezometer is inserted on the floor of the depression. The piezometer can be a perforated copper pipe cast inside a Dixie® cup-sized piece of concrete, with a plastic tube on top. The plastic tube is corked and held up while the hole is backfilled. The backfilling is done by waving a shovel back and forth (winnowing) along the bottom of the channel upstream of the excavated hole. Upon movement of the backfill material upstream of the artificial redd, a small trough remains above the redd. This helps to establish flow into the upstream side of the artificial redd. This process is repeated across the stream, as well as upstream and downstream. The result is that three rows, each containing three artificial redds are constructed.

After the artificial redds are constructed and their location documented, a freeze-core sample should be taken. This should be done as soon after construction as possible to represent the prespawning clean redd condition.

Freeze-core sampling involves installing three metal probes (preferably copper) into the streambed and then freezing the rods. It is often necessary to divert high velocity water around the sample site. A 5-gallon bottomless bucket is then worked a few inches into the streambed at the sample site. The metal rods are then driven 12 to 18 inches into the bed in a triangular pattern within the bucket. The rods should be 3 to 6 inches apart. A tether to a bottle of compressed carbon dioxide is placed to each copper rod, and the rods are frozen for approximately 20 minutes. A heavy aluminum tripod is then placed over the bucket, and a winch is used to remove the frozen sample from the streambed.

The frozen sample should be placed in a box with adjustable separators so that depth increments below the surface can be established. As the sample thaws, the material will fall into the compartments. The bottom of the artificial redd is established by colored rocks or marbles. Each depth increment can then be dried and sieved. Stream freeze-core sampling is repeated with one sample at each location, progressively through the sediment runoff season. Periodic dissolved oxygen measurements can be made by extracting water through the piezometer. More information on the use of this technique can be obtained in Castro and Reckendorf (1995).

Selection of a sampling procedure

Several factors influence both sampling site selection and sampling procedure. The most significant factor is the data necessary to meet the objectives of the study at hand. The objective of a bed-material sampling program may be to determine a representative bed gradation for a particular reach of a stream, or it may be to determine the variability and diversity of the sediment bed. Data needs should be clearly defined before the sampling program is planned. The second factor to consider is field conditions. Different samplers and sampling procedures are appropriate for different environments. Therefore, it is necessary to know the

general streambed characteristics before the sampling program is established. Such reach-specific questions need to be addressed such as:

- Will the bed of the stream be wet or dry?
- Is the site accessible by road, boat, trail, or only by helicopter? Field conditions will determine both the practicality and type of sampling equipment to be used in the sampling program.
- What is the nature of the bed material to be sampled? Sand-bed streams typically have a more uniform bed gradation and therefore require a smaller volume sample than gravel-bed streams. Typically, equipment appropriate for sampling sand-bed streams is inappropriate for gravel-bed streams.

Once these physical issues are assessed, the available resources must be considered as a limiting factor when establishing a bed sampling program. Equipment, manpower, and funds are frequently limited, and therefore, priorities must be established.

Step-by-step field sampling procedures

Step 1 Select and mark out the required cross sections and the sampling locations. Use as many of the site-selection criteria outlined above as possible. The fixed permanent initial point should be on the left bank (looking downstream). Establish the control (horizontal and vertical) and reference all points.

Step 2 Sketch the site on data forms and reference the control points. If the streambed contains a mixture of sand and gravel deposits, map areas and record deposits of different size material. Develop a sampling strategy that will sample each zone.

Step 3 Collect a photographic record of the reach, controls, cross sections, sample locations (if possible), bed material (use a scale for reference), and bank conditions.

Step 4 Select appropriate sampler for the task (based on depth, velocity, and sample requirements). Verify that the sampler is operational.

Step 5 Collect sample as follows:

Surface bulk sample: sand bed. Move to a sampling location. In shallow streams, use a tape to measure from the permanently fixed initial point (IP), and wade to a sampling vertical on the section. Approach the sampling verticals from the downstream side to prevent disturbing the bed at the sampling section. In deep streams, using a boat and some type of positioning system (tag-line in narrow streams, electronic distance measurement (EDM) in wide streams), hold the boat steady over the sampling location. Obtain a sample of about 250 grams at each chosen location using the selected sampler.

Surface areal sample: coarse bed. To obtain a surface areal sample in a coarse bed stream, several techniques are employed. These can include random walks, setting up square or linear grids, or removing all the surface particles within a specified area. Hand-based techniques are typically employed, but they can be biased towards the larger size fractions. Collecting the entire surface layer within a specified area generally requires a specialized sampler.

Surface bulk sample: coarse bed. To obtain a surface bulk sample, carefully remove and collect all sediment in the surface layer to a thickness of the intermediate axis of the largest particle in the area. Care should be taken to ensure that fine sediment is not washed out of the sample. The required sample mass is a function of the largest particle.

Subsurface bulk sample: coarse bed. If the surface layer has not already been removed, then scrape away the surface layer of coarse material to the thickness of the intermediate axis of the largest particle in the area. The required sample mass is also a function of the largest particle.

Step 6 (Field sieving—this step is an alternative to transporting large bulk samples to a laboratory.) Set up a weighing station. This may consist of a tripod with a scale suspended for weighing pails of material. Assemble field sieve sets, and insert correct sieves. Collect pails, spades, template, labels, field note forms, sturdy plastic bags, and tarpaulins. Spread out two tarpaulins. Obtain tare weights for the pails. Shovel subsurface material into pails, weigh, and record. Pour material

into top of the field sieves (8, 16, 32, 64, 128 mm sieves). Rock and shake the sieve set until material has moved to its retained size sieve. Weigh material retained on each sieve and on the pan. Record the results in the field notes. Save the material passing the finest sieve size for laboratory analysis. Save the 10 largest particles. Repeat the process until the required mass has been sieved. Measure the three perpendicular axes of the 10 largest particles. Retain up to 10 kilograms of the combined material from the pan and discard the rest of the sample.

Step 7 Complete and attach a label and sediment field note form for each sample. Specify the stream, station, cross section, vertical location, date, time, bedform and flow conditions, personnel on crew, type of sampler, sample number, and sample depth.

Other bed-material characteristics

While deposited bed material is often characterized by grain size, other characteristics can be of concern, as well. Such particle characteristics include shape, specific gravity, lithology, and mineralogy. In addition, data that describe the distribution of the various particles sizes and of specific contaminates are frequently required. Characteristics of the sediment deposit itself include: stratigraphy, density, and compaction. For some of these purposes, a sample can be disturbed; others require undisturbed sampling.

When the sediment particles are noncohesive, mechanical forces dominate the behavior of the sediment in water. The three most important properties that govern the hydrodynamics of noncohesive sediments are particle size, shape, and specific gravity. A discussion of these properties is found in Sedimentation Investigations in Rivers and Reservoirs, EM 1110–2–4000 (U.S. Army Corps of Engineers (USACE) 1995c). The boundary between cohesive and noncohesive sediments is not clearly defined. It can be stated, however, that cohesion increases with decreasing particle size for the same type of material. Clays are much more cohesive than silts. Electro-chemical forces dominate cohesive sediment behavior. The three most common clay minerals that have electro-chemical forces causing individual particles to stick together are illite, kaolinate, and montmorillonite. The dispersed particle fall velocity, flocculated fall velocity of the suspension, clay and nonclay mineralogy, organic content, and cation exchange capacity characterize cohesive sediment. The fluid is characterized by the concentration of important cations, anions, salt, pH, and temperature. More detailed information is presented in Tidal Hydraulics, EM 1110–2–1607 (USACE 1991c).

Bank material

Many channel stability issues result from a combination and interaction of a number of different causes. These causes can include not only fluvial erosion forces but also seepage problems, as well as properties of the soil. In addition, the bank material can help define the stability of the channel section and may be responsible for a significant percentage of the total sediment load. Therefore, it is often important to determine the characteristics of the stream bank. This is often done coincident with the bed-material sampling. More information on issues related to the assessment and analysis of bank material is provided in NEH654.09.

Conclusion

Bed-material sampling is frequently conducted to make sediment transport calculations. For this purpose, the sampling program should identify not only a representative bed-material gradation, but also any lateral, longitudinal, vertical, and/or temporal variation in bed-material composition. Water depth, velocity, and bed-material size are the most important factors used to identify appropriate samplers and sampling procedures. In sand-bed streams, the sample is typically taken from the upper 5 centimeters of the bed surface. In gravel-bed streams with coarse surface layers, samples of both the surface and subsurface layers are required. Surface sampling of large particles can be done by hand using a pebble count method. However, a pebble count can be biased if there is a significant size fraction that is below 8 millimeters in size. For smaller particles, an adhesive surface sampling approach is often considered necessary.

Technical Supplement 13B

Sediment Budget Example

Issued August 2007

Cover photos: *Top*—Sediment comes from a variety of sources including the watershed and bed and bank materials. For a successful restoration, the amount of sediment entering a stream reach must be balanced by the sediment transport capacity of the stream.

Bottom—An inventory of erosion types and the amount of sediment coming from these sources may be needed for design.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 13B

Sediment Budget Example

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Technical Supplement 13B

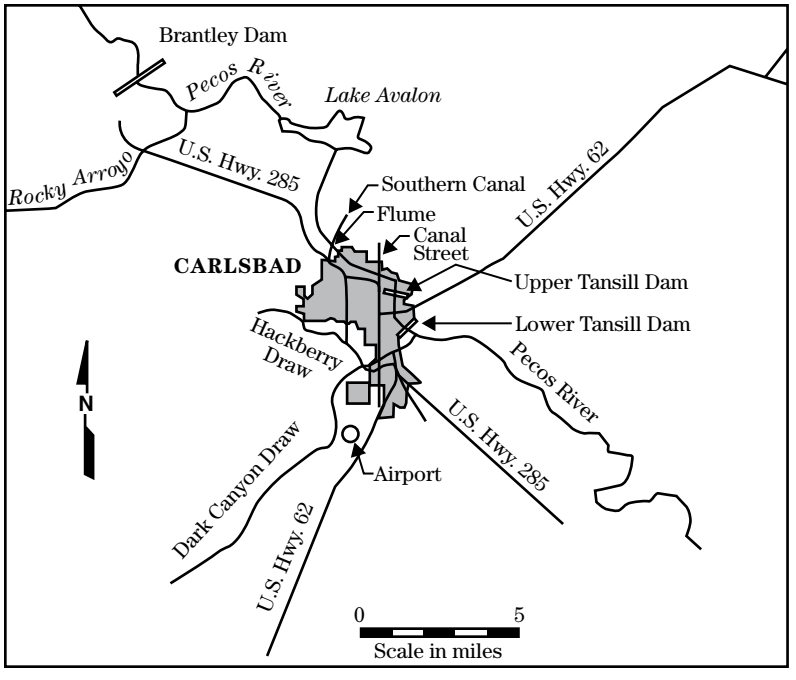
Sediment Budget Example

Introduction

A sediment budget analysis was conducted by the U.S. Army Corps of Engineers (USACE) as part of the reconnaissance level planning study for a flood-damage reduction project for the City of Carlsbad, New Mexico (Copeland 1995). This example describes the sediment budget analysis used to identify the magnitude of possible sediment problems that might be associated with one of the proposed project designs. One potential source of flooding was Dark Canyon Draw, a tributary of the Pecos River (fig. TS13B–1). One of the flood damage reduction alternatives being considered was a bypass channel that would divert Dark Canyon Draw around the city of Carlsbad. The proposed diversion would begin near the city airport and flow northeasterly to the Pecos River to a location about 5 miles downstream from the city.

The sediment budget analysis was conducted to determine the magnitude of possible sediment degradation or aggradation problems that might occur with a proposed design for the diversion channel. Depending on the diversion channel design, several sedimentation and channel stability problems could occur. If a threshold channel is constructed that is designed with little or no sediment transport potential, then bed material delivered from upstream would deposit at the diversion entrance. Sediment deposits would have to be removed periodically. If a channel is designed to carry the incoming sediment load, the channel would undergo a period of adjustment as the bed and banks become established. Bed armoring could progress quickly or slowly, with extensive degradation, depending on the consistency of the material through which the diversion channel is cut and the sequence of annual runoff that occurs. Finally, if the diversion channel is too efficient in terms of sediment transport capacity, it could degrade and induce additional channel degradation upstream from the diversion location.

Figure TS13B–1 Carlsbad and surrounding areas



Field reconnaissance

Preliminary assessments of channel stability and potential sediment impacts were determined during the site assessment and investigation phase of the study conducted prior to the project design phase. Data collected during this phase of the study were used in the sediment budget analysis, which was conducted after channel design.

Dark Canyon Draw transitions from a wide, shallow alluvial channel, characteristic of southwestern United States alluvial fans, at its canyon mouth to an incised arroyo at its confluence with the Pecos River. Gravel mining is currently active in the lower reaches of Dark Canyon Draw between the Pecos River and the city airport and has been occurring for many years. The channel had been both widened and deepened due to the gravel mining. The channel also showed signs of incision/degradation upstream from the airport. The bed and banks of the incised channel were capable of supplying significant quantities of sediment to the stream. The bed surface of Dark Canyon Draw consisted primarily of coarse gravel and cobbles. Banks were generally composed of loose alluvial material ranging in size from clays and silts to boulders. The channel tended to migrate laterally, eroding banks, and creating remnant gravel bars in former channels. Armoring was generally observed in the existing low-flow channel. However, the channel would migrate at high flows, mobilizing significant amounts of sediment from the gravel bars and from eroded bank materials.

Bed-material samples were collected during the field reconnaissance. Sample size class distributions were determined using the Wolman (1954) pebble count method and the volumetric bulk method. Due to the limited scope of the sediment impact assessment, samples were collected at only two sites. Both surface and subsurface samples were collected at the mouth of the canyon several miles upstream from the proposed diversion channel. There was no coarse surface layer at the second site, located on a gravel bar about 1 mile downstream from the canyon mouth. The thoroughly mixed bedform was an indication that active-layer mixing had occurred during the last flow event at this site (fig. TS13B–2). Median grain size ranged between 22 and 55 millimeters for all the samples. The gradation determined at the downstream site was selected as the representative gradation for the sediment budget analysis because it was characteristic of a fully mobile bed. Bed-material gradations determined from these samples are shown in figure TS13B–3.

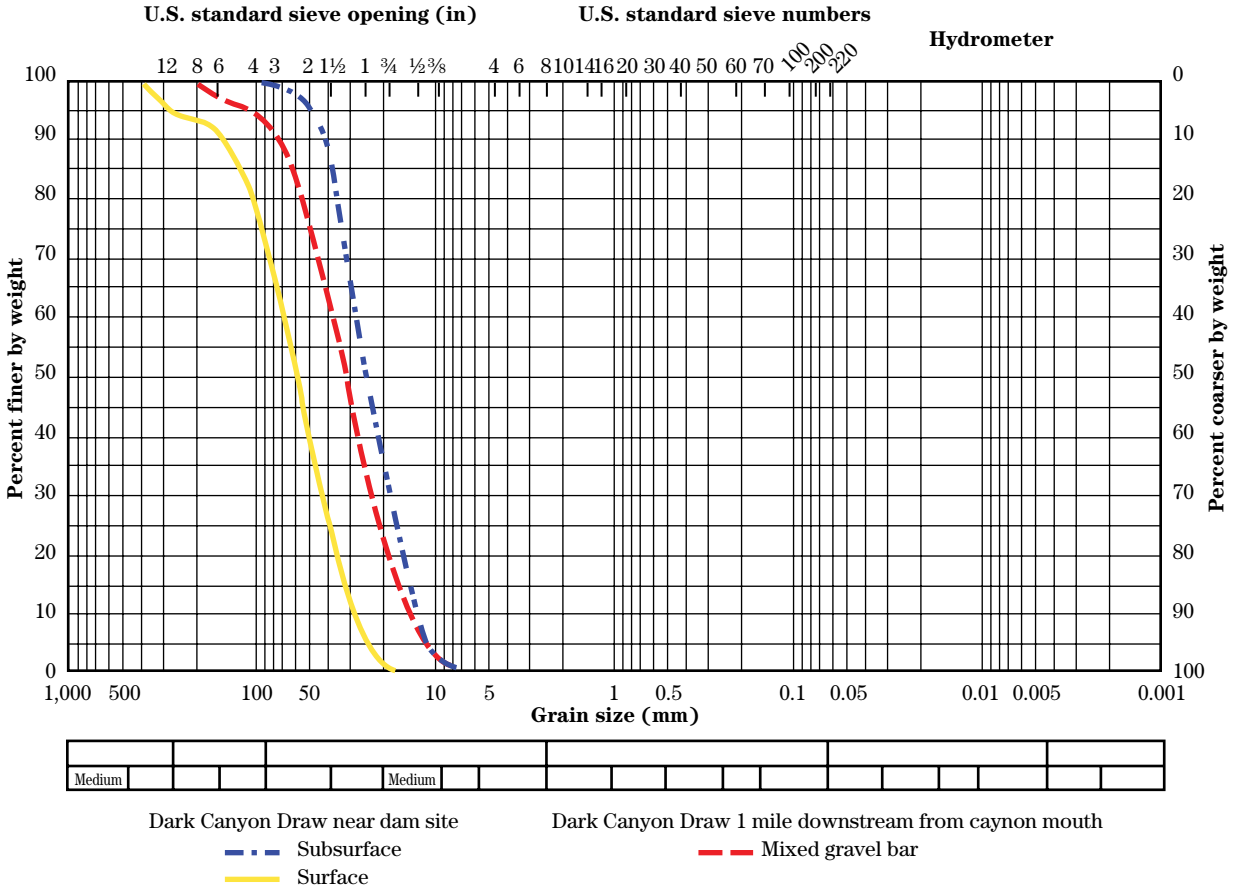
Hydrology

Hydrographs used in the sediment budget analysis were developed using the HEC–1 hydrograph package (USACE 1998b). These were used to calculate sediment yield for flood events. The peak discharge for the 1 percent exceedance flood was 2,000 cubic meters per second (75,000 ft^3/s). The 10 percent chance exceedance hydrograph was assumed to have the same shape as the 1 percent chance exceedance flood. Discharges on the hydrograph were calculated by multiplying the 1 percent exceedance hydrograph by the ratio of the peaks. The peak discharge for the 10 percent chance exceedance was 570 cubic meters per second (20,000 ft^3/s).

A flow-duration curve was developed from 18 years of U.S. Geological Survey (USGS) mean daily flow data from the Dark Canyon at the Carlsbad gage. Durations of published peak flows greater than the maximum mean daily flow were added to the flow-duration data by assuming that the historical flood hydrographs had

Figure TS13B–2 Mixed-gravel bedform, Dark Canyon Draw

Figure TS13B–3 Bed-material gradations, Dark Canyon Draw



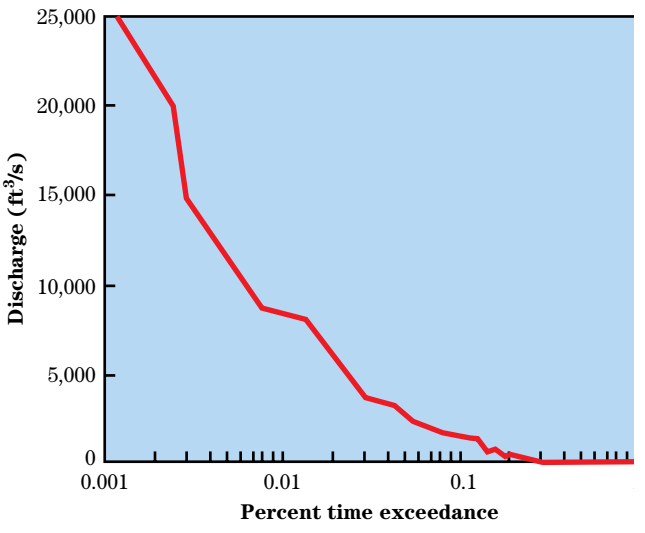
shapes similar to the 1 percent chance exceedance hydrograph. The flow-duration curve is shown in figure TS13B–4.

Average hydraulic parameters

A typical reach in the existing Dark Canyon Draw channel was selected from a HEC–2 backwater model (USACE 1990b). The typical reach chosen for this analysis was about 2 miles long and located adjacent to the Carlsbad Airport. The reach was considered to be in a state of nonequilibrium due to its proximity to gravel mining operations. A reach further upstream, less influenced by gravel mining operations, would have been preferred for determining long-term sediment yield. However, the existing backwater model did not extend any further upstream. It was recommended that additional cross-sectional surveys be obtained upstream for more detailed sediment studies.

Water-surface elevations and hydraulic variables were calculated using the HEC–2 model for a range of discharges. Average values for hydraulic variables were then determined using the reach-length weighted averaging procedure in SAM (Thomas, Copeland, and McComas 2003).

Figure TS13B–4 Dark Canyon Draw, Carlsbad, NM, flow-duration curve (1973–1992)

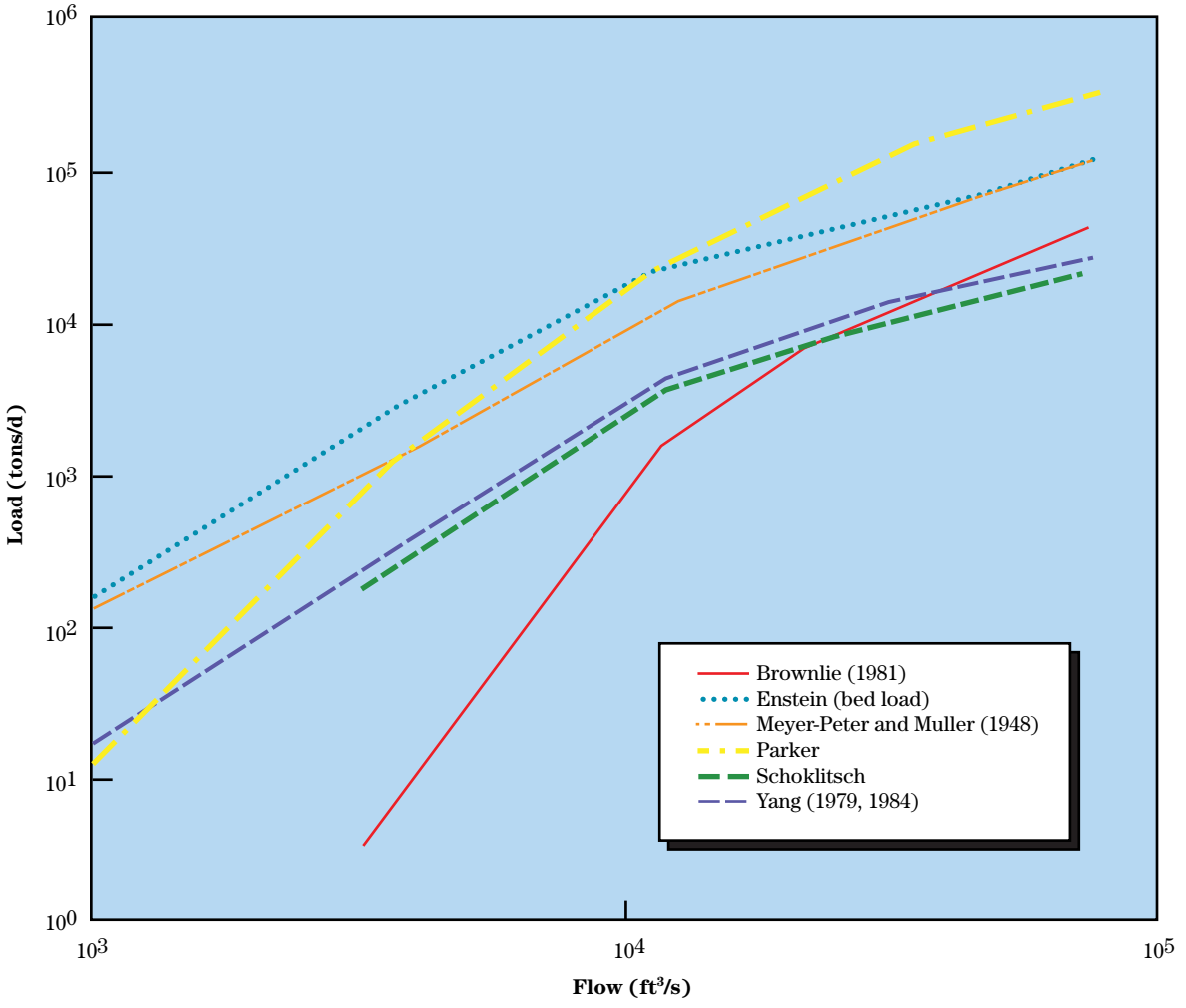


Sediment transport rating curve

The bed-material sediment yield from Dark Canyon Draw is important when considering sediment transport and channel stability questions. The bed-material sediment load consists of the sediment sizes that exchange with the streambed, as they are transported downstream. The bed-material yield is most likely to be relatively small compared to the total sediment yield because the bed of Dark Canyon Draw consists primarily of gravels and cobbles. The wash load component of the total sediment yield will be transported through the system to the Pecos River unless it is trapped by a reservoir or introduced into a ponded area.

Sediment transport was calculated using several sediment transport equations available in the SAM program. The equations chosen included at least some data from gravel-bed rivers in their development. As can be seen from the sediment discharge rating curves (fig. TS13B–5), predicted sediment transport rates cover a wide range. No data are available on Dark Canyon Draw to aid in the selection of a transport equation. However, the guidance program in SAM identified the North Saskatchewan and Elbow Rivers in Saskatchewan, Canada, as having similar median bed grain sizes, depths, velocities, and slopes as Dark Canyon Draw at high flow. The guidance program from the available set of equations in SAM determined that the Schoklitsch equation (Shulits 1935) best reproduced measured data on the North Saskatchewan and Elbow Rivers. Calculated sediment transport rating curves were compared using different sediment transport functions, as shown in figure TS13B–5. The conclusion is that the Schoklitsch equation will produce a relatively low sediment yield. To cover the uncertainty range in the calculated bed-material sediment yield, two additional sediment transport equations were chosen to calculate yield. The Parker equation (Parker 1990) was used to represent a high sediment transport load, and the Einstein (1950) equation was chosen to represent an intermediate sediment transport load.

Figure TS13B–5 Bed-material sediment transport rating curves, Dark Canyon Draw



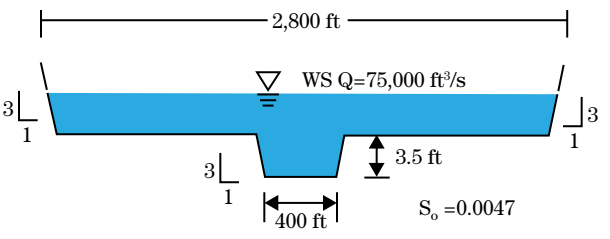
Diversion channel design

The following criteria were chosen for the diversion channel design:

- a composite channel geometry with a low-flow channel designed to carry the effective discharge
- the overbank flow designed using threshold criteria for the 1 percent chance exceedance flood

Assigned side slopes were 1V:3H, with Manning's roughness coefficient of 0.05 for the side slope. The project cross section for the diversion channel to be evaluated with the sediment budget analysis is shown in figure TS13B–6.

Figure TS13B–6 Cross section, Dark Canyon Draw diversion channel



Sediment budget

The magnitude of potential aggradation or deposition problems in the Dark Canyon channel can be determined by calculating bed-material sediment yield through a typical reach of the existing channel and comparing it to calculated sediment yield in the project reach.

Bed-material sediment yield was calculated for the existing channel using the flow-duration sediment transport curve method and SAM. Sediment yields were calculated for the 1 percent and 10 percent chance exceedance floods using synthetic hydrographs, and for average annual conditions, using the flow-duration curve. Bed-material sediment yields were calculated using three different sediment transport equations. Results are shown in table TS13B–1.

Sediment yield was determined in the diversion channel using the same procedure that was used to calculate sediment yield in the typical reach of the existing channel. Sediment trapping efficiency was then determined for flood hydrographs and for average annual conditions.

Table TS13B–1 Calculated bed-material sediment yield[1/], Dark Canyon Draw

Bed-material transport function	1 percent exceedance flood		10 percent exceedance flood		Average annual	
	m³	yd³	m³	yd³	m³	yd³
Schoklitsch	2,400	3,100	530	690	180	230
Einstein	11,300	14,800	3,300	4,300	1,300	1,700
Parker	27,700	36,200	4,100	5,400	1,100	1,500

1/ Sediment yield volume calculated assuming specific weight of deposit of 1,500 kg/m³ (93 lb/ft³)

The potential for aggradation or degradation in the diversion channel for a 10 and 1 percent chance exceedance floods and for average annual conditions was determined using the sediment budget approach. Bed-material sediment yield was calculated using three sediment transport equations and compared to the calculated bed-material sediment yield in the existing Dark Canyon Draw. Bed-material sediment transport was assumed to occur only in the low-flow channel in the diversion.

Calculated bed-material sediment yield and its percentage of the total bed-material yield calculated for Dark Canyon Draw is shown in table TS13B–2. This tabulation indicates that deposition will occur in the diversion channel for all cases tested. For the 1 percent chance exceedance flood, between 34 and 38 percent of the inflowing bed-material sediment load will be deposited in the diversion channel. For the 10 percent chance exceedance flood, between 12 and 17 percent of the inflowing bed-material load will be deposited. For average annual conditions, between 6 and 18 percent of the inflowing sediment load will be deposited. A range anticipated deposition rates can be determined from these calculations. Recall that the Schoklitsch equation produced sediment transport quantities closest to the measured data from a river with similar characteristics.

Further analysis

At the next level of planning, it would be necessary to evaluate the temporal development of the diversion channel using the HEC–6 numerical sedimentation model. In this sediment impact assessment, the bed-material gradation was assumed to be already developed. A more detailed study would require knowledge of the existing soil profile through which the channel will be cut. The armoring process would then be simulated with a numerical model. In addition, the slope of the diversion channel will vary between the diversion point and the Pecos River. This requires a more detailed analysis of spatial variability in the sedimentation processes.

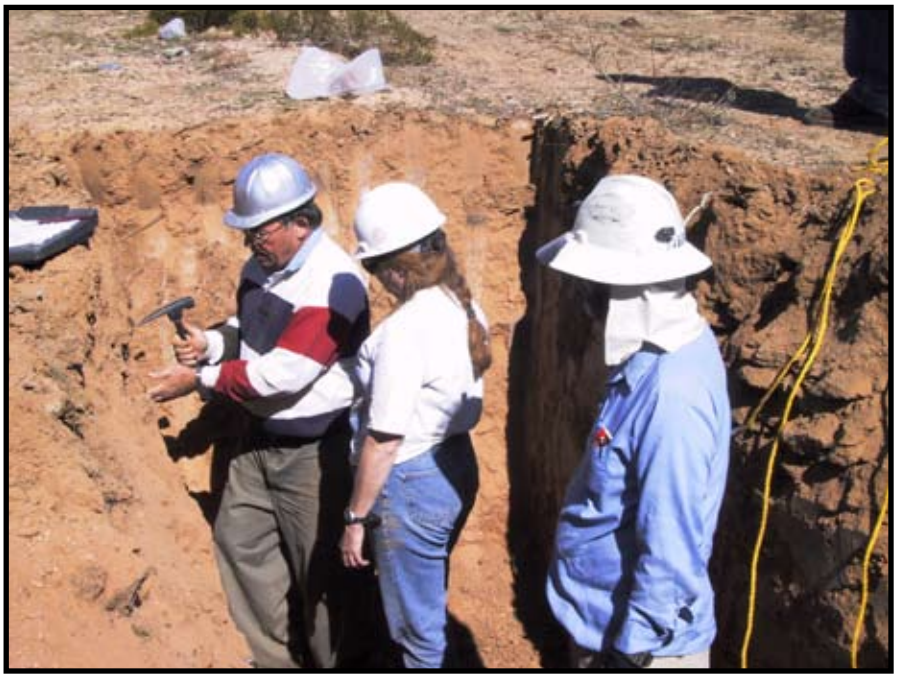
Table TS13B–2 Calculated bed-material sediment yield[1/], diversion channel

Sediment transport function	1 percent exceedance flood			10 percent exceedance flood			Average annual		
	m^3	yd^3	% of inflow	m^3	yd^3	% of inflow	m^3	yd^3	% of inflow
Schoklitsch	1,600	2,050	66	450	590	86	150	190	82
Einstein	7,500	9,800	66	2,900	3,800	88	1,200	1,600	94
Parker	17,100	22,400	62	3,400	4,500	83	1,000	1,300	87

1/ Sediment yield volume calculated assuming specific weight of deposit of 1,500 kg/m^3 (93 lb/ft^3)

Technical Supplement 14A

Soil Properties and Special Geotechnical Problems Related to Stream Stabilization Projects



(210–VI–NEH, August 2007)

Issued August 2007

Cover photos: *Top*—Where streambanks are unstable, their drained and undrained conditions may need to be evaluated and tested, both onsite and in the laboratory.

Bottom—Geotechnical analysis begins with logging and sampling of the bank materials.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14A

Soil Properties and Special Geotechnical Problems Related to Stream Stabilization Projects

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Technical Supplement 14A

Soil Properties and Special Geotechnical Problems Related to Stream Stabilization Projects

Purpose

The purpose of this technical supplement is to describe special geotechnical problems related to stream stabilization projects.

Topics addressed in this section include:

- parameters used for classifying soils into engineering behavior groups
- recognizing streambank instability and erosion problems that have geotechnical root causes
- piping/sapping of streambanks
- surficial failures in blocky-structured, highly plastic clays
- severe erosion in dispersive clays
- remedial methods for stabilizing slopes where oversteepening is a result of erosion of the toe of the slope

Introduction

Soil bioengineering measures increase stream roughness and slow the water velocity near the slope face. They also armor and reinforce the surface soils. However, some problems with instability and excessive erosion of streambanks are not readily solved by soil bioengineering techniques alone. Problems involving rotational failures of streambanks, piping (sapping) of bank soils, and shallow slides in highly plastic soils are difficult to solve using only soil bioengineering techniques. Erosion on streambanks in highly dispersive clay soils also cannot be solved with soil bioengineering measures alone. If appropriate remedial solutions are to be designed, engineers and planners must recognize and understand special instability problems that have underlying geotechnical causes.

Analyzing bank slopes for geotechnical stability requires an understanding of a complex system of forces. Evaluating how to protect the soils in the slopes from the erosive forces of flowing water acting against otherwise unprotected streambanks frequently is only part of the task. Even if banks are protected from the erosive forces of the water in the channel, external forces including seepage from the bank and gravity acting on soils in the bank can induce slope failures. The forces involved in bank instability problems include gravity acting on the soils in the slope, the internal resistance of soils in the slope, seepage forces in the soils in the slope, as well as the tractive stresses imposed on the soils by flowing water.

Designing various methods for streambank stabilization, such as retaining walls, reinforced fills, sheet piles, and others, requires specialized engineering experience and knowledge. Analytical methods require parameters that are either estimated from other soil properties or obtained in laboratory testing designed for obtaining them.

Soil classification

The Unified Soil Classification System (USCS) is used to group soils based on similar engineering behavior. The USCS is described in two American Society for Testing and Materials International (ASTM) Standards. ASTM D2487 details classifying soils in the USCS using laboratory data. ASTM D2488 describes methods for estimating the classification of a soil from field tests. Classifying soils by the USCS requires data on the following parameters:

- The percentage by dry weight of the total sample that is of three size categories: fines, sands, and gravels. The USCS only considers the portion of a deposit finer than 3 inches. Larger particles are described, but not included in classification procedures. A more detailed description of the three particle size groups is:
  - Percent fines is the percent of the sample finer than the #200 sieve. These particles are smaller than 0.075 millimeter. Particles finer than the #200 sieve include silt and clay size particles that are usually also evaluated with Atterberg limit tests described later in this section. Percent fines is one of the most important parameters in identifying soil types.
  - Percent sand is the percentage of the sample consisting of sand size particles, which are particles larger than the #200 sieve (0.075 mm) and smaller than gravel size particles described next (smaller than 4.76 mm).

– Percent gravel size is the percentage of the total sample consisting of particles larger than 4.76 millimeters, but smaller than 3 inches.

- Soils with 50 percent or more fines content and those coarse-grained soils with significant clay and silt content (more than 5% fines), are usually also evaluated by performing Atterberg limit tests on the portion of the sample smaller than a #40 sieve. Atterberg tests are useful in identifying the water holding and plasticity characteristics of those soils.

The relative denseness or looseness of sandy and gravelly soils with few fines may be characterized with simple field tests such as the one described in table TS14A–1.

The saturated consistency of fine-grained soils with significant plasticity (plasticity index greater than about 7) correlates well with the soils' undrained shear strength. Saturated undrained strength of plastic fine-grained soils may be estimated with a field torvane device such as the one shown in figure TS14A–1 or from the descriptions provided in table TS14A–2.

Figure TS14A–1 Hand held torvane device (*Photo courtesy Geotest Instruments, Inc.*)

Table TS14A–1 Description of coarse-grain soil relative density

Density description	Evaluation/description
Very loose	A ½-in-diameter rod can be pushed easily by hand into soil
Loose	Soil can be excavated with a spade. A 2-in, square, wooden peg can easily be driven to a depth of 6 in
Medium dense	Soil is easily penetrated with a ½-in rod driven with a 5-lb hammer
Dense	Soil requires a pick for excavation. A 2-in, square, wooden peg is hard to drive to a depth of 6 in
Very dense	Soil is penetrated only a few cm with a ½-in rod driven with a 5-lb hammer

Table TS14A–2 Description of fine-grain soil consistency

Saturated consistency	Evaluation/description	Estimated undrained shear strength (lb/ft^2)
Very soft	Thumb will penetrate greater than 1 in. Soil is extruded between fingers	<250
Soft	Thumb will penetrate about 1 in. Soil molded by light finger pressure	250–500
Medium	Thumb will penetrate about ¼ in. Soil molded by strong finger pressure	500–1,000
Stiff	Indented with thumb	1,000–2,000
Very stiff	Indented by thumb nail	2,000–4,000
Hard	Thumbnail will not indent	>4,000

Soil shear strength

The shear strength of soils may vary depending on the rate that load is added to the soil, duration of the load, whether a previous load has been exerted on the soil (in particular for overconsolidated clays), and the permeability of the soil. Shear strength parameters are often characterized as undrained and drained parameters. The terms undrained and drained are not a description of the water level in the soils, but rather a description of the pore pressure condition in the soil when it is loaded. An undrained condition (also called short term, quick, total stress, or unconsolidated-undrained) assumes that pore pressures will develop due to a change in load. The assumption is that the pore pressures that develop are not known and must be implicitly considered in the methods used to test samples for this condition.

A drained condition (also called long term, slow, effective stress, or consolidated-drained) implies that either no significant pore pressures are generated from the applied load or that the load is applied so slowly that the pressure dissipates during the slowly applied loading.

Relatively permeable soils

Soils with a permeability of 1×10^{-4} centimeter per second or greater are often assumed to have a permeability rate high enough that excess pore pressures do not develop from loads applied at normal rates. Soils with these characteristics are generally in the following groups:

- coarse-grain soils with less than 5 percent fines
- coarse-grain soils with more than 5 percent fines, but with fines which have a plasticity index less than 8
- fine-grain soils with a plasticity index less than 5

The shear strength of this category of soils is measured using consolidated-drained (CD) or consolidated-undrained conditions with pore pressure measurements (CU′) shear tests. The shear strength of this group of soils may also be estimated from *in situ* tests such as standard penetration tests or cone penetration tests. The drained shear strength applies to both short-term and long-term load conditions. Estimated shear strength parameters for this category of soil types are shown in table TS14A–3.

Soils with relatively low permeability

Soils with relatively low permeability (a coefficient of permeability less than about 1×10^{-4} cm/s) behave in a more complex manner. The shear strength of these soils varies depending on the rate of load application. Soils that are not in the categories described are usually in this group. If a soil has low permeability and experiences a fast change in load, undrained shear strength parameters are appropriate for analyses. After a load is maintained for a sufficient period of time, the pore pressures generated by the load application will dissipate. At that time, the soil will exhibit drained shear stress parameters.

Analyses of fine-grain soils should consider both undrained and drained conditions, with the most critical condition governing the design. Typical soil properties for fine-grain materials are shown in table TS14A–3 (U.S. Army Corps of Engineers (USACE) 1994c, EM 1110–2–2504, Design of Sheet Pile Walls; Pile Buck Steel Sheet Piling Design Manual; and U.S. Navy, Naval Facilities Engineering Command (NAVFAC) DM–7.2, Foundations and Earth Structures). Peak effective phi (ϕ) angles for slowly permeable soils may be estimated with empirical charts such as shown in figure TS14A–2 (Hopkins, Allen, and Dean 1974; Kenny 1959; Bjerrum and Simons 1960).

Stiff, fissured clays

Overconsolidated clay soils often contain fissures and slickensides. They behave differently than soils with similar plasticity, which do not have these features. Slope stability analyses and the design of sheet pile walls should consider the fully softened shear strength, which models the effect on shear strength of the network of discontinuities in the soil. If the slope or wall is designed to stabilize a recent slide, the residual shear strength should be considered. Both

Table TS14A–3 Estimated soil properties

Soil type 1/	Moist unit weight (lb/ft^3)	Saturated unit weight (lb/ft^3)	Undrained shear strength properties		Drained shear strength properties		Angle of wall friction (steel pile), δ	Wall/soil adhesion 3/ (lb/ft^2)
			Cohesion (lb/ft^2)	Angle of internal friction, ϕ	Cohesion (lb/ft^2)	Angle of internal friction, ϕ		
Loose sand	95–125	120–130	0	28	0	28	0.5xϕ	0
Medium dense sand	110–130	125–135	0	32	0	32	0.5xϕ	0
Dense sand	110–140	130–140	0	38	0	38	0.5xϕ	0
Very soft clay	85–100	85–100	0–250	0	0	See note 2	0.5xϕ	0–250
Soft clay	100–120	100–120	250–500	0	0	See note 2	0.5xϕ	250–500
Medium clay	110–125	110–125	500–1,000	0	0	See note 2	0.5xϕ	500–750
Stiff clay	115–130	115–130	1,000–2,000	0	50–100	See note 2	0.5xϕ	750–950
Very stiff clay	120–140	120–140	2,000–4,000	0	100	See note 2	0.5xϕ	950
Hard clay	>130	>130	>4,000	0	100	See note 2	0.5xϕ	950

Notes:
1/ See tables TS14A–1 and TS14A–2 for qualitative descriptions of soil types.
2/ See figure TS14A–2.
3/ Wall/soil adhesion is typically 0 for drained (long-term) conditions.

the fully softened phi angle and the residual phi angle of these soils are independent of the original strength of the clay and such factors as water content and liquidity index. The strength of these soil types seems to depend only on the size, shape, and mineralogical composition of the constituent particles and the effective normal stress (Stark and Hisham 1997). Fully softened phi angles are usually assumed to be in the range of 18 to 26 degrees and residual phi angles in the range of 6 to 18 degrees. This special type of soil is described further in the following sections with photographs and problems that are associated with the soil type.

Stability evaluations of bank slopes

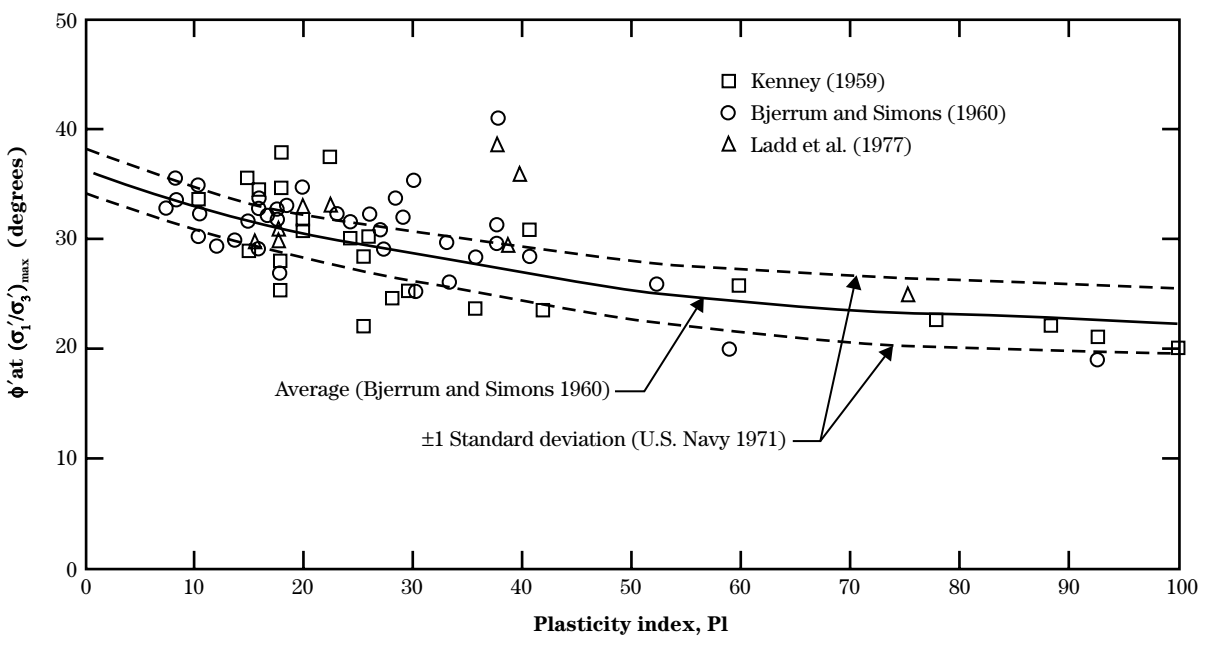
Stream channel banks can fail when conditions change that affect the stability of the slope. Examples of changes in conditions include changes in the potentiometric surface (water table) in the slope; changes in the slope configuration including increased height of the slope due to stream bed degradation, bank erosion, or toe erosion; and load added to the top of the streambank such as adding spoil.

A slope is stable as long as the internal forces in the bank soils resisting failure exceed those causing failure. Computerized analyses are available to enable engineers to evaluate how changed conditions can impact this ratio of forces. The ratio of the resisting force to the causative or driving forces is usually termed the factor of safety of a slope.

$$FS = \frac{\sum \text{RESISTING Forces}}{\sum \text{DRIVING Forces}}$$

Analyses compute these forces for assumed or known potential failure surfaces using parameters to represent soils in the slope and ground water conditions. If the factor of safety is greater than 1.0, a failure is not predicted. In existing failed slopes, analyses are conducted to determine what changes can be made to increase the factor of safety to a desirable value. A

Figure TS14A–2 Empirical correlation between effective phi angle (ϕ) and plasticity index (PI) from triaxial tests on normally consolidated clays



factor of safety of at least 1.3 is ordinarily considered desirable.

Resisting forces include the frictional resistance of soil particles along a potential failure surface, cohesive forces if the soil contains significant clay, and the passive resistance of the weight of soil at the toe of the slope, if the slope is not vertical. Driving forces that cause failure consist mainly of the gravity forces of the soil in the slope above the center of rotation, together with any seepage forces present. Conditions that may change in a stable slope to create instability were previously described.

An example of a change in slope geometry is removal of the toe of the slope by streamflow. This removal of soil at the toe of the slope reduces the gravity forces resisting failure and may cause the factor of safety of the slope to be reduced to less than 1.0. Slope failures normally occur when the factor of safety is less than 1.0. This type of change in the geometry of the slope is probably responsible for more slope stability failures in streambanks than any other single cause. Figure TS14A–3 shows a factor of safety computation for a simple example slope before and after toe erosion. The eroded toe of the slope reduces the forces resisting failure so that the computed factor of safety changes from 1.1 to less than 1.0, and a failure is predicted. Repeated occurrences are common in this scenario. After a slope failure occurs from erosion of the toe, the failed material at the bottom of the slope can be subsequently eroded and the process repeats itself, with the top width of the channel increasing at each occurrence. This process is common in curves of streams, where the erosive attack at the toe of the slope is particularly severe.

Figure TS14A–4 shows a slope where erosion of the toe has caused slope instability. In figure TS14A–4(a), the overall slope is seen with erosion that occurred at the toe of the slope following a large runoff event. Figure TS14A–4(b) shows the effect of the slope failure at the top of the slope, and figure TS14A–4(c) shows the middle of the failed slope area. Other ways that slope geometry or conditions may change, resulting in instability, include:

- A change in the geometry of the slope may occur when the streambed lowers or degrades due to the instability of the stream system. The increased height of the slope and the oversteepening that occur may cause the factor of safety to be reduced to below 1.0.
- A load may be added to the bank soils at the top of the slope. This additional load may be from construction or the additional weight of soil or rock spoil. This added load may increase the forces acting with gravity to cause a slope failure. Examples of added load are dikes added for flood protection.
- The potentiometric surface (water table) may become elevated in the bank soils after prolonged high rainfall events, changing moist soils to a saturated condition. Saturation usually substantially reduces the shear strength of soils and increases their weight. This may cause the factor of safety to become less than 1.0.

Figure TS14A–3 Factor of safety computed before and after erosion of toe of slope: (a) before, FS=1.107; (b) after, FS=0.99

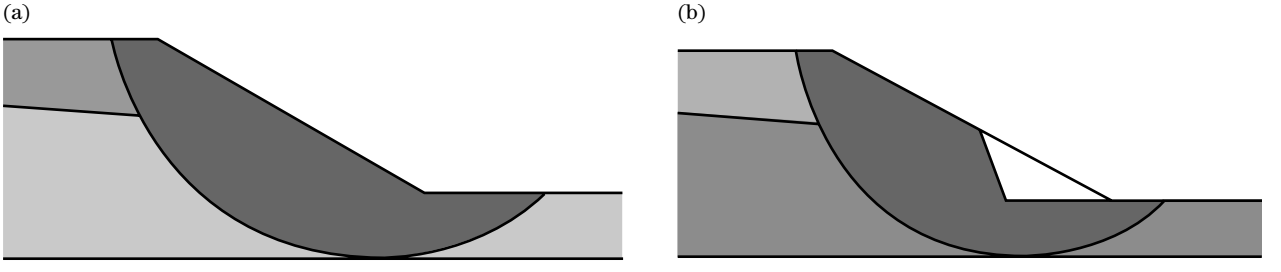


Figure TS14A–4 Failed streambank in Tarboro, NC, following high channel flows following Hurricane Floyd. Erosion of the toe caused bank instability in the slope. Measures to protect the toe of the slope are essential, in addition to assuring stability of the system.

(a)

(b)

(c)

- Soils in the bank may become saturated from prolonged storage of water in the stream channel. When the water level in the stream recedes, the saturated zone of soils may then have a reduced factor of safety from the increased weight of the soils and the resulting lower shear strength. This condition is sometimes termed a drawdown condition. Its severity is a function of the time that the water level remains high, charging the banks with infiltrated water; the permeability of the bank soils; and the rate at which drawdown occurs. Banks of high clay content soils are subject to failure and collapse under rapid drawdown after prolonged high flows.
- The nature of the soils in the slope may change over time. This may occur from weathering of minerals in the soil, development of a desiccated structure in clays, an opening of a slickensided structure from stress relief, and other causes. The phenomena of desiccated clays and how they affect the stability of streambanks is described in detail in the following sections.

Evaluation of streambank characteristics contributing to streambank failure

NEH654.03 describes how geology, tectonic history, climate, surficial processes, and time determine the types of landscapes and streams. In many landscapes, the streams reflect a continuum of the same processes, such as downcutting, erosion, and sedimentation, over a long time period. Materials from certain geologic processes and landscape locations can have higher streambank stability than others. Glacial till and loess are more stable in streambanks than sediments deposited by other geomorphic processes such as materials deposited by braided steams. Peat, formed in a lake or marsh, may form a vertical streambank if the peat is not layered with other materials. A boulder or cobble streambed and streambank will be more stable than a stream in a finer textured material because of the higher resistance provided by the coarse textured material.

The side slope (cotangent) of the streambank is an important factor in the probability of a potential failure. The steeper a streambank, the higher the probability of a slope failure occurring. The extreme condition is an overhanging slope. Overhanging conditions can only occur in streambank materials which have cementation, plant roots, or unusual temporary stabilizing forces such as capillary stresses. Overhanging slopes are inherently unstable and can fail with only slight changes in the bank conditions.

Ground water flow emerging from the surface of a streambank contributes to reducing the stability of the streambank. This topic is described in more detail in a following section. Streambank height is often a reflection of stream type (Rosgen stream classification F and G channels versus E and C channels). The probability of streambank instability is inversely proportional to streambank height. If two streambanks of different heights have the same soil type, the higher streambank will have more potential for rotational failure.

This is the condition reflected by the downcutting and widening in the channel evolution model (CEM) (Schumm, Harvey, and Watson 1981, 1984), Type III, where the critical bank height exceeds the stable bank height ($h_c > h$). Density of roots in the streambank can also be a factor in streambank stability. The reinforcement of dense mats of roots may reduce the probability of some failures to occur. The effectiveness of the root mass reinforcement varies by plant species and whether the plants are alive or dead.

Typical slope instability problems and behavior of common soil types

Bank instability problems and slope failures may have many shapes. Failures may appear shallow and only involve surficial sloughing. Some failures involve deep-seated rotational failures. Failures involving limited thin seams of weak soil may be wedge or block-shaped. The appearance of a slope failure often provides clues to the type of soil involved in the failure and possible contributing factors in the failure.

Figure TS14A–5 shows a shallow slide occurring from a zone of saturated sand sloughing from seepage forces. Figure TS14A–6 shows the results of a deep-seated rotational failure in clayey soils. A later section in the document describes this type of failure in detail.

Sands and gravels in streambank slopes typically fail with a shallow sloughing type failure. These failures occur when the bank soils are subjected to oversteepening by toe erosion, or when subjected to seepage forces. The phenomenon of sapping refers to the sloughing of saturated zones of sand below the water table in the exposed streambanks. Slope failures in soils that have clay fines with significant plasticity typically have a circular appearance and are relatively deep-seated. These types of slides are usually precipitated by downcutting of the streambed. Slides of this type may be extensive and affect property some distance from the stream.

Solutions to typical slope instability problems

Instability in the side slopes of streambanks can be prevented or repaired after it occurs. The following outlines preventative methods and methods used to remediate problems, using the same outline as above for the basic causes.

One approach to prevent problems caused by erosion of the streambank toe is to protect it from attack by flowing water. A wide variety of methods can be used including:

- riprap and other armoring techniques including cellular blocks and similar hard armor methods
- soil bioengineering methods such as crib walls at the toe of the slope
- realignment of the channel to reduce scour
- barbs and other methods for deflecting flows away from the toe of the slope

Figures TS14A–7, TS14A–8, and TS14A–9 show examples of methods used to protect the toe of slopes in a project in the city of Austin, Texas. Figure TS14A–10 shows the use of stream barbs to prevent erosion of the toe of a slope. Protecting the toe of a repaired slope from subsequent erosion is essential to the success of most streambank stabilization projects.

Figure TS14A–5 Shallow sloughing failure in zone of saturated sand

Figure TS14A–6 Typical slope stability failure in clay soil type. Failure is a low-radius, deep rotational failure. (*Photo credit City of Fargo, ND*)

Figure TS14A–7 Before and after pictures of project, Tannehill Branch Givens Park in Austin, TX. Note method for protecting toe of slope from erosion. (*Photo courtesy of City of Austin, TX*)

(a)

(b)

Figure TS14A–8 Before and after pictures of project, West Bouldin Creek at South 6th Street in Austin, TX. Slope undercut and oversteepened by erosion of toe, which led to sloughing and bank failures. Note method for protecting toe of slope from erosion. (*Photo courtesy of City of Austin, TX*)

(a)

(b)

Figure TS14A–9 Before and after pictures of project, Shoal Creek in Austin, TX. The toe of the slope was protected with riprap and the bank shaped above the protected toe. (*Photo courtesy of City of Austin, TX*)

(a)

(b)

Figure TS14A–10 Barb structures installed to protect toe of slope from erosion

Figure TS14A–11 shows a project where the toe of the slope was protected using both riprap and a sheet pile wall. In many circumstances, rigid boundary constraints, improvements, or other obstacles at the top of the slope do not allow the slope to be flattened. Consequently, constructing a vertical feature at the toe of the slope may be needed.

Another way of reconstructing a slope where limited right of way occurs is the use of gabion baskets to protect the toe of the slope (fig. TS14A–12).

Measures to increase the stability of slopes may include the use of geosynthetic reinforcement, which enables the slope to be reconstructed to a steeper

Figure TS14A–11 Sheet pile wall at toe of slope in Tarboro, NC. Vegetation will be established above wall. Note erosion control fabric.

(a)

(b)

Figure TS14A–12 Gabion wall constructed at toe of slope enabled reconstruction of failed slope where right-of-way limitations prevented flattening of the upper slope and use of other conventional toe protection measures at the toe

angle than would otherwise be possible. Examples of geosynthetic reinforcement include the use of geogrids and geocell products. These products are described in NEH654 TS14D. Figure TS14A–13 illustrates a project where the toe of the slope was protected by armoring, and the slope was reconstructed with reinforced soil lifts.

Figure TS14A–13 shows the reconstruction of the slope at the West Bouldin Creek at South 6th Street project. Figure TS14A–13(a) shows the site in the initial stages of construction. The toe of the slope has been excavated in preparation for installing limestone boulder armor and beginning placement of geocell used to form the reinforced fill for the slope. Figure TS14A–13(b) shows the layers of geocell and gravel that were used to form the reconstructed slope. The geocell is a strong plastic honeycomb type of product that allows the slope to be rebuilt to a nearly vertical configuration at a much lower cost than a retaining wall.

Figure TS14A–14(a) shows the West Bouldin Creek at South 6th Street Austin project during last stages of construction, and figure TS14A–14(b) shows the project after completion of the backfill and establishment of vegetation. The use of soil in the filled geocells

Figure TS14A–13 Reinforced fill at toe of slope in Austin, TX. Note large stones used to protect reinforced fill from erosive forces in stream.

(a)

(b)

Figure TS14A–14 West Bouldin Creek at South 6th Street, Austin, TX, project during last stages of construction

(a)

(b)

allows vegetation to grow and mask the construction. Geocells allow very steep slopes to be used in the reconstruction. These six photographs show a project to stabilize unstable streambanks at the Shoal Creek Project, City of Austin, Texas (fig. TS14A–15).

Figure TS14A–15 shows other slope stabilization projects where reinforced fill protected by rock armor riprap placed at the toe of the slope were employed. Figure TS14A–15(e) shows geogrids, a heavy lattice of very strong plastic placed in layers in the granular fill used to reconstruct the slope. Figure TS14A–15(f) shows the completed project.

Figure TS14A–16 shows construction of a geotextile reinforced slope with large derrick stone armoring at toe of slope. The reinforcement provided by the geotextile layers in the backfill allowed reconstructing the slope to a steep angle to accommodate a road at the top of the slope. The geotextile used was a heavy weight geosynthetic product that is anchored to the rock anchor wall and extends back into the granular backfill used to reconstruct the slope. The finished slope is vegetated and requires little maintenance. The rock toe wall is required to protect the fabric wrapped backfill from the abrasive forces of the water and debris in the channel. The cost of the rock toe wall is more than half the cost of the total project. Figure TS14A–16(a) shows the placement of the large rocks after the bottom lift of geosynthetic fabric and backfill have been placed. Figure TS14A–16(b) shows the geotextile wrapped over the rock toe wall, while the next layer of backfill is placed. The fabric will be folded from right to left over the layer of compacted fill to form a layer of reinforcement within the backfill. The use of geosynthetics in stream restoration is addressed in more detail in NEH654 TS14D.

Information required for slope stability evaluation of slopes

Performing detailed slope stability evaluations is a highly specialized endeavor. Evaluations should be performed by personnel who are competent in the techniques of slope stability analysis and have the experience and tools to do the analyses. For analyses to be worthwhile, the shear strength parameters used in the analyses must be appropriate for the conditions, and they must reflect the properties of the soils in the streambank. Soil properties may be estimated, but preferably, samples are obtained of representative horizons in the soil profile and tested in a geotechnical laboratory. Obtaining information on the soil horizons in a streambank from surface exposures may be helpful, but often, geotechnical investigations involving drill holes and sampling followed by laboratory testing may be needed.

Often, correctly classifying the soil types in the bank, identifying the ground water conditions, and characterizing the condition of the soils provide most of the needed information for a preliminary evaluation of stability. Information on water table conditions may be gathered by hand auger holes that are left open for several days and monitored with a tape measure or other sounding device. More sophisticated measurements using observation wells installed along the stream may be useful for monitoring changes in water levels over a time period that involves several seasons.

The following sections describe unique classes of commonly occurring slope stability problems that may be evaluated by methods that do not require computer analyses. When computerized analyses are required, specialists experienced in their use and application should be involved. The purpose of including discussions on these unique problems in this document is to enable field personnel to recognize these common situations and what remedial measures are appropriate.

Sloughing and piping/sapping of streambanks

Slope failures can result where silts and sands with slight or no plasticity occur in the lower portion of a streambank. If horizons of this type of soil become saturated and seepage occurs at the streambank face, the soils can fail in several ways. In one mode of failure, particles may be detached and removed by the seepage exiting the bank. This can form an overhanging condition. The overhanging portion of the slope is prone to failure with any additional stress. The process by which these saturated silts and sands fail has been termed piping or sapping. For piping to occur, soils overlying the layer that is sloughing must

Figure TS14A–15 Other slope stabilization projects

(a)

(b)

(c)

(d)

(e)

(f)

Figure TS14A–16 Construction of a geotextile reinforced slope with large derrick stone armoring at toe of slope

(a)

(b)

(c)

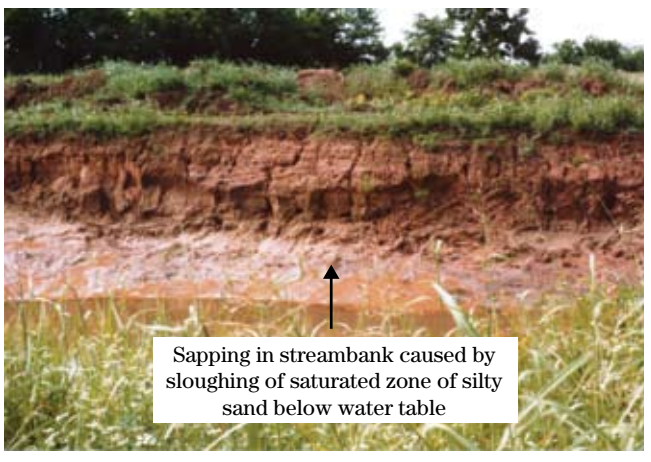
(d)

be able to form a roof. In a related mode of failure, the saturated bank in this zone of low plasticity soils slumps or sloughs under the seepage forces. Sloughing of the lower banks can undermine overlying zones that are stable on a steeper slope. Figure TS14A–17 shows sloughing in the lower saturated banks of an excavated stream.

Piping/sapping failures and sloughing failures may occur quickly as the water table rises next to a stream or if excavation or degradation lowers the stream bottom. For instance, if a stream is degraded several feet by erosion and the lower banks of the stream consist of cohesionless sands with a high water table in the banks, a flow failure of the saturated sands usually occurs at almost the same time as the bed elevation is lowered. Sloughing of banks can also occur when flood storage in the stream saturates the banks, and the water level in the stream suddenly recedes. The saturated banks may fail in an infinite slope type of failure.

Soil bioengineering methods usually cannot be established soon enough to prevent these types of failure. After a sapping failure has already occurred, soil bioengineering techniques may help to stabilize the toe area and prevent subsequent erosion of the toe. Protecting the toe of the slope from erosion and preventing future downcutting of the stream bottom are essential to minimizing future sapping problems.

Figure TS14A–17 Sloughing of saturated low plasticity zone in lower streambanks caused by seepage forces



High ground water levels in the streambank soils may continue to cause bank sloughing, even if the toe is protected.

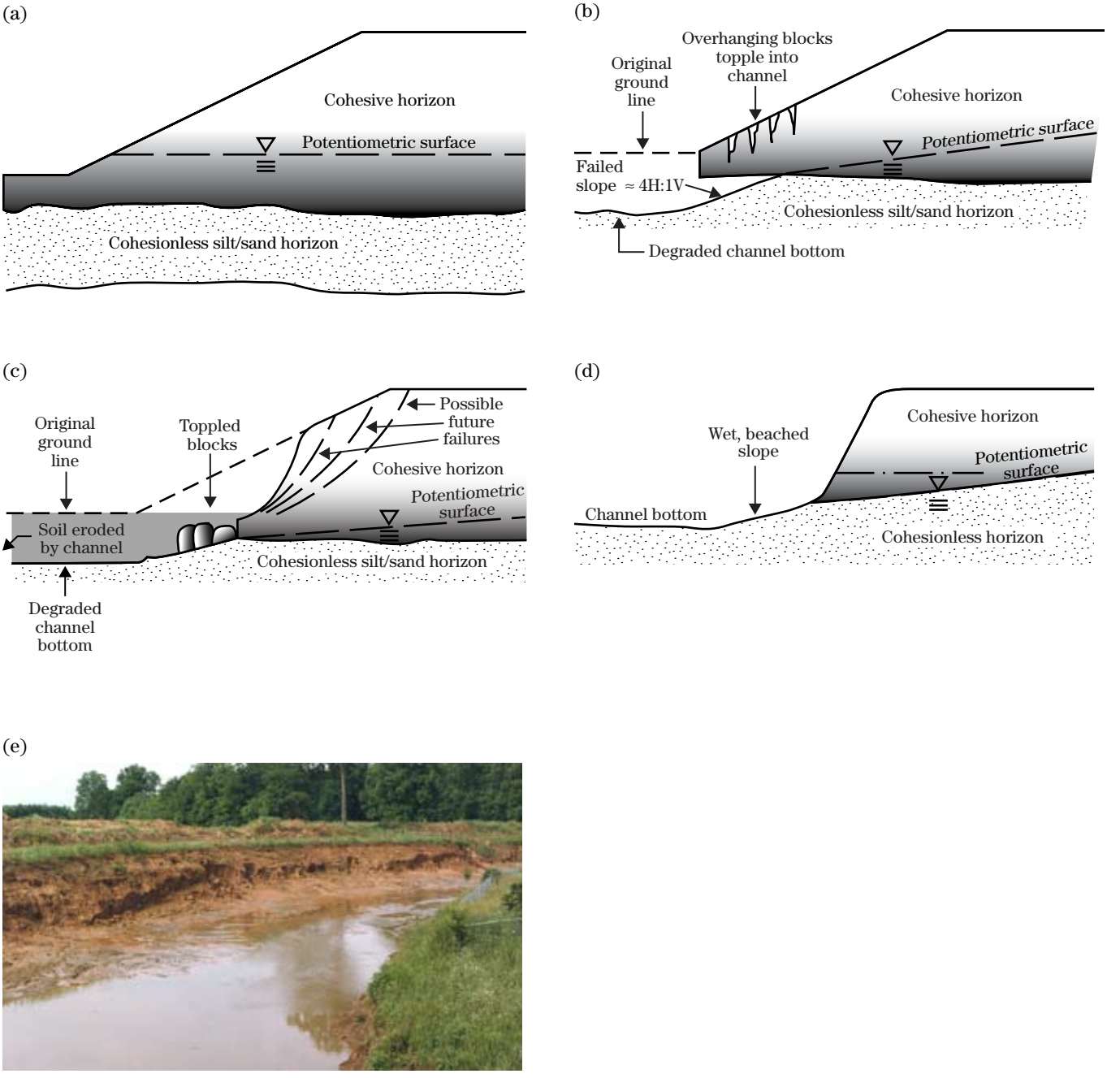
The stability of slopes of low plasticity soils is analyzed using a set of equations that are termed infinite slope equations. Commonly, stable saturated slopes for low plasticity soils range from about 2.5H:1V to 3.5H:1V. Some stream slopes may be stable in these types of soils in a moist condition on slopes as steep as 1H:1V because the surface tension forces in moist sands and silts resist failure. If the soils in these steeper banks are subsequently saturated, sloughing of the soils can occur.

Figure TS14A–18 illustrates the progression of this sloughing type of failure mechanism in a streambank. Figure TS14A–18(a) shows an initially stable condition, prior to stream degrading into a horizon of soils susceptible to sloughing and sapping/piping. Figure TS14A–18(b) illustrates how if the stream bottom degrades into an underlying horizon of low plasticity silty sands or silts, and a high ground water table exists, the seepage forces in the newly exposed sand horizon cause instability. Figure TS14A–18(c) shows that as further streamflows occur, the toppled blocks are eroded, and the sloughing process repeats itself. The top banks of the stream continue to recede unless the toe of the slope in the cohesionless soils is protected. Figure TS14A–18(d) shows the typical appearance of a slope where sloughing has occurred. Figure TS14A–18(e) shows the typical appearance of a slope where sloughing has occurred.

Piping/sapping failures are most common in unconsolidated alluvium. Because alluvial soils are layered, cleaner lenses of sand or silt may occur between lenses of lower permeability clays. If seepage forces are concentrated in these cleaner soil lenses, the problem may be worse than it would be in a more homogeneous soil profile. When a horizon of saturated, cohesionless soil below the water table saps or flows from the slope, the lower part of the slope flattens and can cause an overhang in the uppermost soils in the streambank.

These overlying soils then fail by toppling into the stream, as shown in figure TS14A–18(c). If erosion of the lower slopes is occurring or if the streambed is degrading, the process repeats. This results in substantial

Figure TS14A–18 Progression of sloughing type of failure mechanism in a streambank



widening of the stream top width. Soil bioengineering techniques may be effective in protecting the toe of these failed slopes from erosion, preventing subsequent failures.

Piping/sapping problems are likely in a stream system that is degrading more quickly than the ground water level can lower by drainage where the bank soils are susceptible. If the streambank soils were able to drain at the same time the stream bottom degraded, the problem would not occur, because the streambanks would remain in a moist, rather than saturated condition.

Piping/sapping failures may be initiated or accelerated by high rainfall, which recharges the water table in the streambank soils adjacent to the stream. Ground water flow may also be affected by nearby larger bodies of water. Ponding of water at the top of the streambank, especially where levees have been constructed, can also contribute to seepage pressures. Surface flow should be diverted and outletted into the stream away from a streambank area that is susceptible to this problem.

Soil bioengineering methods alone are ineffective in addressing the seepage flow in permeable sand deposits because the flow quantities probably exceed the evapotranspiration ability of plants. This is one situation that will require granular filters in combination with a gravel/rock face to outlet the seepage and prevent piping of the bank and bed soils. Soil bioengineering may be integrated with granular filters to stabilize the upper banks and to reinforce the granular filter layers.

Piping/sapping and sloughing may also occur when a stream has been full of water during a prolonged flood stage, and later the water in the stream lowers rapidly. The water in the stream stored during the higher stage may saturate sands and silts in the streambanks, and the saturated soils may fail by sloughing following the drawdown of the stream water. The likelihood of drawdown slope failures in a cohesionless soil horizon depends on how long the water is stored in the stream to saturate the slopes, how quickly the stream storage is emptied, and the permeability of the soil horizons in question.

Recognizing the problem

Recognizing a situation where piping/sapping has occurred or is occurring is easy if it is occurring at the time of the inspection. The shape of the streambanks and the appearance of soils in the banks are clues. The saturated zone at the toe of the streambank will be much flatter than the overlying horizons. Free water is visible as it exits the slope. Figure TS14A–17 illustrates the typical appearance of a streambank where sloughing has occurred. At the point where seepage emerges from the bank, the low plasticity soils that are saturated are on a very flat slope, usually in the range of 3H:1V to 4.5H:1V. The streambank is in a temporarily stable condition. The bank will likely remain stable at this condition, until the toe of the slope is again eroded, ground water levels rise, or the stream bottom degrades.

If a piping/sapping failure is not active when it is inspected, recognizing the problem may be more difficult. Vegetation may have become established at the toe of the slope on the soils that have previously sloughed, obscuring clues of the earlier failure. For more information on diagnosis, see the articles by Hagerty (1991). Geologic and geotechnical investigations that determine whether these soil types occur in the streambanks, the elevations of seasonal high water tables, and other factors are important to recognize these potential problems.

Traditional solutions

The most common method for solving piping/sapping problems is to use a layer of graded filter sand or sand/gravel mixture placed on the saturated zone of soil in the streambank. The filter material is designed to be more permeable than the bank soils, but fine enough to filter particles and prevent them from moving through the sand. The filter material is often protected from the erosive forces in the flowing stream with a layer of riprap, manufactured paving blocks, or other suitable methods. Usually, a geotextile separator is used between the filter layer and the overlying cover of riprap or paving blocks. This is described in more detail in NEH654 TS14K.

Some designers use a geotextile placed directly on the bank, covered by riprap or other erosion resistant material, without placing a filter layer. This type of de-

sign is usually less expensive, but may not be suitable for very fine silty soils. In a few situations, interceptor drains are installed away from the streambanks, parallel to the stream, to intercept ground water flow and prevent it from exiting at the slope face. Soil bioengineering methods may be effective in stabilizing a failed site if the conditions that caused the failure will most likely not recur with the same severity at that site.

Soil bioengineering solutions to piping/ sapping problems

Soil bioengineering techniques are most useful in protecting lower silt/sand slopes after they have achieved a stable angle of repose. Soil bioengineering may also be effective in protecting upper cohesive soil horizons after the slopes have been shaped to a stable configuration. Soil bioengineering measures alone will not prevent a sapping/piping failure from occurring where conditions change rapidly to cause the failures. Soil bioengineering may be useful in transpiring excess water from streambanks, but vegetation will probably not be able to transpire the quantities of water available from high ground water conditions in permeable soils or from the saturation of banks, which occurs during flood staging of the watercourse.

Shallow slope failures in blocky structured (fissured) highly plastic clays

Highly plastic clays with a fissured or blocky structure can also cause severe stability problems in streambanks. The blocky structure of these soils results from desiccation that occurred after the soils were originally deposited. Repeated drying and wetting cycles cause a structure that is sometimes termed a slickensided structure.

Figure TS14A–19 shows photographs of highly plastic clays with slickensides and blocky structure. Shallow surface slides commonly occur in these soils where streams have been modified or the stream system is not in equilibrium and bed degradation is occurring.

Figure TS14A–19 Slickensides in exposure of clay soil

Figure TS14A–20 shows a slope failure on a streambank that occurred when the stream was deepened about 4 feet to increase the flow capacity and blocky structured clays occurred in the streambanks. Root reinforcement of large trees was inadequate to resist the large forces active in this type of failure. Note that large trees were displaced by the failure.

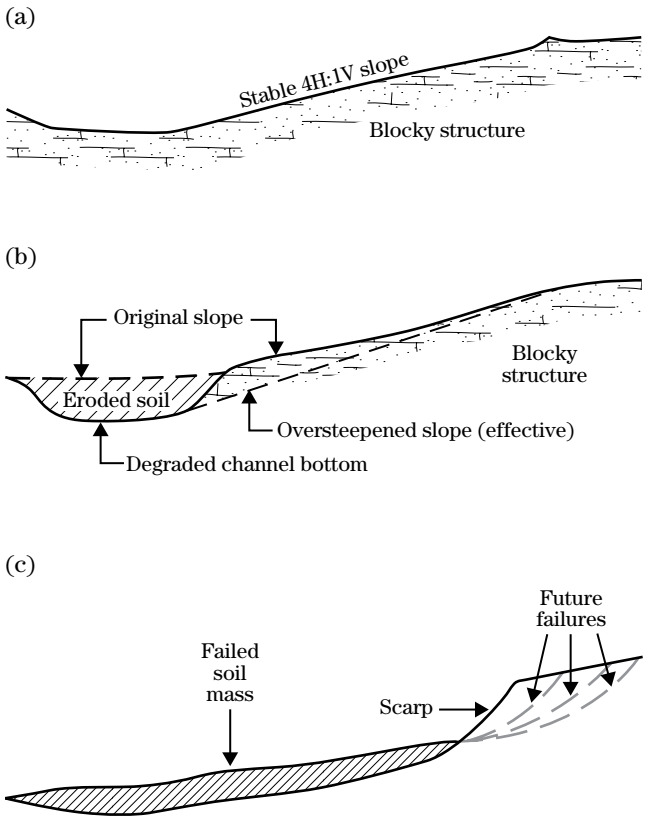
These types of failures have a shallow circular shape; are about 3 to 4 feet deep, measured normal to the slope surface; and frequently occur progressively. Larger failures follow small initial slides, if no corrective measures are taken. The scarp face (near-vertical surface at the top of the slide) may extend past the crest of the stream slope, but usually only after a series of failures has occurred at the same location.

This type of slope failure often recurs if the failed material that has sloughed into the stream is eroded and the slope is again oversteepened. Protecting the toe area with soil bioengineering measures may reduce the severity of future failures, provided the streambed is not degrading. Because these plastic clays are seeking a stable slope that is usually very flat, the upper slope surfaces that remain after a failure must also be stabilized by flattening and using vegetation.

Figure TS14A–20 Stream slope that failed when the stream was deepened by about 4 feet to increase the flow capacity

The series of sketches in figure TS14A–21 show the progression of events that can result in bank instability. Figure TS14A–21(a) shows an initially stable condition prior to stream degrading into a horizon of soils that are susceptible to slope failures because of the blocky soil structure. Figure TS14A–21(b) shows streambed loss or degradation causes an effective steepening of the streamside slopes. The blocky structured clays are subject to slope failures when the stable condition is disturbed.

Figure TS14A–21 Progression of events that can result in bank instability



Terzaghi and Peck (1967) describe slope failures in excavated slopes in highly plastic clay soils as follows. Note that this description refers to excavations, but the same principles apply to a slope that is deepened or oversteepened either by human activities or degradation of the streambed.

Almost every stiff clay is weakened by a network of hair cracks or slickensides. If the surface of weakness subdivides the clay into small fragments 1 inch or less in size, a slope may become unstable during construction or shortly thereafter. On the other hand, if the spacing of the joints is greater, failure may not occur until many years after the cut is made...

If the spacing of the joints in a clay is greater than several inches, slopes may remain stable for many years or even decades after the cut is made. The lapse of time between the excavation of the cut and the failure of the slope indicates a gradual loss of the strength of the soil.

Before excavation, the clay is very rigid, and the fissures are completely closed. The reduction of stress during excavation causes an expansion of the clay, and some of the fissures open. Water then enters and softens the clay adjoining these fissures. Unequal swelling produces new fissures until the larger chunks disintegrate, and the mass is transformed into a soft matrix containing hard cores. A slide occurs as soon as the shearing resistance of the weakened clay becomes too small to counteract the forces of gravity.

Most slides of this type occur along toe circles involving a relatively shallow body of soil, because the shearing resistance of the clay increases rapidly with increasing distance below the exposed surface. The water seems to cause only the deterioration of the clay structure; seepage pressures appear to be of no consequence.

After a slide occurs, the material underlying the newly exposed slide begins to soften, and the process continues until another slide occurs. The process does not stop until the slope angle becomes compatible with the softest consistency the clay can acquire… Thus the slopes become gentler...

In summary, shallow slope failures on streambanks composed of plastic clay soils are attributed to a network of fissures and blocky structures that develop due to alternating drying and wetting cycles. The desiccation of the clays may be recent, or it may have occurred long ago when the clays were originally deposited in an alluvial flood plain. Blocky structured clays may also occur very deep in older alluvial profiles and in glaciated areas. Instability is common after heavy rains or from elevated ground water. Water stored in the stream in a storm may be another source. The climatic regime in which these sites exist affects the severity of and time required to develop the fissured structure.

Predicting stable slopes in plastic clays

A reliable analytical method for predicting a stable slope for highly plastic clays is not available. The most reliable method may be empirical examination of stable natural slopes in the same materials near the site. Nearby natural slopes that have not been significantly modified for at least 30 years should be studied. At one site studied by NRCS engineers, the cotangent of natural slopes was from 4H:1V to 7H:1V where the highly plastic clays were found. Failures occurred when the slopes were shaped to 3H:1V during enlargement of the stream cross section.

The strength of highly plastic fissured soils for first-time failures may be modeled using the fully softened condition as recommended by some authors including Stark and Hisham (1997). The fully softened strength of these soils depends primarily on the plasticity of the soils. These problems are most severe for CH soils with plasticity index (PI) values above 40. Soils with PIs greater than 80 are stable only on very flat slopes. Available laboratory testing techniques are inadequate to model the fissured structure in these soils. A conservative design assumes that the effective cohesion value in these blocky structured soils is zero, and the design is based only on the fully softened phi angle. The fully softened phi angle is independent of the original strength of the clay and such factors as water content and liquidity index (fig. TS14A–22). It seems to depend only on the size, shape, and mineralogical composition of the constituent particles and the effective normal stress (Stark and Hisham 1997). Fully softened phi angles are generally assumed to be in the range of 14 to 19 degrees. On this basis, clay soils

with PI values of about 30 to 40 are stable on slopes of about 3H:1V, while clays with high plasticity indices (greater than about 80) may require slopes as flat as 6.5H:1V for long-term stability.

Analyses where the effective cohesion is significant

In silty, clayey, coarse-grained soils, a significant cohesion property may be present in effective stress parameters. For this assumption, computerized slope stability analyses can be used to calculate factors of safety by various methods such as the method of slices. These types of computer analyses are not appropriate for zero-cohesion soils because the critical failure surfaces are very shallow on the slope face with factors of safety approaching those obtained using the infinite slope equations.

The equation for slightly cohesive sands with seepage parallel to the slope (eq. TS14A–1) is shown in figure TS14A–22, after an equation presented in Lambe and Whitman (1969):

$$FS = \frac{\bar{c} \times \gamma_b z \times \cos^2\theta \times \tan\bar{\varphi}}{\gamma_{sat} \times H \times \sin\theta\cos\theta} \qquad \text{(eq. TS14A–1)}$$

Recognizing the problem

This problem is identifiable by visual examination and textural evaluation of the soils in the streambanks. These failures occur usually in highly plastic soils classifying as CH, or fat clays, in the USCS. In the soil survey system of the NRCS, these soils are often classified as vertisols and are identified in the soil survey as having high shrink-swell potential. The most problematic soils have liquid limit values greater than 60 and PI values greater than 40. The severity of the problem is directly proportional to the liquid limit and plasticity index of the soils.

The soils have an observable strongly blocky structure in an exposed face. The blocks of clay may be from one quarter to three-quarter inch in dimension. Figure TS14A–23 shows an exposure in a blocky clay soil. Other photographs show slickensides that are present in many such deposits. Slickensides develop from repeated wetting and drying cycles in clays.

Figure TS14A–22 Infinite slope equation for slope with small cohesion value

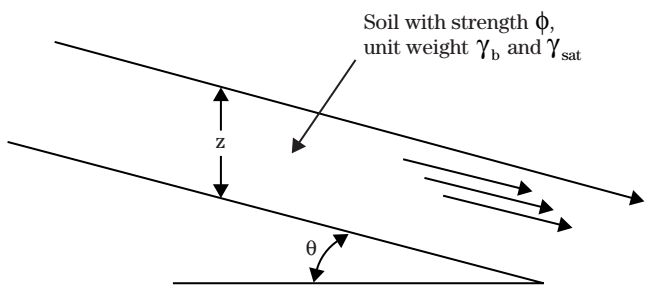


Figure TS14A–23 Blocky structured clay in streambank exposure

The failures usually occur in a progressive manner. The first evidence of a failure may be a small roll of soil at the toe of the slope. The final failure configuration is circular arc-shaped with a flat radius, and it encompasses at least two-thirds of the slope length. The slides are usually no more than 3 or 4 feet deep, measured normal to the slope. Some slides may be deeper if previous sliding has occurred at the site, with depths normal to the slope of up to 8 feet. Slides are often triggered by a high-intensity rainfall event that closely follows a prolonged droughty period. Flood storage in the stream can also provide water to fill the cracks in the soil, causing failures when the stream empties.

Streambanks seek their stable angle of repose. In a stable system, slopes on natural streams in these soil types reach a slope of from 4H:1V to 7H:1V, depending on the clay content and plasticity of the bank soils. Failures with highly plastic clay soils most often occur in streams modified by man or where streambed degradation has occurred, and the oversteepened slopes fail. These failures may not occur until many years following oversteepening of the streambanks.

Soil bioengineering solutions to shallow slope failures in blocky clays

Soil bioengineering techniques that develop a root system capable of reinforcing the streambanks about 4 feet normal to the outer slope may provide added protection against these types of failures. A disadvantage is that transpiration from vegetation may aggravate the drying and shrinkage crack development of the soils. Litter and shade provided by vegetation may deter the drying of the clays from direct sunlight. The large forces that result from the weight of a saturated clay mass are difficult to overcome solely with root mass reinforcement.

Some flattening of the slopes or replacement of the plastic clays with less plastic soils is required, in addition to soil bioengineering techniques, for these worst case situations. Vegetation may be effective in stabilizing the toe area of a failed slope, preventing it from being removed by subsequent erosion from flowing water in the stream. This allows stabilization of the upper slopes to be effective.

Soil bioengineering techniques, such as supplementing cribwalls with vegetation, brush layering, and similar measures, can be useful, but will probably only be long lasting if the more plastic clay soils are replaced behind the walls, and soils are shaped to a flat slope on the streambank above the reinforced structure.

Traditional solutions

As with most problems related to slope instability, stabilizing the toe of the slope and ensuring that the stream grade will not degrade further are essential to the long-term success of any treatment of the problem. The most common methods for treating stream slopes in highly plastic clays that have failed may be summarized as follows:

- Flattening the failed slope to a predicted stable slope is perhaps the most positive solution. This option requires considerable right-of-way if the slopes must be flattened significantly. For example, consider a 15-foot-deep stream that is on a slope of 3H:1V. If the slope needs to be flattened to about 5.5H:1V to be stable, the top width is increased by 37.5 feet.
- Installing gravel trenches in the slope has been an effective treatment. The USACE used this type of repair successfully on a slope failure on the Sunflower River in Mississippi. Sills and Fleming (1992) describe a similar repair. This method is relatively economical, but has not been used widely, and designers may not be as confident in the results.
- Another method of repairing these slides is covering the highly plastic soils in the slide area with soils having more favorable properties such as silty sands, gravels, or riprap.
- Highly plastic clays may be modified by incorporating either hydrated or quick lime. About 5 percent by dry weight is added to the soils to treat them. This alternative is expensive because the lime is costly, and the construction procedures to apply and mix the lime into the streambanks are expensive.

Dispersive clays

Dispersive clays are different in their chemical composition than ordinary, erosion-resistant clays. Dispersive clay soils are far more erodible than ordinary clays because the interparticle attraction is much reduced by imbalanced electro-chemical bonds. Streamside slopes in dispersive clay deposits often develop a highly rilled appearance, also showing a phenomenon referred to as jugging. Dispersive clays are described in detail in Soil Mechanics Note 13, Dispersive Clays.

Recognizing the problem

Dispersive clay slopes often are severely rilled. These rills often develop in a short time. Jugholes are an ideal diagnostic tool for dispersive clays. These features develop when a drying crack in the exposed soil provides an entrance for precipitation that can then erode the sides of the wall. This internal erosion of the crack results in subterranean cavities often termed jugholes. Figure TS14A–24 illustrates the special appearance of dispersive clays.

Figure TS14A–24 Photographs of exposure of dispersive clays in streamside slopes in TX and OK

(a)

(b)

(c)

Another diagnostic tool is that vegetation has little effect on the severity of the erosion of dispersive clays. Vegetation is less effective in reducing erosion on dispersive clays because the particles that are eroding are colloidal in size. They are much too fine to filter with vegetation, and they can go into suspension in essentially standing water.

An excellent field test for dispersion is the crumb test. ASTM Standard Test Method D6572 covers methods for performing the test. The test requires a minimum of equipment and is excellent for screening purposes to determine if dispersive clays are likely present.

Crumb test reactions

A common field test for dispersive clays is the crumb test, a test for dispersive soils (fig TS14A–25). A small clump of the soil (about a half-inch cube) is placed in a cocktail glass that has about an inch of distilled water, and observed for at least an hour. A rapid formation of a cloud around the soil indicates that it is dispersive. The observed reaction is typically given a rating from the criteria listed.

1—No reaction, water in glass remains clear. Ignore any slaking of clod; examine only for turbidity.

2—Cloud immediately around clod. A hint of a cloud occurs very near the clod. However, it does not spread significantly away from the clod.

3—A colloidal cloud spreads a considerable distance from the clod. However, it does not spread completely to meet the opposite side of the glass.

4—Cloud spreads around bottom and may cover bottom. The colloidal cloud may be so extensive that the whole bottom of the glass is covered.

Reactions 3 and 4 are a very positive indicator of dispersive soil.

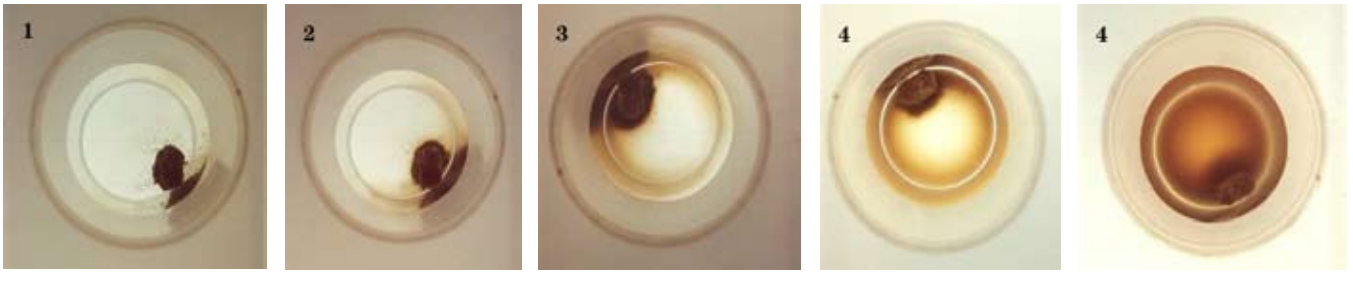
Soil bioengineering solutions to dispersive clay problems

Soil bioengineering techniques are not likely to be effective in preventing or curing dispersive clay erosion problems on stream slopes. Severe erosion has been observed on the best vegetated sites.

Traditional solutions

Several repair or preventive design measures have successfully been applied to dispersive clay sites. In the first method, the dispersive clays are covered with an insulating blanket to prevent the cracking of the clays that leads to more severe erosion. Blankets of sand/gravel/silts have been used with success. For silty sand or gravel blankets, the fines in the sand or gravel should be checked to assure they are not dispersive.

Figure TS14A–25 Typical reactions in crumb test, a test for dispersive soils



Another method for treating dispersive clays is to chemically alter the soils. Dispersive clays are predominated by sodium, and replacing the sodium ions with calcium, magnesium, or aluminum ions improves the interparticle attraction of the clays. Chemicals that have been successfully used include hydrated lime (fig. TS14A–26), quick lime, fly ash, alum, and gypsum. Refer to Soil Mechanics Note 13 for more detailed information.

Low plasticity sands and silts

The problems of bank sloughing in low plasticity sands and silts with a high ground water table were described previously in this chapter. This section provides additional description of the problem and analytical tools.

Analysis of slope stability in low plasticity sands and silts

A slope, such as the banks of a stream composed of sandy or silty soils with a low clay content, may slough if it is too steep. The slope that is stable for a given soil is studied with several equations termed infinite slope equations. The equations will be addressed in detail in the following sections. The stability of a stream slope in susceptible soil types is studied by computing a factor of safety. Designers always prefer that the factor of safety for this type of analysis be greater than 1.1.

Three equations are used to compute the factor of safety of the slope for three different assumed ground water flow conditions. The factor of safety of a slope depends on the following three factors: unit weights of soil and water, effective friction angle (phi angle), and the direction of seepage forces.

Figure TS14A–26 A streambank where highly dispersive clays were treated by applying and mixing in hydrated lime to the slope soils

Unit weights

The equations for computing factors of safety for infinite slope stability consider several possible conditions for a soil. Soil horizons above the water table are in a moist condition. The moist unit weight is computed as:

a. *Moist unit weight.* A moist unit weight value is computed using equation TS14A–2:

$$\gamma_{moist} = \gamma_{dry} \times \left(1 + \frac{w(\%)}{100}\right) \qquad \text{(eq. TS14A–2)}$$

The moist unit weight of soils requires an estimate or measurement of the soil's dry density and natural water content. In the absence of measurements, the dry unit weight of sands can be estimated from table TS14A–4.

The natural water content depends on the climatic regime, antecedent rainfall, and many other environmental factors. For most sands, the natural water content of moist soils is in the range of 8 to 20 percent.

Table TS14A–4 Estimated values for dry unit weight of sands

Relative density of sand	γ_d – Dry density (assumed), lb/ft^3
Loose	90
Medium	105
Dense	115

Example: Assume that it has been determined that the sands in a streambank horizon are loose in relative density, and the natural water content is about 12 percent. From table TS14A–4, one can assume that the soils have a dry unit weight of about 90 pounds per cubic foot. Compute the moist unit weight of the sand (eq. TS14A–3).

b. *Buoyant unit weight*. Several equations used to evaluate the stability for the infinite slope condition require a value for the buoyant unit weight of the soil. The buoyant unit weight is also called the submerged unit weight. Two parameters are used. First, the dry density of the soil is needed. It may be estimated from table TS14A–4 whether the approximate relative density is known. The second term needed is the specific gravity of the soil particles. The value for G_s is usually assumed to be 2.65 for most sands. This is the specific gravity of quartz, the predominant constituent of many sands. The buoyant unit weight of soil is computed using equation TS14A–4.

$$\gamma_{moist} = \gamma_{dry} \times \left(1 + \frac{w(\%)}{100}\right)$$

$$= 90.0 \text{ lb/ft}^3 \times \left(1 + \frac{12}{100}\right) \qquad \text{(eq. TS14A–3)}$$

$$= 90 \text{ lb/ft}^3 \times 1.12$$

$$= 100.8 \text{ lb/ft}^3$$

$$\gamma_{buoyant} = \frac{G_s - 1}{G_s} \times \gamma_{dry} \qquad \text{(eq. TS14A–4)}$$

Example: Assume that it is determined that the sands in a stream horizon are loose to medium in relative density, and the specific gravity of the sand is 2.65. From table TS14A–4, assume the soils have a dry unit weight of about 97.5 pounds per cubic foot. Compute the buoyant unit weight of the sand (eq. TS14A–5):

$$\gamma_{buoyant} = \frac{G_s - 1}{G_s} \times \gamma_{dry} = \frac{2.65 - 1}{2.65} \times 97.5 \text{ lb/ft}^3 = 60.7 \text{ lb/ft}^3 \qquad \text{(eq. TS14A–5)}$$

c. *Saturated unit weight*. The saturated unit weight of soil is simply the buoyant unit weight plus the unit weight of water, 62.4 pounds per cubic foot in the United States system of measurement. For the previous example, the soil would have a saturated unit weight of 60.7+62.4 = 123.1 pounds per cubic foot.

Effective phi angle

The other parameter used in these equations is the effective friction angle, also called the effective phi parameter, of the soil in the bank. The value may be estimated or measured in laboratory tests. The value generally varies from about 28 degrees to 40 degrees, depending on the relative density of the sand or silt. Table TS14A–5 may be used to estimate a numerical value for relative density from a narrative description.

Example: Assume that the soils in the streambank are loose silty sands. From table TS14A–5, estimate a relative density value of 30 percent. From figure TS14A–27 (NAVFAC 1986), read an effective friction angle for a silty sand with a relative density of 30 percent to be 27.5 degrees.

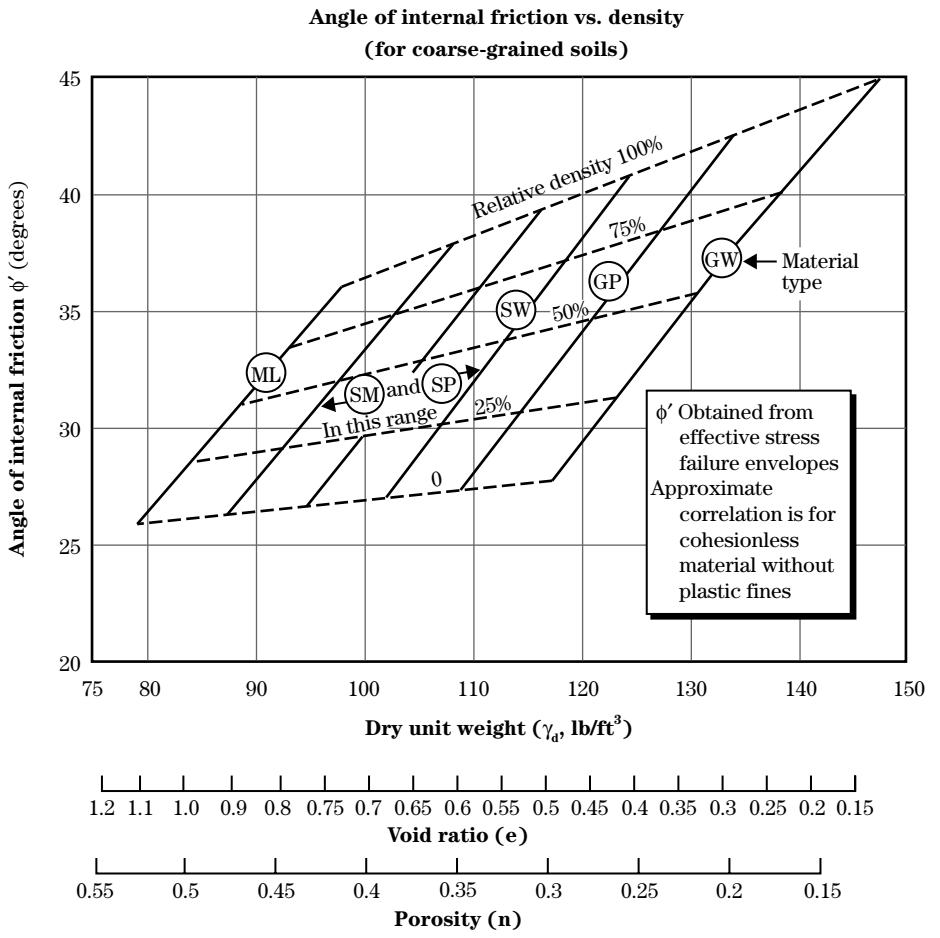
Direction of seepage forces

Three conditions are possible and are described in following sections.

Table TS14A–5 Estimated values of relative density for described soils

Relative density description	Numerical value of relative density
Very loose	<15
Loose	15–35
Medium	35–65
Dense	65–85
Very dense	>85

Figure TS14A–27 An example of a chart used to estimate effective friction angles for different types of soils



Equations for analyzing infinite slope stability

A separate equation is used for each of the three following seepage conditions:

- no seepage
- seepage flowing generally parallel to slope
- seepage generally following horizontal flow paths

The equation for no seepage is used to study the stability of soils above the water table. The condition where seepage generally follows horizontal flow paths is probably the one to use for most alluvial deposits because the layering of the alluvial soil profile creates this type of preferential flow path. Assume parallel flow paths for soils without much layering.

Because the assumption for this analysis is that the soils have zero cohesion, the height of the slope is not a factor. The same factor of safety is calculated for any height of slope. A factor of safety of 1.1 is commonly regarded as acceptable for this condition because the failures are shallow sloughing types of failures and not usually disastrous in nature.

Moist slope equation

If no seepage is exiting the slope face being examined, the factor of safety for that slope is simply stated as:

$$FS = m \times \tan(\phi') \qquad \text{(eq. TS14A–6)}$$

where:
m = slope cotangent
ϕ' = internal friction angle of cohesionless slope soil

Consider the example for 3H:1V permeability side slopes and soils with an effective ϕ' angle of 26 degrees, the calculated factor of safety is:

$$FS = 3 \times \tan(26)$$
$$FS = 1.46$$

Seepage parallel to slope

When seepage exits the slope face and the direction of the flow is parallel to the slope face, equations TS14A–7 and TS14A–8 are applicable. This pattern of seepage occurs in very homogeneous soils with little or no horizontal layering. This assumption is typically not used to represent soil that has been compacted in layers or in alluvial soils. The equations are:

$$FS = \frac{\gamma_b \times \tan(\phi')}{\gamma_{sat} \times \tan(\theta)} \qquad \text{(eq. TS14A–7)}$$

or

$$FS = m \times \frac{\gamma_b}{\gamma_{sat}} \times \tan(\phi') \qquad \text{(eq. TS14A–8)}$$

where:
θ = slope angle
m = slope cotangent = cot (θ)
γ_b = buoyant unit weight
γ_{sat} = saturated unit weight
ϕ' = effective friction angle

Horizontal seepage

Equations TS14A–9 and TS14A–10 assume that the seepage forces acting on the soils in the slope are due to flow along horizontal planes.

$$FS = \frac{\left(\gamma_{sat} \times \cos^2(\theta) - \gamma_w\right) \times \tan(\phi')}{\gamma_{sat} \times \sin(\theta) \times \cos(\theta)} \qquad \text{(eq. TS14A–9)}$$

or

$$FS = \frac{\left(\gamma_b \times m^2 - \gamma_w\right) \times \tan(\phi')}{m \times \gamma_{sat}} \qquad \text{(eq. TS14A–10)}$$

where:
θ = slope angle
m = slope cotangent = cot (θ)
γ_b = buoyant unit weight
γ_{sat} = saturated unit weight
ϕ' = effective friction angle

Technical Supplement 14B

Scour Calculations

Issued August 2007

Cover photo: Where sediment transport capacity of a stream exceeds the sediment supply, scour may occur. Scour may be estimated using analytical tools and accommodated or re-mediated by the design.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14B

Scour Calculations

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Technical Supplement 14B

Scour Calculations

Purpose

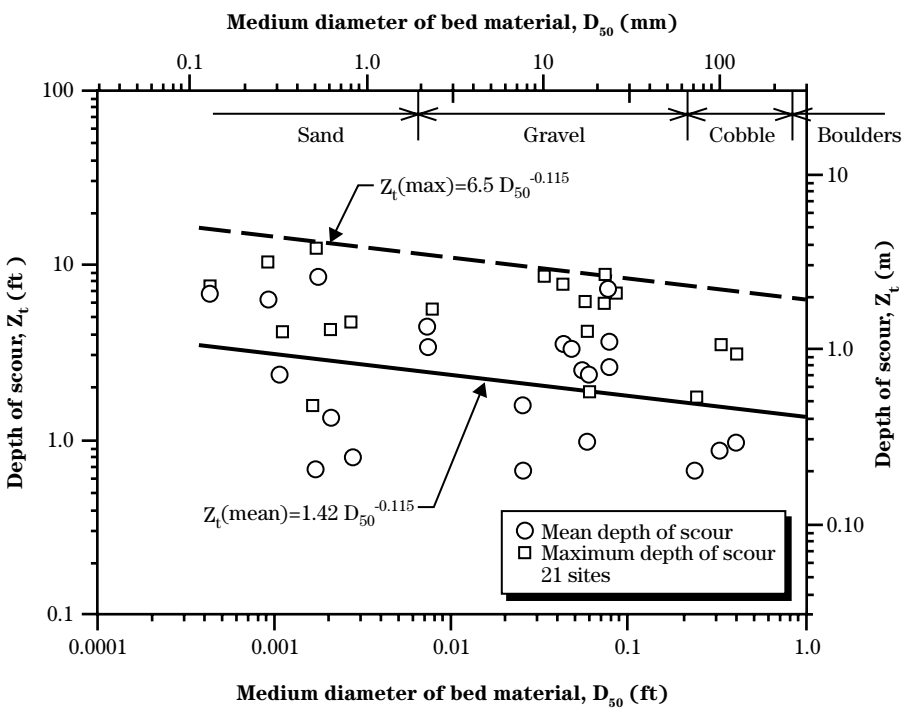
Scour is one of the major causes of failure for stream and river projects. It is important to adequately assess and predict scour for any stream or river design. Designers of treatments such as barbs, revetments, or weirs (that are placed on or adjacent to streambeds) must estimate the probable maximum scour during the design life of the structure to ensure that it can adjust for this potential change. This technical supplement provides guidance useful in performing scour depth computations.

Introduction

Streams continually mold and remold their streambeds by eroding and depositing sediments. Scour and fill of alluvial channels not undergoing long-term aggradation or degradation occur as fluctuations about some average condition. Blodgett (1986) presented information regarding bed elevation fluctuations from 21 sites on streams with a range of bed material sizes. Monthly or annual measurements were made at the same location within generally straight reaches, free of features like bedrock, bridge piers, or large boulders that might cause local scour. Mean and maximum scour depths are plotted in figure TS14B–1 (Blodgett 1986) as a function of median bed-material size. In this figure, the scour depth is defined as the depth of scour below a reference plane, which was set at the highest thalweg elevation measured during the period of observation. Clearly, scour depths can be quite significant.

Scour is perhaps the primary cause of failure of riverine hydraulic structures, and failure to adequately assess and predict scour hazard represents a major design flaw. For example, most failures of continuous bank protection projects like revetments are due to toe scour. The most spectacular examples of structural failure due to scour involve bridges, such as those

Figure TS14B–1 Scour observations from typical reaches of alluvial rivers



summarized in table TS14B–1 (Lagasse and Richardson 2001). Less well known, but also important, scour problems account for high failure rates sometimes reported for stream habitat structures and modified or realigned stream channels (Frissell and Nawa 1992). A survey of U.S. Army Corps of Engineers (USACE) flood control channel projects found that most reported problems were linked to some form of bed or bank instability, including local scour and vertical instability (McCarley, Ingram, and Brown 1990).

An analysis of potential scour is required for all types of streambank protection and stabilization projects. In addition, scour analysis should be a part of the design of any hard structure placed within the channel. Scour and deposition, of course, are processes that affect any movable bed channel design as described in NEH654.09. An analytical approach is needed because many streams tend to scour during high flows and fill during hydrograph recession. Therefore, severe scour can occur during periods when the bed is obscured, only to refill and appear completely different at baseflow.

Although the term scour includes all erosive action of running water in streams, including bed and bank erosion, the emphasis in this technical supplement is on erosion that acts mainly downward or vertically, such as bed erosion at the toe of a revetment or adjacent to a bank barb. Designers of objects placed on or adjacent to streambeds such as bridge piers, revetments, spurs, barbs, deflectors, weirs, sills, or grade control structures must estimate the probable maximum scour during the design life of the structure and ensure that the structure extends below maximum scour depth. This technical supplement provides guidance on scour depth computations.

Processes

Scour occurs due to several related processes, and estimated maximum scour is typically computed by summing the scour due to each individual process. Terms used to describe bed erosion processes include degradation, local scour, contraction scour, bend scour and others, and these are related as shown in table TS14B–2. Aggradation and degradation refer to an increase or decrease, respectively, in bed elevation over a long reach, through sediment deposition or erosion. Aggradation and degradation are major adjustments of a fluvial system due to watershed changes. In contrast, scour is erosion of the streambed that, except locally, does not influence the longitudinal profile or gradient of the stream. Scour may also be of a temporary, cyclic nature, with significant local erosion occurring during high flows, and refilling during the receding portion of the flow.

All types of scour are loosely categorized as either general or local scour (Brice et al. 1978). Local scour refers to erosion of the streambed that is immediately adjacent to (and apparently caused by) some obstruction to flow (fig. TS14B–2) (Brice et al. 1978)). General scour commonly affects the entire channel cross section, but it may affect one side or reach more than another (fig. TS14B–3) (Brice et al. 1978)). Both types

Table TS14B–1 Examples of bridge failures associated with scour

Date	Event	Stream	Conditions at time of event	Consequences	Cause
April 1987	NY State Thruway bridge collapses	Schoharie Creek, NY	Near-record flood	10 deaths	Cumulative effect of local scour over 10 yr
1989	U.S. 51 bridge collapses	Hatchie River, TN	—	8 deaths	Northward migration of the main river channel
March 10, 1995	Interstate 5 bridges collapse	Los Gatos Creek, CA	Large flood	7 deaths	Stream channel degradation combined with local scour

Table TS14B–2 Streambed erosion and deposition processes

General process	Specific process	Description and subtypes	
Aggradation or degradation		An increase or decrease in bed elevation over a long reach through sediment deposition or erosion	
Scour	General scour	Longitudinally local erosion that affects the entire channel cross section	Contraction scour Bend scour
	Bedform scour	Formation of troughs between crests of bedforms, usually in sand-bed streams	Dunes Antidunes
	Local scour	Erosion of the streambed that is immediately adjacent to (and apparently caused by) some obstruction to flow	Bridge pier and abutment scour Scour at structures that span the channel, such as weirs and sills Scour at structures that do not fully span the channel

Figure TS14B–2 Examples of local scour

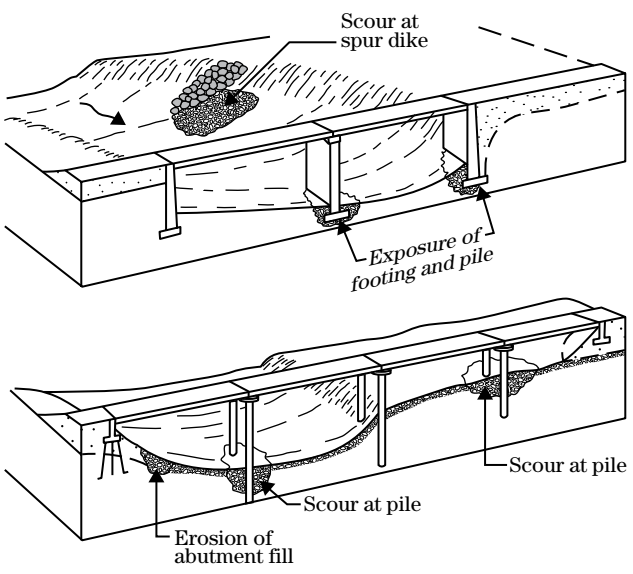
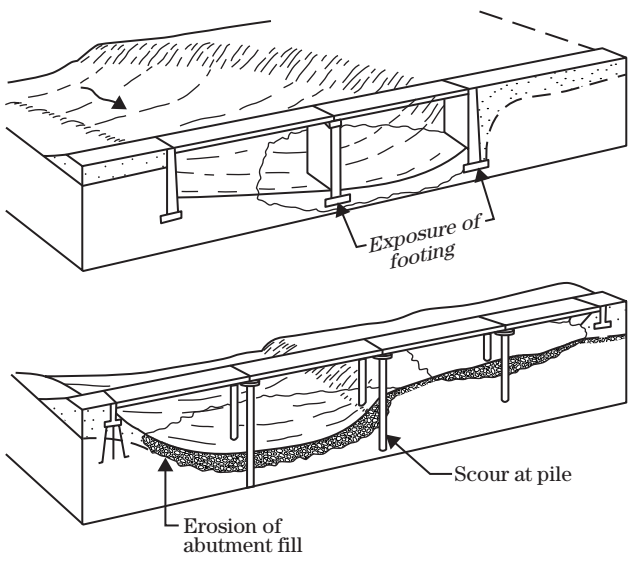


Figure TS14B–3 Examples of general scour



of scour occur discontinuously in the longitudinal direction along a reach, and both types can be cyclic in time. Note that spatially continuous vertical displacement of the streambed is referred to as either aggradation or degradation.

In many cases, physical deficiencies in streams with degraded habitat are addressed by inducing scour with structures that create pool habitat and cover (Brookes, Knight, and Shields 1996; Shields, Knight, and Cooper 1998; Lenzi, Comiti, and Marion 2004). In such cases, the designer seeks to maximize scour hole depth and volume subject to channel and structural stability constraints. Procedures for estimating this type of scour are presented below.

Scour is difficult to accurately measure in the field, and most design equations are based on theory supported by laboratory data. However, the following qualitative principles are useful in understanding scour processes (Laursen 1952; Vanoni 1975):

- The rate of scour is equal to the difference between the capacity for transport out of the scoured area and the rate of transport into the scoured area.
- Scour rates decline as scour progresses and enlarges the flow area.
- Scour asymptotically approaches a limiting extent (volume or depth) for a given set of initial conditions.

Effects of stream types

Flow regime—Stream channels may be classified as perennial, intermittent, or ephemeral based on their flow regime, as described in NEH654.07. Similar approaches are used to analyze and predict scour depths in all three types of channels. However, application of the three qualitative principles outlined indicates that extreme flow variations can lead to extreme variations in scour depths and patterns. Since scour asymptotically approaches a limiting depth for a given hydraulic condition, if flows are flashy, the limiting depth for a given flow may never be reached.

Bed material and sediment transport regime—Alluvial and threshold channels are fully described in NEH654.01 and NEH654.07. Scour in alluvial channels is usually live-bed scour, which implies that there is significant transport of sediment from upstream reaches into the reach in question. Scour occurs when transport capacity exceeds supply from upstream, and cyclic scour behavior is normal. Deposition or fill during waning stages of floods restores the scoured bed to near its preflood position, although scour holes may persist during oscillating flow conditions in gravel-bed streams (Neill 1973).

Scour in threshold channels tends to be clear-water scour unless flows become high enough that the threshold of bed sediment motion is exceeded. Clear-water scour implies that there is little or no movement of bed material from upstream reaches into the design reach. Clear-water scour is typically associated with coarse beds, flat gradient streams at low flow, local deposits of bed materials larger than the largest size being transported by the flow, armored streambeds, and vegetated channels or overbank areas where flow forces are less than those required to remove sediments protected by the vegetation.

Due to the complexity of scour, many of the studies used to support the equations presented below were conducted in flumes under clear-water conditions. The flow strength or bed shear stress was just lower than needed to erode and transport sediments from the bed, but adequate to trigger scour at a model contraction, spur, bridge pier, or other flow obstruction. The user must be aware that these equations probably will yield conservative results when applied to alluvial channels. Clear-water scour in coarse-bed streams reaches a maximum over a longer period of time than live-bed scour, but is about 10 percent greater than the equilibrium live-bed scour (Richardson and Davis 2001). During a flood event, streams with coarse-bed material may experience clear-water scour at low discharges during rising and falling stages, and live-bed scour at the higher discharges.

Different materials scour at different rates—Noncohesive silts and sands scour rapidly, while cohesive or cemented soils are much more scour resistant and erode relatively slowly. However, ultimate scour in cohesive or cemented soils may be just as great, even though the ultimate scour depth is reached more slowly. Under constant flow conditions, scour reaches maximum depth in sands within hours; in cohesive bed

materials in days; in months in glacial till, sandstones, and shale; in years in limestone; and in centuries in dense granite. These are generalities; actual measurements show additional complexity. For example, tests of cohesive streambeds showed that erodibility varied across four orders of magnitude in a single north Mississippi watershed (Simon and Thomas 2002).

Planform—Channel planform interacts with scour processes (USACE 1991b). During high flows, the bed scours in meander bends and builds up in the shallower reaches (thalweg crossings) between the bends. On the recession side of a flood, the process is reversed. Some observers note that braided channels experience greatest scour during intermediate flows, when current vectors attack bank lines at sharp, impinging angles. In meandering channels, the thalweg in bends often moves toward the outer (concave) bank following placement of revetment or other types of direct erosion protection. The amount of additional bend scour is related to the relative erodibility of the bed and banks. Channels with highly erodible bed and banks experience most significant additional scour at the toe of a newly placed revetment.

Scour computations for design

The total scour depth needed for design of key-in or toe-down elevations may be computed by summing all the components of vertical bed change:

$$z_t = FS\left[z_{ad} + z_c + z_b + z_{bf} + z_s\right] \qquad \text{(eq. TS14B–1)}$$

where:

z_t = total scour depth, ft (m)
FS = factor of safety
z_{ad} = bed elevation changes due to reach-scale deposition (aggradation) or bed erosion (degradation), ft (m)
z_c = contraction scour, ft (m)
z_b = scour on the outside of bend, ft (m)
z_{bf} = bedform trough depth, ft (m)
z_s = local scour depth associated with a structure, ft (m)

Guidance for computing each component of scour is provided in table TS14B–3.

An overview of the analyses presented in this technical supplement, organized by channel bed type, is presented in a summary table later in this technical supplement.

Table TS14B–3 Types of scour analyses

Type of scour or process	Symbol	Type of analysis
Long-term bed elevation change	z_{ad}	Armoring, equilibrium slope, or sediment continuity
Total general scour		Empirical equations or regime equations
Contraction scour	z_c	Live-bed or clear-water contraction scour
Bend scour	z_b	Bend scour formulas, most include all types of scour
Bridge scour	Not treated herein	Guidance provided by Richardson and Davis (2001)
Bedform scour	z_{bf}	Formulas for dunes or antidunes. Select type of bedform using bedform predictor equation.
Local scour	z_s	Empirical relations for each major type of structure

Long-term bed elevation change

Although any given point in a stream is constantly changing, a stable stream maintains the same average vertical position for its bed when viewed over a long reach (>20 channel widths) over a long period of time (several decades to a few centuries). Such stable streams are rare, however, because disturbances in the form of human activities or natural events (rare flood events, volcanic eruptions, earthquakes, landslides, fires) are common. Human activities are the most common cause of vertical instability, and among these are urbanization, dam construction, channelization, streambed mining, and land use changes. The effect of each of these activities on fluvial systems is described in NEH654.01 and in USACE (1994a). In general, these activities change the supply of sediment or water to a reach (for example, a dam) or increase the sediment transport capacity of a reach (for example, channel straightening, which increases channel slope). Vertical (and often lateral) instability occurs as the channel degrades or aggrades in response to the imbalance between supply and transport (fig. TS14B–4).

For long-term scour estimation, the designer must compute the long-term bed elevation change required to produce an equilibrium slope. If coarse materials are present in the bed and are not mobilized by a design event, the designer should also compute the depth of scour needed to produce an armor layer. The correct scour depth to use in design (eq. TS14B–1) will be the lesser of the two depths. In general, armoring limits degradation in beds with gravels and cobbles, while beds of finer material degrade until they reach an equilibrium slope.

Armoring

Streambeds often feature a layer of coarse particles at the surface that overlies a heterogeneous mixture containing a wide range of sediment sizes. This layer, which is usually only one or two particle diameters thick, is referred to as the armor layer. Formation and destruction of armor layers on streambeds is described in NEH654.07. When a streambed contains at least some sediment that is too large to be transported by the imposed hydraulic conditions, finer particles are selectively removed. The layer of coarser materials left behind forms an armor layer that limits further scour unless and until higher levels of shear stress destroy the armor layer. For example, armor layer formation is often observed downstream from dams. Borah (1989) proposed the following relationship to compute the scour depth below a dam in a channel with a well-mixed bed comprised of particles with the same specific gravity (fig. TS14B–5):

Figure TS14B–4 Conceptual representation of the relationship between long-term average vertical stability and sediment transport

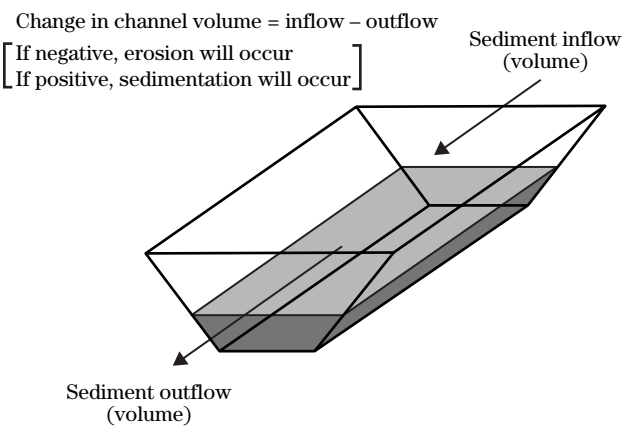
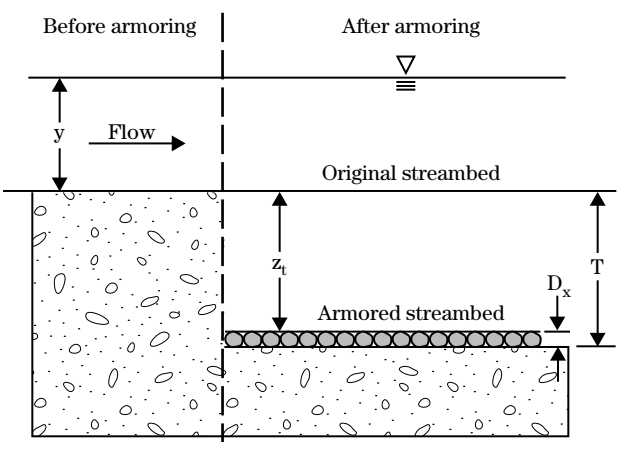


Figure TS14B–5 Definition of terms for armor limited scour



$$z_t = T - D_x \qquad \text{(eq. TS14B–2)}$$

where:
T = thickness of the active layer of the bed, ft (m)
D_x = smallest armor size or the size of the smallest nontransportable particle present in the bed material, ft (m)

T is related to D_x as follows:

$$T = \frac{D_x}{(1-e)P_x} \qquad \text{(eq. TS14B–3)}$$

where:
e = porosity of the bed material
P_x = fraction of bed material (expressed as a decimal) of a size equal to or coarser than D_x

Various approaches may be used to compute the smallest armor particle size, D_x. Borah (1989) suggested using relations based on the Shields curve for the initiation of motion. These relations take the form:

$$D_x = K\left(\frac{yS_e}{\Delta S_g}\right)^a \left(\frac{U_*}{\nu}\right)^b \qquad \text{(eq. TS14B–4)}$$

where:
y = flow depth, ft (m)
S_e = energy slope
ΔS_g = relative submerged density of bed-material sediments ≅ 1.65
U_* = shear velocity = $(gyS_e)^{0.5}$

where:
g = acceleration of gravity, 32.2 ft/s^2 (9.81 m/s^2)
ν = kinematic viscosity of water, ft^2/s (m^2/s)

K, a, and b are constants based on the particle Reynolds number as shown in table TS14B–4.

Table TS14B–4 Constants for computation of minimum armor particle size

Particle Reynolds number $\frac{U_* D_{50}}{\nu}$	K	a	B
≤10	68	1.67	0.67
Between 10 and 500	27	0.86	–0.14
≥500	17	1.0	0.0

where:
D_{50} = median grain size of sediment mixture in ft (m). The bed porosity e, D_{50}, and P_a are all estimated from analyses of bed-material samples. Bed porosity may also be estimated using a formula (Komura and Simons 1967):

$$e = 0.245 + \frac{0.0864}{(0.1D_{50})^{0.21}} \qquad \text{(eq. TS14B–5)}$$

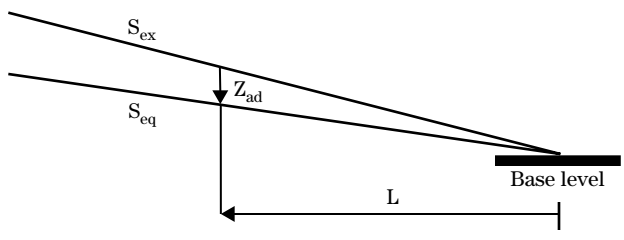
where:
D_{50} = median grain size in mm

Equilibrium slope

When sediment transport capacity exceeds sediment supply, channel bed degradation occurs until an armor layer forms that limits further degradation or until the channel bed slope is reduced so much that the boundary shear stress is less than a critical level needed to entrain the bed material. This new, lower slope may be called the equilibrium slope, S_{eq}. Slope adjustment in a sediment deficient reach occurs by degradation, proceeding from the upstream end to the downstream, and the downstream extent of degradation is often limited by a base level control. Figure TS14B–6 illustrates the relationship between existing slope, S_{ex}, equilibrium slope and ultimate general scour due to bed degradation, Z_{ad}, for a reach of length L with base level control).

For example, the reach downstream from a reservoir without major tributaries may degrade first just below the dam, and a wave of bed degradation will proceed downstream, gradually tapering off as a base level control (a culvert or a downstream reservoir) is reached. Without downstream control, degradation will continue until halted by channel bed armoring, or until the

Figure TS14B–6 Definition of equilibrium slope, S_{eq}



entire profile reaches equilibrium slope. The amount of ultimate degradation at a given location upstream from the base level control may be estimated by:

$$z_{ad} = L\left(S_{ex} - S_{eq}\right) \qquad \text{(eq. TS14B–6)}$$

where:
L = distance upstream of the base level control

Equilibrium slope is a function of the contributing drainage area. Equilibrium slopes are greater for smaller drainage areas, and therefore equilibrium slope and ultimate degradation must be computed reach by reach.

Several approaches for computing equilibrium slope are presented below (Lagasse, Schall, and Richardson 2001), as outlined in table TS14B–5. If the computed stable slope is greater than the existing slope, the risk of additional degradation is low, and the streambed may already be armored.

Use of the relationships below is complicated by channel response. If bed degradation is associated with bank failure, sediment supply may be replenished from the eroding banks, at least temporarily. A rough technique for computing sediment supply from banks is described by Pemberton and Lara (1984), and more detailed computations are contained in the ARS bank stability model, available at the following Web site:

http://www.ars.usda.gov/Business/docs.htm?docid=5044

Channel incision may also lead to narrowing, which affects discharge. It is also difficult to select a single discharge for use with the above relationships. See the discussion in NEH654.05 and NEH654.09 regarding channel-forming discharges.

When a base level control is lowered or removed (the downstream bed elevation is lowered due to channel change), channel degradation will proceed upstream, migrating up each of the tributaries to the watershed divide. Watershedwide consequences can be severe (Simon and Thomas 2002), with sediment yield increasing by an order of magnitude due to enlargement of the channel. Ultimate degradation may again be computed based on equilibrium slope. Critical shear stresses are very low for sands, and the associated equilibrium slopes are so flat that the amount of potential degradation is quite large.

Calculation of equilibrium slope as a stability assessment is also described in NEH654.13.

Table TS14B–5 Approaches for determining equilibrium slope

Bed type	Sediment supply from upstream	Approach for equilibrium slope	Equation(s)
Cohesive silt or clay	Any	Watershed-specific regression	n/a
Sand to fine gravel $0.1 \le D_{50} \le 5.0$ mm	Drastically reduced or none	Back calculation based on critical shear stress	TS14B–7
	Reduced	Back calculation based on sediment supply	TS14B–12 or TS14B–13
Coarser than sand	Drastically reduced or none	Manning and Shields	TS14B–14
		Meyer-Peter and Müller	TS14B–15
		Schoklitsch	TS14B–16
		Henderson	TS14B–17
Sand or gravel	Any	Sediment continuity	TS14B–18

Cohesive beds

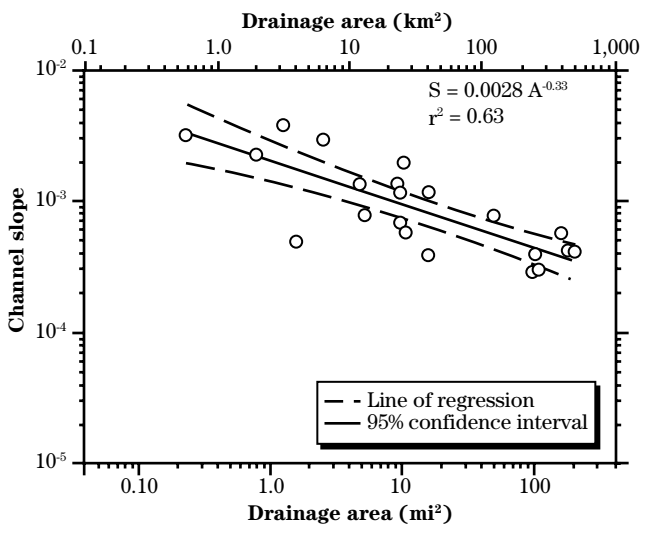
Nickpoint (or knickpoint) migration is a dramatic form of vertical instability that occurs in cohesive soils (fig. TS14B–7). A nickpoint (or headcut) is an abrupt change or inflection in the longitudinal profile of a cohesive streambed. In noncohesive materials, analogous features are manifest as short, steep reaches known as nickzones (or knickzones). Both types of features tend to migrate upstream, particularly during high flows. Bed degradation in the immediate vicinity of a migrating nickpoint can be dramatic, as the bed may be lowered or degraded up to several meters in a single flow event.

The sequence of changes that typically occurs in channels when a headcut passes through have been described in a conceptual model known as the channel evolution model (CEM), or incised channel evolution model (ICEM) (Simon 1989) as described in NEH654.01, NEH654.03, and NEH654.13. Due to the complexities of cohesive bed erosion, it is difficult to predict the rate of nickpoint migration, even given the hydrograph. However, the ultimate amount of degradation may be estimated by extending a thalweg profile from a fixed downstream base level upstream at a slope equal to the equilibrium slope, S_{eq}, determined as described (fig. TS14B–6).

Some investigators have developed watershed-specific regressions for predicting S_{eq} for watersheds with beds of sand and consolidated cohesive outcrops. These formulas may be used to predict S_{eq} from the contributing drainage area. The regressions are based on bed slope and drainage area for reaches that have undergone enough degradation to attain equilibrium (fig. TS14B–8 (Simon and Thomas 2002)). This approach may be sufficient for estimation purposes, but it ignores the unsteady nature of sediment supply and resultant complex response. The scatter in predicted values is large (fig. TS14B–8). A similar alternative approach involves fitting an exponential function to plots of thalweg elevation at a given cross section versus time to predict future bed elevations (Simon 1992).

Figure TS14B–7 Headcut migrating upstream through cohesive streambed toward bridge in north-central MS. Headcut was triggered by downstream channelization.

Figure TS14B–8 Empirical equilibrium slope–drainage area relationship for Yalobusha River watershed in northern MS



Sand and fine gravel—no bed-material sediment supplied from upstream

For channels with bed material coarser than sand, armoring and slope reduction processes may occur simultaneously. Both types of analyses must be performed to determine which will provide the limiting factor. Pemberton and Lara (1984) also suggest that stable slopes may be computed for channels with noncohesive beds with sediment sizes between 0.1 millimeter and 5 millimeters by obtaining a critical shear stress value from the graphical compilation published by Lane (1952) (fig. TS14B–9).

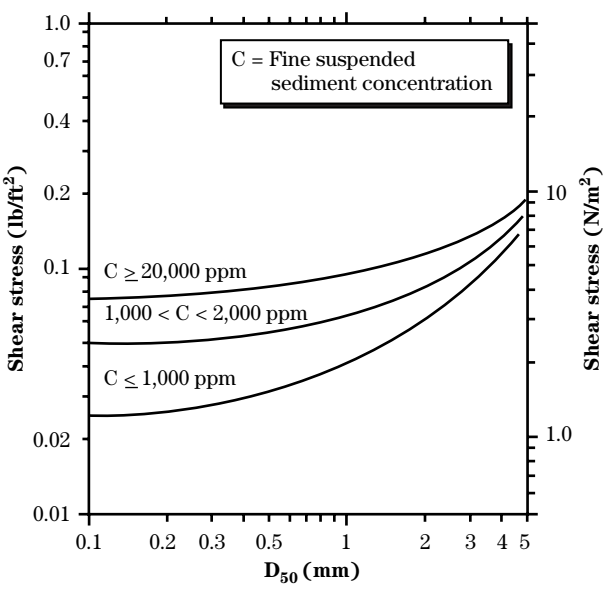
$$S_{eq} = \left(\frac{\tau_c}{\gamma_w y}\right) \qquad \text{(eq. TS14B–7)}$$

where:

τ_c = critical shear stress from the curve in figure TS14B–9, lb/ft^2 (N/m^2)

y = mean flow depth, ft (m)

Figure TS14B–9 Critical shear stress for channels with boundaries of noncohesive material. Critical shear stress increases with increasing fine suspended sediment concentration.



Sand and fine gravel—reduced sediment supply from upstream

It is not uncommon to have the sediment supply reduced to a stream reach. This occurs when a watershed is reforested, in later stages of urbanization, bed material is mined, diversions are constructed, or when reservoirs are placed in one or more subwatersheds. The concept of equilibrium slope remains valid for these conditions. Use observed bed-material sediment discharge data to fit a regression function of the form:

$$q_s = au^b y^c \qquad \text{(eq. TS14B–8)}$$

where:

q_s = sediment transport capacity in dimensions of volume per unit width per unit time, ft^2/s (m^2/s)

u = mean streamwise velocity, ft/s (m/s)

y = mean flow depth, ft (m)

a, b, c = coefficients and exponents from regression

The sediment transport capacity may be converted to dimensions of weight per unit width per unit time (tons/d) by multiplying by 7,144 (228,960 to convert m^2/s to metric tons/d).

For purposes of equilibrium slope computation, q_s, should be computed using the mean velocity and flow depth corresponding to the channel-forming discharge as defined in NEH654.05. Since sediment supply and sediment transport capacity are determined for the same water discharge, computation of equilibrium slope is not very sensitive to errors in determining effective discharge. If available sediment data are inadequate to generate a reliable regression, a sediment transport relationship may be used to synthesize coefficients. For sand streambeds, the following formulas are available for the coefficients a, b, and c in equation TS14B–8 (Yang 1996):

$$a = 0.025n^{(2.39-0.8\log D_{50})}\left(D_{50} - 0.07\right)^{-0.14} \qquad \text{(eq. TS14B–9)}$$

$$b = 4.93 - 0.74\log D_{50} \qquad \text{(eq. TS14B–10)}$$

$$c = -0.46 + 0.65\log D_{50} \qquad \text{(eq. TS14B–11)}$$

D_{50} has units of millimeters.

When SI units are used in the equation for q_s coefficients b and c are unchanged, and the coefficient, a, must be multiplied by a factor of $0.3048^{(2-b-c)}$. These formulas are based on regression of a large data set with ranges given in table TS14B–6.

Similar regression coefficients for sediment transport under conditions outside these ranges (0.1 mm < D_{50} < 5.0 mm) are provided by Richardson, Simons, and Lagasse (2001). If it is assumed that bed-material size and channel width do not change as the channel degrades, the equilibrium slope may be computed by:

$$S_{eq} = \left(\frac{a}{q_s}\right)^{\frac{10}{3(c-b)}} q^{\frac{2(2b+3c)}{3(c-b)}} \left(\frac{n}{K}\right)^2 \qquad \text{(eq. TS14B–12)}$$

where:
K = 1.486 (1.0 for SI units), and other variables are as previously defined

For a reduction in sediment supply to a reach in which all other characteristics remain unchanged (water discharge, roughness, and channel width), the equilibrium slope may be computed by:

$$S_{eq} = S_{ex} \left(\frac{Q_{s\,(future)}}{Q_{s\,(existing)}}\right)^{\frac{10}{3(b-c)}} \qquad \text{(eq. TS14B–13)}$$

where:
S_{ex} = existing channel slope
Q_s = sediment supply, ft^3/s (m^3/s)

Table TS14B–6 Ranges for data set underlying the Yang sediment transport relation (eqs. TS14B–8 through TS14B–11)

Quantity	Range (SI units)
D_{50}, mm	0.1–2.0
u, velocity, ft/s (m/s)	2.0–8.0 (0.6–2.4)
y, depth, ft (m)	2.0–25 (0.6–7.6)
S, slope	0.00005–0.002
Manning *n*	0.015–0.045
Froude No.	0.07–0.70
q, unit discharge of water, ft^3/s (m^3/s)	4.0–200 (0.37–18.6)

The sediment supply for existing conditions may be measured or computed, while the supply for future conditions must be computed using an appropriate sediment transport relation. In both cases, the sediment transport rate must correspond to the channel-forming discharge.

Beds coarser than sand—no sediment supplied from upstream

When a reservoir with long storage time is placed on a river or stream, bed-material sediment supply to downstream reaches is drastically reduced and can be cut off entirely. A similar reduction occurs in the latter stages of urbanization when construction sites and other disturbed areas are covered with impervious surfaces or vegetation. Four equations are presented for S_{eq}, the equilibrium slope, in conditions where sediment transport rates are negligibly small. Variable definitions follow the fourth equation. Equilibrium slope may be selected as an average of that provided by the four relations, or the most applicable relationship may be selected for use based on a study of the original references.

- Simultaneous solution of the Manning and Shields equations (for D_{50} > 6 mm):

$$S_{eq} = \left[\theta_c D_c \Delta S_g\right]^{\frac{10}{7}} \left(\frac{K}{qn}\right)^{\frac{6}{7}} \qquad \text{(eq. TS14B–14)}$$

- Based on Meyer-Peter and Müller sediment transport relationship for material coarser than sand:

$$S_{eq} = K \frac{(D_{50})^{\frac{10}{7}} n^{\frac{9}{7}}}{(D_{90})^{\frac{5}{14}} q^{\frac{6}{7}}} \qquad \text{(eq. TS14B–15)}$$

- Based on the Schoklitsch equation for coarse sand or gravel:

$$S_{eq} = K \left(\frac{D_m}{q}\right)^{\frac{3}{4}} \qquad \text{(eq. TS14B–16)}$$

- Based on the Henderson formula for materials larger than 6 mm:

$$S_{eq} = K\, Q_d^{-0.46} D_{50}^{1.15} \qquad \text{(eq. TS14B–17)}$$

where:

K = constants given as shown in table TS14B–7

S_{eq} = equilibrium channel slope at which sediment particles of size D_c and larger will no longer move

ΔS_g = relative submerged density of bed-material sediments ≅ 1.65

q = channel-forming discharge per unit width, $ft^3/s/ft$ ($m^3/s/m$)

n = Manning's roughness coefficient

D_{90} = sediment size for which 90% by weight of bed material is finer, m (ft)

D_{50} = median sediment size, ft (m) (**Note units**)

D_m = mean bed-material particle size, mm

Q_d = design discharge, ft^3/s (m^3/s)

Q_b = discharge over bed of channel, ft^3/s (m^3/s). Normally $Q_d/Q_b = 1$ for wide channels

y = mean flow depth, ft (m)

n_b = Manning's roughness coefficient for streambed

Sediment continuity analysis

In theory, sediment continuity analysis may be used for channels with any type of bed material. In practice, the lack of reliable sediment transport relations for channels with bed material finer than sand or coarser than cobbles makes such analysis difficult. In continuity analysis, the volume of sediment deposited in or eroded from a reach during a given period of time is computed as the difference between the volumes of sediment entering and leaving the reach:

$$\Delta V = V_{s_{in}} - V_{s_{out}} \qquad \text{(eq. TS14B–18)}$$

where:

ΔV = volume of bed-material sediment stored or eroded, ft^3 (m^3)

$V_{s_{in}}$ = volume of bed-material sediment supplied to the reach, ft^3 (m^3)

$V_{s_{out}}$ = volume of bed-material sediment transported out of the reach, ft^3 (m^3)

From equation TS14B–18, the average amount of bed level change may be computed by:

$$z_{ad} = \frac{\Delta V}{W_c L_r} \qquad \text{(eq. TS14B–19)}$$

where:

W_c = average channel width, ft (m)

L_r = reach length, ft (m)

Values of V_s may be computed using appropriate sediment transport relationships if bed-material sediment grain size distribution, design discharge, and reach hydraulics are known. Selection and use of sediment transport relationships are described in NEH654.09 and Richardson, Simons, and Lagasse (2001). Lagasse, Schall, and Richardson (2001) demonstrate the use of the Yang equations for sand and gravel for this purpose. Normally, sediment concentrations are computed only for the design discharge and converted to volume by multiplying by the water discharge and a time $\Delta\tau$ corresponding to the duration of the design discharge. For a more complete analysis, sediment concentration may be computed for a range of water discharges and combined with a flow-duration curve to obtain long-term values of ΔV. Alternatively, the design event hydrograph may be expressed as a series of

Table TS14B–7 Constants for equilibrium slope formulas for coarse bed channels with little or no sediment load input

Relationship	U.S. units	SI units	Reference
Manning and Shields	1.486	1.0	Lagasse, Schall, and Richardson (2001)
Meyer-Peter and Müller	60.1	28.0	Lagasse, Schall, and Richardson (2001)
Schoklitsch	0.00174	0.000293	Pemberton and Lara (1984)
Henderson	0.44 (D_{50} in ft)	0.33 (D_{50} in m)	Henderson (1966)

n discrete time intervals, and ΔV values computed for the average discharge occurring during each interval. Numerical integration is then used to obtain the total ΔV for the event:

$$\Delta V = \sum_{i=1}^{n} \left(V_{s_{in}\,i} - V_{s_{out}\,i} \right) \Delta t_i \qquad \text{(eq. TS14B–20)}$$

This type of analysis may be laborious if several events are simulated, and changes in reach hydraulics due to changes in bed-material gradation may be hard to track. More sophisticated methods are described in the following section.

More complex approaches for long-term aggradation or degradation

Detailed assessment of scour and deposition in a channel reach under natural (unsteady) inputs of water and sediment require numerical (computer) simulation modeling. Since flow records are input as hydrographs, it is not necessary to select a single design flow. Primary types of simulation models include one-dimensional models, which simulate changes in bed elevation with streamwise distance, but ignore variations from one side of the channel to the other, and two-dimensional models, which represent the channel bed as a mosaic of rectangular areas, but do not simulate variations in velocity in the vertical direction. One-dimensional models have limited capability to simulate local scour.

The models route sediment down a channel and adjust the channel geometry (usually bed elevation, but not bank position) to reflect imbalances in sediment supply and transport capacity. The BRI–STARS (Molinas 1990) and HEC–6 (USACE 1993c) models are examples of sediment transport models that can be used for single event or long-term degradation and aggradation estimates. HEC–6 is introduced in NEH654.13, and simulates only changes in bed elevation, while BRI–STARS has an option for predicting width adjustment. The USDA ARS model CONCEPTS includes a full suite of routines for assessing the geotechnical stability of channel banks and erosion of bank material through both hydraulic and geotechnical processes, as well as one-dimensional flow modeling (Langendoen 2000). The information needed to run these models includes (Lagasse, Schall, and Richardson 2001):

- channel and flood plain geometry
- structure geometry
- hydraulic roughness
- geologic or structural vertical controls
- downstream water surface relationship
- event or long-term inflow hydrographs
- tributary inflow hydrographs
- bed-material gradations
- upstream sediment supply
- tributary sediment supply
- selection of appropriate sediment transport relationship
- depth of alluvium

CONCEPTS also requires data describing geotechnical properties of bank soils. None of the models can predict the formation of nickpoints or their migration rates. Modeling movable-bed channels requires specialized training and experience. A description of how models should be used is presented by USACE (1993c).

General scour

Process description

General scour refers to all types of scour that are not local (fig. TS14B–3). General scour commonly, but not necessarily, occurs over the entire cross section, and may involve reaches of varying length depending on the type of scour and site-specific conditions. General scour includes contraction scour and bend scour. Presumably, most of the scour measured at the 21 sites observed by Blodgett (1986) was general scour. He noted that:

$$z_t\,(\text{mean}) = K\,D_{50}^{-0.11} \qquad \text{(eq. TS14B–21)}$$

and

$$z_t\,(\text{max}) = K\,D_{50}^{-0.11} \qquad \text{(eq. TS14B–22)}$$

where:

z_t (mean) = best fit curve (fig. TS14B–1) for observed scour depth, ft

z_t (max) = enveloping curve (fig. TS14B–1) for maximum scour depth, ft

K = coefficient = 1.42 and 6.5 for z_t mean and z_t max, respectively (0.84 and 3.8 for SI units), and D_{50} is the median size of the bed material, ft (mm)

Pemberton and Lara (1984) suggested that regime equations provided by Blench (1970) and Lacey (1931) could be used to predict general scour in natural channels. A designer may compute scour depth using both formulas, and average the outcome or take the largest value.

These regime relationships may be expressed as:

$$z_t = KQ_d^a W_f^b D_{50}^c \qquad \text{(eq. TS14B–23)}$$

where:

z_t = maximum scour depth at the cross section or reach in question, ft (m)

K = coefficient (table TS14B–8)

Q_d = design discharge, ft^3/s (m^3/s)

W_f = flow width at design discharge, ft (m)

D_{50} = median size of bed material (mm)

a, b, c = exponents (table TS14B–8)

Values for the coefficient and exponents are as shown in table TS14B–8 when U.S. units are used for Q_d, and W_f and D_{50} is in millimeters.

Values for the exponents, a, b, and c are unchanged when SI units are used, but values for the coefficient K when SI units are used for Q and W_f, and D_{50} is in millimeters (table TS14B–9).

Contraction scour

Contraction scour occurs when the flow cross section is reduced by natural features, such as stone outcrops, ice jams, or debris accumulations, or by constructed features such as bridge abutments. Contraction scour is most often observed when bridge approaches force flood plain flow back into the main channel to pass under the bridge. According to the law of continuity, a decrease in flow area requires an increase in the mean velocity component normal to the area, which produces an attendant increase in boundary shear stresses and bed-material transport, assuming the boundary is erodible. As erosion progresses, area increases and velocity decreases, leading to an equilibrium condition in which the rate of bed material transported into the contracted reach is equivalent to the rate of transport out of the contracted reach. Contraction scour is a

Table TS14B–8 Constants for Lacey and Blench relations, U.S. units (D_{50} in mm)

Condition	Lacey				Blench			
	K	a	b	c	K	a	b	c
Straight reach	0.097	1/3	0	–1/6	0.530	2/3	–2/3	–0.1092
Moderate bend	0.195	1/3	0	–1/6	0.530	2/3	–2/3	–0.1092
Severe bend	0.292	1/3	0	–1/6	0.530	2/3	–2/3	–0.1092
Right angle bend	0.389	1/3	0	–1/6	1.105	2/3	–2/3	–0.1092
Vertical rock wall	0.487	1/3	0	–1/6				

Table TS14B–9 Constant K for Lacey and Blench relations, SI units (D_{50} in mm)

Condition	Lacey	Blench
Straight reach	0.030	0.162
Moderate bend	0.059	0.162
Severe bend	0.089	0.162
Right angle bend	0.119	0.337
Vertical rock wall	0.148	0.000

form of general scour because material is removed from all, or almost all, of the wetted perimeter of the contracted section.

Live-bed contraction scour

Live-bed conditions may be assumed at a site if the mean velocity upstream exceeds the critical velocity for the beginning of motion, V_c, for the median size of bed material available for transport, D_{50}. When the velocity falls below the critical level, clear-water scour dominates. Both types of scour may occur during a given hydrologic event. The critical velocity may be estimated by:

$$V_c = Ky^{\frac{1}{6}} D_{50}^{\frac{1}{3}} \qquad \text{(eq. TS14B–24)}$$

where:
V_c = critical velocity, ft/s (m/s)
y = average flow depth in the reach in question, ft (m)
D_{50} = median bed particle size, ft (m) (note units)
K = a constant which is 11.17 for U.S. units or 6.19 for SI units

Richardson and Davis (2001) provide guidance for estimating contraction scour associated with bridges. In general, the procedure consists of using the following equations for estimating contraction scour depth under live-bed conditions:

$$z_c = y_2 - y_o \qquad \text{(eq. TS14B–25)}$$

and

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_{b1}}{W_{b2}}\right)^{a} \qquad \text{(eq. TS14B–26)}$$

where:
z_c = contraction scour
y_o = average initial depth in the contracted section
y_1 = average depth in the upstream channel
y_2 = average ultimate depth in the contracted section
Q_1 = flow rate in upstream channel, ft^3/s (m^3/s)
Q_2 = flow rate in the contracted section, ft^3/s (m^3/s)
W_{b1} = bottom width of the upstream channel, ft (m)
W_{b2} = bottom width of the contracted section, ft (m)
a = empirical exponent based on ratio of shear velocity to fall velocity of bed material determined (table TS14B–10):

Table TS14B–10 Exponent a for contraction scour relation

U_*/ω	a	Mode of bed-material transport
<0.50	0.59	Mostly contact bed-material discharge
0.50 to 2.0	0.64	Some suspended bed-material discharge
>2.0	0.69	Mostly suspended bed-material discharge

$$U_* = \left(\frac{\tau_o}{\rho}\right)^{\frac{1}{2}} = \left(gy_1S_e\right)^{\frac{1}{2}}$$

where:
U_* = $(\tau_o/\rho)^{1/2} = (gy_1S_e)^{1/2}$, shear velocity in the upstream section, ft/s (m/s)
g = acceleration of gravity, 32.2 ft/s^2, (9.81 m/s^2)
S_e = slope of energy grade line of main channel, ft/ft (m/m)
τ_o = average bed shear stress in the upstream section, lb/ft^2 (N/m^2), given by:

$$\tau_o = \gamma_\omega RS_e$$

where:
R = hydraulic radius, ft (m)
S_e = energy slope
ρ = density of water, 1.94 $slugs/ft^3$ (1,000 kg/m^3)
ω = fall velocity of bed material based on the D_{50}, ft/s (m/s)

Fall velocity for sand-sized particles may be read from the curves in figure TS14B–10 (Richardson and Davis 2001) or computed from formulas provided by Ahrens (2000).

$$\omega = \frac{K_1 \Delta S_g g D_s^2}{\nu} + K_2 \sqrt{\Delta S_g g D_s} \qquad \text{(eq. TS14B–27)}$$

where:

$$K_1 = 0.055 \tanh\left[12A^{-0.59} \exp\left(-0.0004A\right)\right] \qquad \text{(eq. TS14B–28)}$$

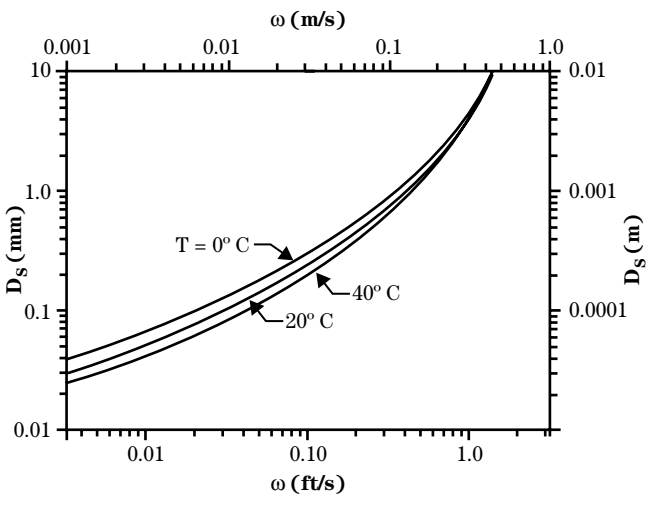
$$K_2 = 1.06 \tanh\left[0.016A^{0.50} \exp\left(-\frac{120}{A}\right)\right] \qquad \text{(eq. TS14B–29)}$$

ΔS_g = relative submerged density of bed-material sediments $\simeq$ 1.65
g = acceleration of gravity, 32.2 ft/s^2 (9.81 m/s^2)
ν = kinematic viscosity of water, ft^2/s (m^2/s)
D_s = a characteristic sediment diameter, ft (m)

$$A = \frac{\Delta S_g g D_s^3}{\nu^2} \qquad \text{(eq. TS14B–30)}$$

If bottom width is not easily defined, it is permissible to use top width for W_{b1} and W_{b2}, but it is important to use a consistent definition of width for both quantities. In sand-bed streams, a contraction scour zone is often formed during high flows and refilled during falling stages. In such a case, y_o may be approximated by y_1. Live-bed scour depths are sometimes limited by coarse sediments within the sediment mixture that form an armor layer. When gravel or larger sized material is present, it is recommended that scour depths be calculated using both live-bed and clear-water approaches, and that the smaller of the two scour depths be used. The procedure is a simplified version of one described in greater detail in Petersen (1986). An alternative approach for gravel-bed contraction scour is presented by Wallerstein (2003) that is based on sediment continuity.

Figure TS14B–10 Fall velocity for sand-sized particles with a specific gravity of 2.65



Clear-water contraction scour

Clear-water scour occurs when there is insignificant transport of bed-material sediment from the upstream into the contracted section. In some cases, a channel constriction creates enough of a backwater condition to induce sediment deposition upstream. Scour in the contracted section normally increases as the flow velocity increases. Live-bed scour becomes clear-water scour in the contracted section. The magnitude of clear-water contraction scour may be computed as follows (Richardson and Davis 2001):

$$z_c = y_2 - y_o \qquad \text{(eq. TS14B–31)}$$

and

$$y_2 = \left[\frac{KQ_2^2}{D_s^{\frac{2}{3}} W_{b2}^2}\right]^{\frac{3}{7}} \qquad \text{(eq. TS14B–32)}$$

where:

K = 0.0077 for U.S. units and 0.025 for SI units

Q_2 = discharge through the contracted section, ft^3/s (m^3/s)

D_s = diameter of the smallest nontransportable particle in the bed material. Assumed = $1.25D_{50}$, ft (m), (note units)

W_{b2} = bottom width of the contracted section, ft (m)

The assumption that $D_m = 1.25D_{50}$ implies that some armoring takes place as scour occurs. If the bed material is stratified, the ultimate scour depth may be determined by using the clear-water scour equation sequentially with successive D_s of the bed-material layers.

Bridge scour

Flow under bridges often produces local scour around bridge piers. Contraction of the floodway by bridge abutments and approaches also causes contraction scour across the cross section. Due to the economic and safety implications of bridge scour, it has received more study than any other type of scour, with extensive analyses of the effects of pier and abutment geometry, flow regime, sediment load, and bedforms. Scour at bridges is intensified when debris becomes trapped against the upstream side of piers (fig. TS14B–11 (Huizinga and Rydlund 2001). When the water surface upstream from a bridge opening is higher than downstream, a special condition known as pressure flow occurs. Pressure flow scour may be two to three times as great as normal contraction scour.

NEH654 TS14Q provides guidance for the analysis and design of small bridge abutments. The reader is also referred to Richardson and Davis (2001) for further design guidance.

Bend scour

Flow through channel meander bends results in water moving in a corkscrew or helical pattern that moves sediment from the outside (concave bank) to the inside of the bend, which is often a point bar. Velocity components not in the streamwise direction are referred to as secondary currents, and the secondary currents that occur in meander bends, though often quite complex, generally have the effect of eroding outer banks. The bank toe is often the locus of maximum shear and erosion, particularly if the bank is armored or otherwise resistant to erosion. Empirical relationships between the maximum depth of scour in a bend and the average depth in a bend have been developed using much of the field data as described in

Figure TS14B–11 Downstream face of Horse Island Chute bridge near Chester, IL, as viewed from left (north) embankment. Note debris trapped on upstream face of bents.

NEH654.09. Briefly, the field data lead to a conservative formula for bend scour, $z_b = y_{mean} - y_{max}$.

$$z_b = y\left(\frac{y_{max}}{y} - 1\right) \qquad \text{(eq. TS14B–33)}$$

where:
y = average flow depth in the bend, ft (m)
y_{max} = maximum flow depth in the bend, ft (m)

$$\frac{y_{max}}{y} = 1.5 + 4.5\left(\frac{W_i}{Rc}\right) \qquad \text{(eq. TS14B–34)}$$

where:
W_i = channel width at bend inflection point, ft (m)
Rc = bend radius of curvature, ft (m)

This equation is an asymptotic relationship with a theoretical minimum y_{max}/y_{mean} of 1.5 representing pool scour depths expected in a straight channel with a pool-riffle bed topography. From this upper-bound relationship, y_{max}/y_{mean} ranges from 4 to 3 for W_i/Rc between 0.33 and 0.56. For channels with W_i/Rc > 0.56, y_{max} is independent of bend curvature, and it is recommended that a value of 4 be used for y_{max}/y_{mean}. Consult NEH654.09 for additional detail.

Relations for predicting the location of maximum depth are also provided in NEH654.09. The length of the scoured zone may be approximated using a relationship by Chen and Cotton (1988):

$$\frac{L_p}{R} = 0.0604\left(\frac{R^{\frac{1}{6}}}{n}\right) \qquad \text{(eq. TS14B–35)}$$

where:
L_p = recommended length of protection, ft (m), measured downstream from the bend apex (fig. TS14B–12)
R = hydraulic radius = flow area/wetted perimeter, ft (m)
n = Manning n value for the bend

This relationship (eq. TS14B–35 and fig. TS14B–13) is only approximate, and scour locations vary considerably from bend to bend and with time in a given bend. NEH654.09 presents information regarding the observed distribution of scour locations (referred to as the pool-offset ratio) in a study of bends along the Red River.

Scour depths at bank toes on the outside of bends usually increase after construction of armored bank revetments. Increased resistance to bank erosion must intensify stresses acting at the bank toe, causing deeper scour. Maynord (1996) suggested the following empirical relationship for estimating toe scour in such a situation. This equation is embedded in the CHANLPRO software (Maynord, Hebler, and Knight 1998):

$$\frac{y_{max}}{y_c} = FS\left[1.8 - 0.051\left(\frac{Rc}{W_i}\right) + 0.0084\left(\frac{W_i}{y_c}\right)\right] \qquad \text{(eq. TS14B–36)}$$

where:
y_{max} = maximum water depth in the bend, ft (m)
y_c = mean water depth in the crossing upstream from the bend, ft (m)
FS = a factor of safety defined below

This relationship is limited to situations where ($1.5 < Rc/W_1 < 10$) and ($20 < W_1/y < 125$). The factor of safety, FS, may vary from 1.00 to 1.10.

The relationship above was developed using 215 data points from several rivers. When FS = 1.00, 25 percent of the observed values of y_{max}/y_c were underpredicted by more than 5 percent. When FS = 1.19, only 2 percent of the observed values of y_{max}/y_c were underpredicted by more than 5 percent (Maynord 1996). The above equation is similar to the one recommended in NEH654.09 for bend scour. In fact, the values of y_{max}/y_c predicted by these relationships vary by less than 25 percent for FS = 1.19 and $5 < Rc/W_1 < 10$. The bend scour equation from NEH654.09 is slightly more conservative than the Maynord (1996) equation. Only one of the two equations should be used, even if the outside of the bend is protected.

Figure TS14B–12 Definition of recommended length for protection downstream from a bend apex, L_p

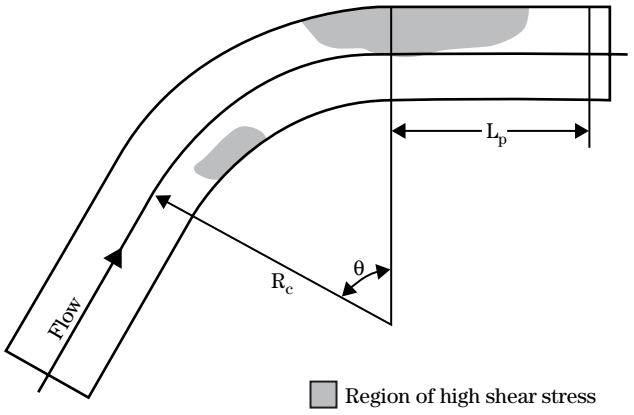
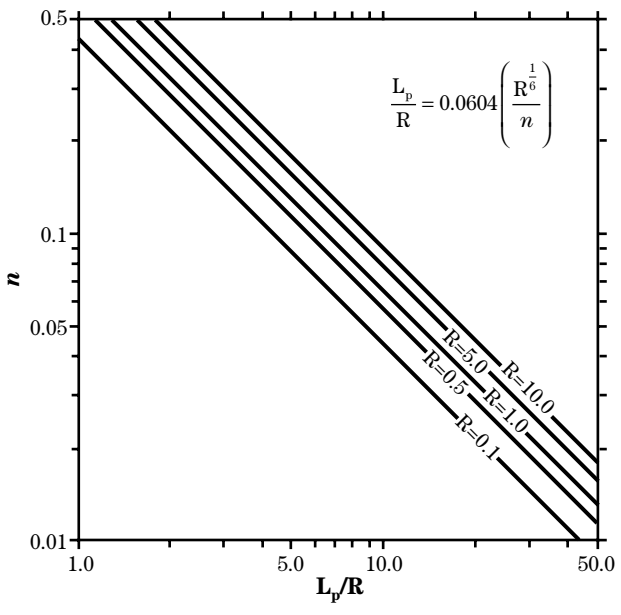


Figure TS14B–13 Recommended length of protection divided by hydraulic radius, L_p/R, as a function of Manning roughness coefficient for the bend, n, and hydraulic radius, R



Bedform scour

Process description

Mobile riverbeds deform to produce ripples, dunes, and antidunes at specific levels of shear stress for a given sediment size (fig. TS14B–14). Most textbooks also recognize large bars (forms having length equal to the channel width or greater) as a type of bedform, but reliable predictors for bars have not been developed. In practice, bedforms other than bars are uncommon in channels dominated by sediments coarser than sand. Dunes and antidunes in sand beds can result in additional scour, since they migrate by a systematic process of erosion and deposition (ripples are too small to be significant), controlled by flow velocities. The passage of a large dune may increase local scour depths as much as 30 percent.

Many attempts have been made to develop relationships to predict the type and dimensions of bedforms based on the bed sediment gradation and the imposed flow. In general, scour analysis involves the use of a bedform predictor that is related to bedform type and amplitude. Half of the amplitude is then assumed to contribute to total scour. Some types of bedforms, however, often occur side-by-side in a cross section or reach of a natural stream. Nonetheless, scour computations normally focus on either dunes or antidunes, which have the greatest amplitude.

Bedform predictors

Water flowing over an erodible bed can produce a variety of configurations. Van Rijn (1984) suggested that dunes would form when:

$$D_* = D_{50}\left(\frac{\Delta S_g g}{\nu^2}\right)^{\frac{1}{3}} > 10 \qquad \text{(eq. TS14B–37)}$$

and

$$3 < T_{ts} < 15 \qquad \text{(eq. TS14B–38)}$$

where:

$$T_{ts} = \frac{\tau_s^* - \tau_c^*}{\tau_c^*} \qquad \text{(eq. TS14B–39)}$$

D_* = dimensionless sediment size
D_{50} = median grain size, ft (m) (note units)
g = acceleration of gravity, 32.2 ft/s^2 (9.81 m/s^2)
ν = kinematic viscosity of water, ft^2/s (m^2/s)
T_{ts} = dimensionless transport-stage parameter
τ_s^* = bed shear stress due to skin or grain friction, lb/ft^2 (N/m^2), which may be computed by

$$\tau_s^* = \frac{\rho g u^2}{\left[18\log\left(\frac{12R}{3D_{90}}\right)\right]^2} \qquad \text{(eq. TS14B–40)}$$

where:
u = mean flow velocity, ft/s (m/s)
R = hydraulic radius, ft (m)
D_{90} = size larger than 90 percent of the bed material by weight, ft (m) (note units)
τ_c^* = critical shear stress for motion from the Shields diagram, which may be computed by $\tau_c^* = 103.0\ \theta D_{50}$ for D_{50} in ft and τ_c^* in lb/ft^2 ($\tau_c^* = 16{,}187\ \theta D_{50}$ for D_{50} in m and τ_c^* in N/m^2)

where:
θ = dimensionless Shields stress ranging from 0.02 to 0.10 for sands and larger sediments. (See compilation of values by Buffington and Montgomery (1997) for appropriate value or use the following equation to compute a value.):

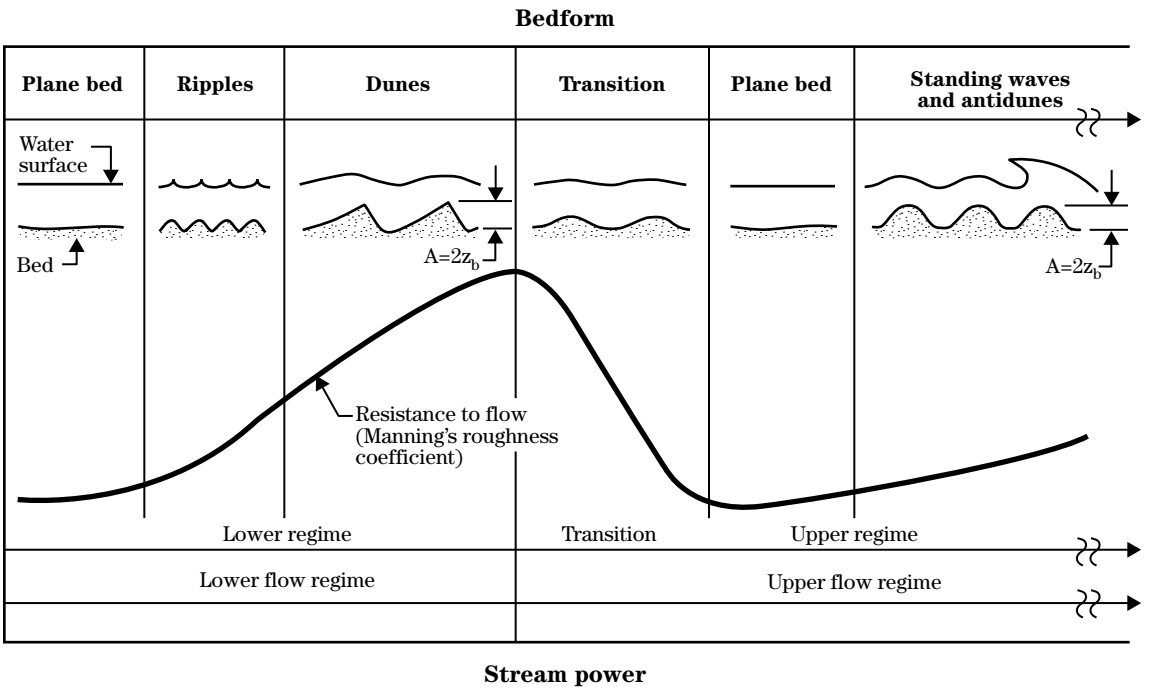
$$\theta_c = \frac{0.24}{D_*} + 0.055\left[1 - \exp\left(-0.02D_*\right)\right] \qquad \text{(eq. TS14B–41)}$$

where:

$$D_* = D_{50}\left(\frac{1.65g}{\nu^2}\right)^{\frac{1}{3}} \qquad \text{(eq. TS14B–42)}$$

where:
D_* = dimensionless sediment size
D_{50} = median grain size in ft (m)
g = acceleration of gravity, 32.2 ft/s^2 (9.81 m/s^2)
ν = kinematic viscosity of water, ft^2/s (m^2/s)

Figure TS14B–14 Relative relationships between progression of alluvial bedforms and flow intensity



Transitional bedforms occur for $15 < T_{ts} < 25$, and antidunes occur when $T_{ts} > 25$. These relationships may be used to determine what type of bedform will occur, given design conditions. Additional complexities arise due to the influence of water temperature and suspended sediment concentration on viscosity, blanketing of coarse sediment beds by sands during certain events, and the fact that mean flow velocity is governed by total flow resistance, which itself is a function of bedform type and geometry.

If the above analysis indicates that dunes will occur, dune height may be computed by:

$$\Delta = 0.11 D_{50}^{0.3} y^{0.7} \left(1 - \exp\left(-0.5 T_{ts}\right)\right)\left(25 - T_{ts}\right) \qquad \text{(eq. TS14B–43)}$$

where:
Δ = dune height, ft (m)
y = mean flow depth, ft (m)

Similar relationships for antidunes are not available, but the flow depth may be used as a conservative estimator of maximum antidune height. For either dunes or antidunes, the scour depth is assumed to be equal to half of the bedform height:

$$z_{bf} = \frac{\Delta}{2} \qquad \text{(eq. TS14B–44)}$$

Some empirical formulas (eq. TS14B–35) that are based on data sets from sand-bed streams implicitly include bedform scour. Usually bedform scour is much smaller in magnitude than other types of scour in sand-bed rivers.

Consult reviews by Garcia (in press) and Yang (1996) for more information on bedforms.

Scour associated with structures

Structures that span the full width of the channel

Structures that span the full width of the channel include sills, grade control structures, and structures comprised of boulders. The latter are intended to create step-pool morphology in steep, coarse-bed streams. Sills may be thought of as very low weirs, and grade control structures are higher weirs with associated appurtenances. These are used for bed erosion control and pool habitat development (fig. TS14B–15). Figure TS14B–15(a) shows a weir immediately after construction in a sand- and gravel-bed stream. The view shown is facing upstream. Central notch was constructed with invert at existing streambed elevation, and figure TS14B–15(b) is facing downstream from the notch about 6 months later.

Figure TS14B–15 Scour associated with low stone weir

(a)

(b)

Predicting scour depths downstream from weirs and grade control structures is too complex for theoretical calculations. Empirical formulas are used and are based on laboratory flume tests and field data. The scour equations are intended to allow prediction of scour depths in unprotected noncohesive alluvial beds. Commonly, grade control structures are built with preformed, stone-protected downstream scour holes (also called stilling basins). In some cases, these basins are sized using scour prediction equations. Since the equations provided below are empirical formulas, the engineer should become familiar with the original references and apply the formulas with care if the project falls outside the ranges of parameters used to generate them. A more comprehensive treatment of this topic is provided by Simons and Sentürk (1992).

Sills

Series of relatively low weirs (sills) are often used to develop pool habitats and to prevent mild to moderate bed degradation. Often these structures are installed by excavating a trench in the bed perpendicular to the flow and placing the structure into the trench so that the initial crest elevation is at bed elevation. Subsequent erosion produces a pool-and-riffle profile. Lenzi et al. (2002) reviewed work by others and conducted a series of flume experiments, resulting in different formulas for low gradient (slope ≤ 0.02) and high gradient (S ≥ 0.08) mountain streams.

For low gradient streams, the scour depth, z_s (fig. TS14B–16), is given by:

$$\frac{z_s}{H_s} = 0.180\frac{a_1}{\Delta S_g D_{50}} + 0.369 \qquad \text{(eq. TS14B–45)}$$

and the length of the scour pool, l_p (based only on tests with gravel sediments) is given by:

$$\frac{l_p}{H_s} = 1.87\frac{a_1}{\Delta S_g D_{50}} + 4.02 \qquad \text{(eq. TS14B–46)}$$

while for high gradient streams, z_s is given by:

$$\frac{z_s}{H_s} = 0.4359 + 1.4525\left(\frac{a_1}{H_s}\right)^{0.8626} + 0.0599\left(\frac{a_1}{\Delta S_g D_{95}}\right)^{1.4908} \qquad \text{(eq. TS14B–47)}$$

and the length of the scour pool is given by:

$$\frac{l_p}{H_s} = 4.479 + 0.023\left(\frac{a_1}{H_s}\right)^{-1.808} + 2.524\left(\frac{a_1}{\Delta S_g D_{95}}\right)^{1.129} \qquad \text{(eq. TS14B–48)}$$

where:

z_s = depth of scour downstream from structure, ft (m), measured from the crest of the structure to the lowest point within the scour pool

H_s = specific energy of critical flow over the sill, ft (m), where:

$$H_s = 1.5 \times \sqrt[3]{\frac{q^2}{g}} \qquad \text{(eq. TS14B–49)}$$

where:

q = flow per unit width over the sill at design discharge, $ft^3/s/ft$ ($m^3/s/m$)

g = acceleration of gravity, 32.2 ft/s^2 (9.81 m/s^2)

a_1 = the "morphological jump" =

$$a_1 = \left(S_o - S_{eq}\right)L_s$$

where:

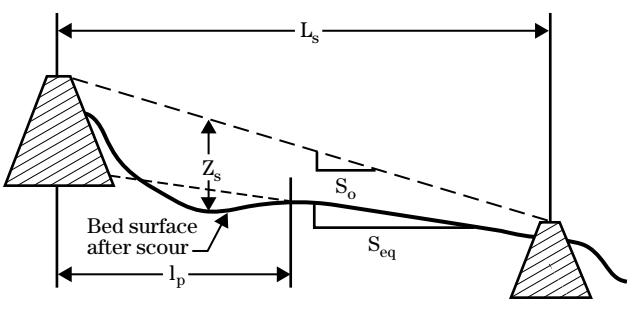
S_o = initial longitudinal bed slope

L_s = horizontal distance between sills, ft (m) (fig. TS14B–16)

S_{eq} = equilibrium bed slope after scour, which may be estimated by:

$$S_{eq} = \frac{\theta_c \Delta S_g D_{50}}{y} \qquad \text{(eq. TS14B–50)}$$

Figure TS14B–16 Definition sketch for computing scour associated with sills



where:

θ_c = critical dimensionless shear stress or-Shields constant, which may be computed for a given sediment size using equation TS14B–41

ΔS_g = relative submerged density of bed-material sediments

D_{50} = median size of bed-material sediments, ft (m) (note units)

l_p = length of scour pool, ft (m)

The formulas were based on data with $0.225 < a_1/H_s < 1.872$ and $0.161 < a_1/\Delta D_{95} < 1.150$, and any application outside these ranges should be done with greatest care. Subsequent application by Lenzi, Comiti, and Marion (2004) to a mountain river predicted scour hole depth below 26 bed sills accurately, but overpredicted scour hole length.

Step-pool structures

Thomas et al. (2000) studied natural step-pools in eight coarse grained mountain streams in Colorado and developed regression equations for design of step-pool structures in steep, boulder-bed streams (fig. TS14B–17):

$$\frac{z_s}{W} = -0.0118 + 1.394\left(\frac{h_d}{W}\right) + 5.514\left(\frac{Sq_{25}}{W^{\frac{3}{2}}\sqrt{g}}\right) \qquad \text{(eq. TS14B–51)}$$

and

$$\frac{l_p}{W} = 0.409 + 4.211\left(\frac{h_d}{W}\right) + 87.341\left(\frac{Sq_{25}}{W^{\frac{3}{2}}\sqrt{g}}\right) \qquad \text{(eq. TS14B–52)}$$

where:

z_s = depth of scour downstream from structure, ft (m), measured from the crest of the structure to the lowest point within the scour pool

W = average active channel width, ft (m)

h_d = height of step crest above controlling bed elevation at downstream end of pool, ft (m)

S = average channel bed slope

q = flow per unit width over the sill at design discharge (q_{25} is for 25-yr discharge), $ft^3/s/ft$ ($m^3/s/m$)

g = acceleration of gravity, 32.2 ft/s^2 (9.81 m/s^2)

l_p = length of scour pool, ft (m)

Grade control structures and weirs

Weirs such as grade control structures (fig. TS14B–18) differ from sills in that they are built with crest elevations well above the existing bed. They normally produce backwater effects at baseflow. Several empirical approaches are available for computing the depth of scour in unprotected, noncohesive beds downstream from a vertical weir. Most of these equations were originally developed to compute the depth of scour downstream from dams. The Veronese (1937) equation yields an estimate of erosion measured from the tailwater surface to the bottom of the scour hole:

$$y + z_s = Kh_d^{0.225}q^{0.54} \qquad \text{(eq. TS14B–53)}$$

where:

y = average depth of flow in channel downstream from scour hole, ft (m)

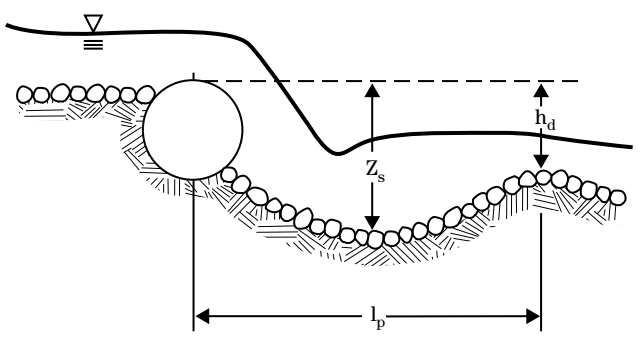
z_s = depth of scour, ft (m)

K = a coefficient = 1.32 for U.S. units (1.90 for SI units)

h_d = vertical distance between water surface elevation upstream and downstream from the weir, ft (m)

q = discharge per unit width, $ft^3/s/ft$ ($m^3/s/m$)

Figure TS14B–17 Definition sketch for computing scour associated with step-pool structures



The Veronese (1937) equation was modified by Yildiz and Üzücek (1994) to include the angle of the weir overfall jet, α

$$y + z_s = Kh_d^{0.225} q^{0.54} \cos\alpha \qquad \text{(eq. TS14B–54)}$$

where:

α = angle the incident jet makes with the vertical. A vertical overfall of water from a cantilevered pipe or sharp-crested weir would have α=0.

Neither version of the Veronese equation contains any expression that reflects the erodibility of the bed, which intuitively seems to be a major deficiency. A more recent formula for scour produced by a free falling jet addresses this issue (Mason and Arumugam 1985). The form is limited to SI units:

$$z_s = K \frac{q^a h_d^b y_t^{0.15}}{g^{0.30} D_m^{0.10}} \qquad \text{(eq. TS14B–55)}$$

where:

z_s = depth of scour (m)

K = $6.42 - 3.2h_d^{0.10}$ (eq. TS14B–56)

q = discharge per unit width ($m^3/s/m$)

h_d = vertical distance between water surface elevation upstream and downstream from the weir (m)

y_t = tailwater depth above original ground surface (m)

a = $0.6–h_d/300$ (eq. TS14B–57)

b = $0.15\ h_d/200$ (eq. TS14B–58)

g = acceleration of gravity, 9.81 m/s^2

D_m = mean bed-material particle size, m (note units). In the case of beds made of rock, a value of 0.25 meter is used.

Figure TS14B–18 (a) Low- and (b) high-drop grade control structures

(a)

(b)

D'Agostino and Ferro (2004) presented a review of previous work dealing with prediction of scour downstream from grade control structures. In addition, they compiled available data sets and analyzed them using stepwise regression to produce a function of dimensionless variables that were formed, using dimensional analysis. They proposed the following relationship for computing the maximum scour depth (fig. TS14B–19):

$$\frac{z_s}{y_w} = 0.540\left(\frac{W_w}{y_w}\right)^{0.593}\left(\frac{y_t}{h_d}\right)^{-0.126} A_{50}^{0.544}\left(\frac{D_{90}}{D_{50}}\right)^{-0.856}\left(\frac{W_w}{W}\right)^{-0.751}$$

(eq. TS14B–59)

where:

y_w = vertical distance between weir crest and upstream channel bed, ft (m)
W_w = width of weir crest, ft (m)
y_t = tailwater depth above original ground surface, m
h_d = difference in water surface elevation upstream of weir and downstream from weir, ft (m)
A_{50} = a dimensionless quantity defined below
D_{50} = median size of bed material, ft (m) (note units)
D_{90} = size of bed material larger than 90 percent of the bed by weight, ft (m) (note units)
W = flow width at design discharge, ft (m)

The quantity A_{50} is given by:

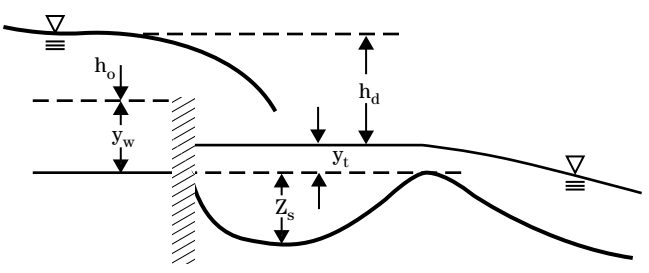
$$A_{50} = \frac{Q_d}{W_w y_w \sqrt{gD_{50}\Delta S_g}}$$

(eq. TS14B–60)

where:

ΔS_g = relative submerged density of bed-material sediments ≅ 1.65
Q_d = design discharge, ft^3/s (m^3/s)

Figure TS14B–19 Definition sketch for computing scour associated with grade control structures



The following relationship was recommended for estimating the horizontal distance between the weir and the deepest point in the scour hole:

$$\frac{L_s\,max}{y_w} = 1.616\left(\frac{W_w}{y_w}\right)^{0.662}\left(\frac{y_t}{h_d}\right)^{-0.117} A_{90}^{0.455}\left(\frac{W_w}{W}\right)^{-0.478}$$

(eq. TS14B–61)

The quantity A_{90} is similar to A_{50}:

$$A_{90} = \frac{Q}{W_w y_w \sqrt{gD_{90}\Delta S_g}}$$

(eq. TS14B–62)

Sloping drop structures such as rock ramps or Newbury riffles may be attractive options in some stream restoration projects, particularly from an aesthetic and fish passage standpoint. Laursen, Flick, and Ehlers (1986) ran a limited number of flume experiments with sloping sills with slopes of 4H:1V and produced the following relationship:

$$\frac{y_2}{y_c} = 4\left(\frac{y_c}{D_s}\right)^{0.2} - 3\left(\frac{D_r}{y_c}\right)^{0.1}$$

(eq. TS14B–63)

where:

y_2 = depth of water in downstream channel after scour, ft (m)
y_c = critical flow depth for the design unit discharge, ft (m)
D_s = characteristic bed sediment size, assumed to be median D_{50}, ft (m) (note units)
D_r = characteristic size of rock or riprap used to build the sloping structure, assumed to be median D_{50}, ft (m) (note units).

The depth of scour, z_s, is given by:

$$z_s = y_2 - y_1$$

(eq. TS14B–64)

where:

y_1 = depth of water in downstream channel before scour, ft (m)

The analysis and design of grade control structures is also described in NEH654 TS14G.

Structures that partially span the channel

Structures that protrude from one bank into the channel include groins (groynes), spur dikes (spurs), deflectors, bank barbs, and bendway weirs. Kuhnle, Alonso, and Shields (1999, 2002) conducted a series of clear-water, steady-flow, movable-bed flume studies using various spur dike geometries and measured the depth and volume of scour adjacent to the spurs. Empirical formulas for scour depth were developed based on earlier work by Melville (1992). The Melville formulas produce scour depth predictions that are likely conservatively large for prototype conditions. Kuhnle also developed a formula for scour hole volume, and both of his formulas produced acceptable estimates for models of paired current deflectors (Biron et al. 2004). Scour volume is of interest if spurs or deflectors are being used to create pool habitats. Figure TS14B–20(a) shows flags delineating scour hole of short spur, and (b) shows a scour hole downstream from a similar spur in the same reach 1 year after a low extension was added (project described by Shields, Bowie, and Cooper 1995).

Figure TS14B–20 Scour associated with stone spur dike

(a)

(b)

The Kuhnle formulas are:

$$\frac{z_s}{y} = K_1 \left(\frac{L_c}{y} \right)^a \qquad \text{(eq. TS14B–65)}$$

and

$$\frac{V_s}{z_s^3} = K_2 \left(\frac{L_c}{y} \right)^b \qquad \text{(eq. TS14B–66)}$$

where:

z_s = maximum depth of local scour associated with spur dike, ft (m)

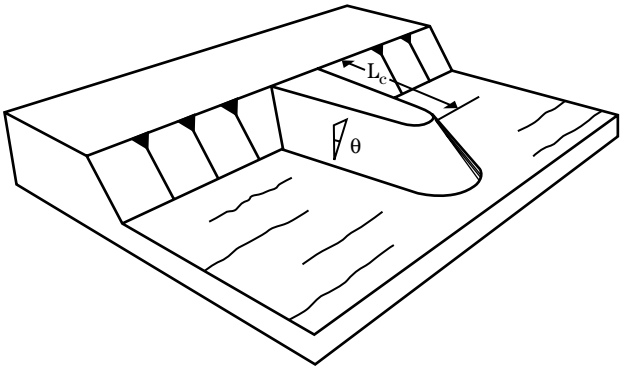
y = mean flow depth in approaching flow, ft (m)

L_c = length of spur crest measured perpendicular to flow direction, ft (m) (fig. TS14B–21)

V_s = volume of scour hole, ft^3 (m^3)

The coefficient K_1 is a dimensionless constant reflecting the effect of flow intensity, flow depth, sediment size, sediment gradation, and channel and spur geometry. Kuhnle suggested a value of $K_1 = 2$ when the water surface elevation is below the spur crest and $K_1 = 1.41$ when the spur is submerged.

Figure TS14B–21 Definition sketch showing crest length, L_c, and side slope angle, θ, for spur dikes



The exponent a is a dimensionless exponent that varies with L_c/y. It has a value of a = 1 for $L_c/y < 1$, a = ½ for $1 < L_c/y < 25$, and a = 0 for $L_c/y > 25$.

K_2 = dimensionless coefficient that varies with the angle the spur crest makes with the approach flow

K_2 = 17.106 for perpendicular spurs, and K_2 = 12.11 for spurs that are at a nonperpendicular angle (45° or 135°)

b = dimensionless exponent that varies with spur crest angle. b = –0.781 for perpendicular spurs, and b = 0 in other cases

Rahman and Haque (2004) suggested that K_1 be modified to reflect the shape of the spur cross section for shorter spurs ($L_c/y < 10$):

$$K_1 = 0.75\left(1 + \frac{2\tan\phi}{\tan\theta}\right)^{-\frac{1}{2}} \qquad \text{(eq. TS14B–67)}$$

where:

ϕ = angle of repose of bed sediment

θ = side slope of spur structure (fig. TS14B–21)

These formulas produce large scour depths for long spurs. Richardson and Davis (2001) suggest an alternative approach that may be used for such cases.

The analysis and design of spurs and deflectors is presented in more detail in NEH654 TS14H.

Table TS14B–11 presents a summary of scour analyses and applicability to various bed types.

Table TS14B–11 Summary of scour analyses and applicability to various bed types

Predominant bed material	Type of analysis								
	Long-term bed elevation change		General scour			Local scour			All types of scour
	Armoring analysis	Equilibrium slope	Contraction scour	Bend scour	Bedform scour	Bridge pier and abutment scour	Structures that span the channel	Structures that do not fully span the channel	Numerical modeling
Clay or silt, cohesive	X	✓ Regional regressions (fig. 8)	O	O	X	O	O	O	O
Sand	X	✓No change in bed-material sediment supply—(12) ✓ Reduction in bed sediment supply—(13)	✓ Empirical relations (21–23)		✓ Dunes (43–44)		✓ Vertical drops (53–55, 59)		✓
Fine gravel <6 mm	✓ (2–4)	✓ Elimination of bed-material sediment supply—(7)	✓ Live-bed conditions (25–26) ✓ Clear-water conditions (31–32)	✓ (33–36)	✓ Antidunes (43, assume Δ = y)	✓Richardson and Davis (2001)	✓ Ramps or sloping drops (63–64)	✓ (65, 67)	✓
Gravel >6 mm, cobble		✓ (14–17)		X	X		✓ Sills (45, 47)	O	✓
Boulders	O	O	O	X	X	O	✓ Step-pool structures (51)	O	O

✓ = applicable, X = process not generally observed in this environment, O = process may occur, but analysis is beyond the state of the art. Numbers in parentheses refer to equations in the text. Gray shading indicates techniques with low precision and high uncertainty.

Example computations

Sand-bed reach

A stream restoration project is planned for a sand-bed channel that is currently straight and extremely wide due to historic channelization and straightening. The channel will be narrowed by 30 percent, and stone spur dikes (also known as bank barbs) will be added for stabilization and scour pool development. Side slope of spurs will be 2H:1V, and crests will be submerged at design discharge. Sediment supply from upstream is expected to be unchanged during the life of the project.

Given:

ρ water	1,000 kg/m^3	1.94 slug/ft^3
ρ solids	2,630	5.10 slug/ft^3
relative submerged density, ΔSg	1.63	1.63 constant in
Manning's equation	1	1.486 C in Strickler
equation	0.034	
Shields constant θ_c	0.038215	0.038215 grain
roughness k_s	2.8 mm	0.009 ft
D_{95}	1.5 mm	0.005 ft
D_{90}	0.96 mm	0.003 ft
D_{84}	0.8 mm	0.003 ft
D_{mean}	0.28 mm	0.001 ft
D_{50}	0.3 mm	0.0010 ft
bed sediment internal friction angle	45 deg	0.785 rad
distance to downstream base level control	2,000 m	6,562 ft
Manning's n	0.027 s/m$^{(1/3)}$	0.027 s/ft$^{(1/3)}$
design discharge, Q_d	392.2 m^3/s	13849 ft^3/s
flow width	60 m	197 ft
channel width, W	70 m	230 ft
mean flow depth, y	3.0 m	9.8 ft
hydraulic radius, R	3.0 m	9.8 ft
mean streamwise velocity, u	2.2 m/s	7.1 ft/s
unit discharge, q	6.5 m^3/s/m	70 ft^3/s/ft
unit discharge, 25 yr, q_{25}	30 m^3/s/m	0.86 ft^3/s/ft
existing bed slope, S	0.0008 m/m	0.0008 ft/ft
bend radius of curvature, Rc	1,000 m	3,281 ft
L, length of spur crest	20 m	65.6 ft
spur side slope	2H:1V	0.46 rad
spur crest above water surface?	N	

Find:
Total predicted scour depth

Step 1 Compute bed elevation change due to reach-scale degradation based on equilibrium slope.

a. Compute the smallest armor particle size, D_x using equation TS14B–4.

$$D_x = K\left(\frac{yS_e}{\Delta S_g}\right)^a \left(\frac{U_*}{\nu}\right)^b$$

$$U_* = (gyS_e)^{0.5}$$

Assume $S_e = S_o$

$$U_* = \sqrt{32.2 \times 9.8 \times 0.0008} = 0.50 \text{ ft/s}$$

$$\text{Particle Re} = \frac{U_* D_{50}}{\nu} = \frac{0.50 \times 0.001}{1.05 \times 10^{-5}} = 48$$

Therefore, K = 27, a = 0.86, b = –0.14.

$$D_x = 27\left(\frac{9.8 \times 0.0008}{1.63}\right)^{0.86} \left(\frac{0.50 \times}{1.05 \times 10^{-5}}\right)^{-0.14}$$
$$= 0.06 \text{ ft}$$
$$= 18 \text{ mm}$$

The bed does not contain material large enough to form an armor layer.

Step 2 Compute depth of scour needed to produce an equilibrium slope assuming no change in sediment discharge into the reach.

a. Using Yang (1996) regression equation, compute sediment discharge (eqs. TS14B–8 to TS14B–11).

$$a = 0.025n^{(2.39-0.8\log D_{50})}\left(D_{50} - 0.07\right)^{-0.14}$$
$$= 0.025\left(0.027\right)^{(2.39-0.8\log D_{50})}\left(0.3 - 0.07\right)^{-0.14}$$
$$= 1.21 \times 10^{-6}$$

$$b = 4.93 - 0.74\log D_{50}$$
$$= 4.93 - 0.74\log\left(0.3\right)$$
$$= 5.32$$

$$c = -0.46 + 0.65\log D_{50}$$
$$= -0.46 + 0.65\log\left(0.3\right)$$
$$= -0.80$$

$$q_s = au^b y^c$$

$$q_s = \left(1.21 \times 10^{-6}\right)\left(7.1\right)^{5.32}\left(9.8\right)^{-0.80} = 0.0066 \text{ ft}^2\text{/s}$$

b. Compute equilibrium slope (eq. TS14B–12).

$$S_{eq} = \left(\frac{a}{q_s}\right)^{\frac{10}{3(c-b)}} q^{\frac{2(2b+3c)}{3(c-b)}} \left(\frac{n}{K}\right)^2$$

$$S_{eq} = \left(\frac{1.21 \times 10^{-6}}{0.066}\right)^{-0.54} 70^{-0.89}\left(\frac{0.027}{1.486}\right)^2 = 0.00083$$

Since the existing channel slope is approximately equal to the equilibrium slope, long-term degradation should be minimal.

Step 3 Compute contraction scour.

a. Check for live bed conditions using equation TS14B–24.

$$V_c = Ky^{\frac{1}{6}} D_{50}^{\frac{1}{3}}$$

$$V_c = 11.17\left(9.8\right)^{\frac{1}{6}}\left(0.001\right)^{\frac{1}{3}} = 1.60 \text{ ft/s}$$

Since u = 7.1 ft/s > 1.6 ft/s, live bed conditions occur.

b. Compute fall velocity with equations TS14B–27 to TS14B–30.

$$A = \frac{\Delta S_g g D_s^3}{\nu^2}$$

$$A = \frac{1.63 \times 32.2 \times \left(0.001\right)^3}{\left(1.05 \times 10^{-5}\right)^2} = 500$$

$$K_1 = 0.055\tanh\left[12A^{-0.59}\exp\left(-0.0004A\right)\right]$$

$$K_1 = 0.055\tanh\left[12\left(500\right)^{-0.59}\exp\left(-0.0004\left(500\right)\right)\right] = 0.014$$

$$K_2 = 1.06\tanh\left[0.016A^{0.50}\exp\left(-\frac{120}{A}\right)\right]$$

$$K_2 = 1.06\tanh\left[0.016\left(500\right)^{0.50}\exp\left(-\frac{120}{500}\right)\right] = 0.291$$

$$\omega = \frac{K_1 \Delta S_g g D_s^2}{\nu} + K_2\sqrt{\Delta S_g g D_s}$$

$$\omega = \frac{0.014 \times 1.63 \times 32.2 (0.001)^2}{\left(1.05 \times 10^{-5}\right)} + (0.291)\sqrt{1.63 \times 32.2 \times 0.001} = 0.135 \text{ ft/s}$$

c. Compute U_*/ω.

$$U_* = \sqrt{32.2 \times 9.8 \times 0.0008} = 0.50 \text{ ft/s}$$

$$\frac{U_*}{\omega} = \frac{0.50}{0.135} = 3.72 \text{ ft/s}$$

d. Using U_*/ω, look up a → a = 0.69 (table TS14B–10).

e. Compute y_2 with equation TS14B–26, assuming $y_1 = y_0 = 9.8$ ft, and since $Q_1 = Q_2$.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}} \left(\frac{W_{b1}}{W_{b2}}\right)^a$$

$$\frac{y_2}{9.8} = \left(\frac{60}{40}\right)^{0.69} = 1.32$$

$$y_2 = 1.32 \times 9.8 = 13.0 \text{ ft}$$

$$z_c = y_2 - y_o = 13.0 - 9.8 = 3.2 \text{ ft}$$

Step 4 Compute bedform scour.
For dunes to form (eq. TS14B–37):

$$D_* = D_{50}\left(\frac{\Delta S_g g}{\nu^2}\right)^{\frac{1}{3}} > 10$$

$$D_* = 0.001\left(\frac{1.63 \times 32.2}{\left(1.05 \times 10^{-5}\right)^2}\right)^{\frac{1}{3}} = 7.8 < 10$$

Dunes should not form.

Step 5 Compute local scour at spur dikes using equations TS14B–65 and TS14B–67.

$$K_1 = 0.75\left(1 + \frac{2\tan\phi}{\tan\phi}\right)^{-\frac{1}{2}}$$

$$K_1 = 0.75\left(1 + \frac{2\tan(45)}{\tan(27)}\right)^{-\frac{1}{2}} = 0.34$$

$$\frac{z_s}{y} = K_1\left(\frac{L_c}{y}\right)^a$$

since 1< L/y < 25, a = 0.5

$$\frac{z_s}{y} = 0.34\left(\frac{65.6}{9.8}\right)^{0.5} = 0.88$$

$$z_s = 0.88y$$
$$= 0.88 \times 9.8$$
$$= 8.6 \text{ ft}$$

Step 6 Compute total scour (eq. TS14B–1).

$$z_t = FS\left[z_{ad} + z_c + z_b + z_{bf} + z_s\right]$$

$$z_t = 1.3\left[0 + 3.2 + 0 + 0 + 8.6\right] = 15.4 \text{ ft}$$

Compare with Blodgett (1986) using equation TS14B–21.

$$z_t \max = KD_{50}^{-0.115}$$

$$z_t \max = 6.5(0.001)^{-0.115} = 14.4 \text{ ft}$$

The predicted z_t value is close to this value. Values of z_t predicted using the Lacey and Blench formulas are somewhat smaller, 5.7 feet and 10.3 feet, respectively.

Gravel-bed reach

Scour analysis is needed to support design of instream habitat structures for a gravel-bed river with a single-thread, nearly straight channel. Low weirs will be placed in a shallow reach to develop pool habitats.

The reach appears to be actively degrading, with a base level control (confluence with larger river) 6,562 feet (2,000 m) downstream. Sediment supply from upstream has been greatly reduced due to advanced urban development.

Given:

relative submerged density, delta S_g	1.63	1.63
angle of repose of sediment, ϕ	45 deg	0.79 rad
constant in Manning's equation	1	1.486
C in Strickler equation	0.034	
Shields constant θ_c	0.056	0.055653
grain roughness k_s	87.5 mm	0.287 ft
D_{95}	175 mm	0.574 ft
D_{90}	65 mm	0.213 ft
D_{84}	25 mm	0.082 ft
D_{mean}	15 mm	0.049 ft
D_{50}	13 mm	0.043 ft
bed sediment internal friction angle	45 deg	0.785 rad
distance to downstream base level control	2,000 m	6,562 ft
Manning's n	0.030 s/m$^{(1/3)}$	0.030 s/ft$^{(1/3)}$
design discharge, Q_d	40.5 m^3/s	1,430 ft^3/s
flow width	18 m	60 ft
channel width, W	19 m	62 ft
mean flow depth, y	1.2 m	3.9 ft
hydraulic radius, R	1.2 m	3.9 ft
mean streamwise velocity, u	1.8 m/s	6.1 ft/s
unit discharge, q	2.2 m^3/s/m	24 ft^3/s/ft
existing bed slope, S	0.0024 m/m	0.0024 ft/ft
bend radius of curvature, Rc	1,000 m	3,281 ft
distance between weirs, L	250 m	820.3 ft
weir height above up streambed, y_w	0.3 m	1.0 ft
weir width, W_w	15 m	49.2 ft
difference in upstream and downstream water surface	0.5 m	1.6 ft
water depth above uneroded bed, y_d	0.7 m	2.3 ft
angle of the overfall jet with the vertical	0 Deg	0.0 rad

Find:
Total predicted scour depth

Step 1 Compute bed elevation change due to reach-scale degradation based on equilibrium slope.

a. Compute the smallest armor particle size, D_x using equation TS14B–4.

$$D_x = K\left(\frac{yS_e}{\Delta S_g}\right)^a\left(\frac{U_*}{\nu}\right)^b$$

$U_* = (gyS_e)^{0.5}$. Assume $S_e = S_o$

$$U_* = \sqrt{32.2\times 3.9\times 0.0024} = 0.55 \text{ ft/s}$$

$$\text{Particle Re} = \frac{U_* D_{50}}{\nu} = \frac{0.55\times 0.043}{1.05\times 10^{-5}} = 2{,}252$$

Therefore, K = 17, a = 1.0, b = 0

$$D_x = 17\left(\frac{3.9\times 0.0024}{1.63}\right) = 0.0976 \text{ ft} = 30 \text{ mm}$$

Particles of this size and larger are present in the bed, so an armor layer can form.

b. Compute T, the active bed layer thickness using equation TS14B–3.

$$T = \frac{D_x}{(1-e)P_x}$$

where the bed porosity given by equation TS14B–5 is:

$$e = 0.245 + \frac{0.0864}{(0.1D_{50})^{0.21}} = 0.245 + \frac{0.0864}{(0.1\times 13)^{0.21}} = 0.327$$

since D_{84} = 30 mm, P_a = 0.16. Therefore,

$$T = \frac{0.0976}{(1-0.327)(0.16)} = 0.91 \text{ ft}$$

c. Compute maximum scour depth limited by armoring, z_x.

$$z_x = T - D_x = 0.91 - 0.098 = 0.81 \text{ ft}$$

Step 2 Compute the depth of scour needed to produce an equilibrium slope. First, find the equilibrium slope.

a. Manning and Shields relation (eq. TS14B–14)

$$S_{eq} = \left[\theta_c D_c \Delta S_g\right]^{\frac{10}{7}}\left(\frac{K}{qn}\right)^{\frac{6}{7}} \quad \text{Let } D_c = D_{50}$$

$$S_{eq} = \left[0.056\times 0.043\times 1.63\right]^{\frac{10}{7}}\left(\frac{1.486}{24\times 0.03}\right)^{\frac{6}{7}} = 0.00068$$

b. Meyer-Peter and Müller (eq. TS14B–15)

$$S_{eq} = K\frac{(D_{50})^{\frac{10}{7}}\, n^{\frac{9}{7}}}{(D_{90})^{\frac{5}{14}}\, q^{\frac{6}{7}}}$$

$$S_{eq} = 60.1\frac{(0.043)^{\frac{10}{7}}\,(0.03)^{\frac{9}{7}}}{(0.065)^{\frac{5}{14}}\,(24)^{\frac{6}{7}}} = 0.0013$$

c. Schoklitsch (eq. TS14B–16)

$$S_{eq} = K\left(\frac{D_m}{q}\right)^{\frac{3}{4}}$$

$$S_{eq} = 0.00174\left(\frac{15}{24}\right)^{\frac{3}{4}} = 0.0012$$

d. Henderson (eq. TS14B–17)

$$S_{eq} = K\, Q_d^{-0.46} D_{50}^{1.15}$$

$$S_{eq} = 0.44(1{,}430)^{-0.46}(0.043)^{1.15} = 0.00042$$

e. Compute bed degradation using equation TS14B–6. Use the average of the first three S_{eq} values computed above = 0.0011.

$$z_{ad} = L(S_{ex} - S_{eq})$$

$$z_{ad} = (6{,}562)(0.0024 - 0.0011) = 8.5 \text{ ft}$$

Since the armor layer is formed after 0.81 ft of degradation, armoring controls.

Step 3 Compute scour downstream from weirs.

a. Using Veronese (1937) formula (eq. TS14B–53):

$$y + z_s = Kh_d^{0.225} q^{0.54}$$

$$y + z_s = 1.32(1.6)^{0.225}(23)^{0.54} = 8.2 \text{ ft}$$

$$\begin{aligned} z_s &= 8.2 - y \\ &= 8.2 - 3.9 \\ &= 4.3 \text{ ft} \end{aligned}$$

b. Using formula of Mason and Arumugam (1985) (eq. TS14B–55):

$$z_s = K \frac{q^a h_d^b y_t^{0.15}}{g^{0.30} D_m^{0.10}}$$

$$z_s = 3.4 \frac{(2.2)^{0.6}(0.5)^{0.15}(0.7)^{0.15}}{(9.8)^{0.30}(0.25)^{0.10}} = 2.7 \text{ m}$$

c. Using formula of D'Agostino and Ferro (2004) (eqs. TS14B–59 and TS14B–60):

$$A_{50} = \frac{Q_d}{W_w y_w \sqrt{g D_{50} \Delta S_g}}$$

$$A_{50} = \frac{1,430}{(49.2)(1.0)\sqrt{32.2 \times 0.043 \times 1.63}} = 19.3$$

$$\frac{z_s}{y_w} = 0.540 \left(\frac{W_w}{y_w}\right)^{0.593} \left(\frac{y_t}{h_d}\right)^{-0.126} A_{50}^{0.544} \left(\frac{D_{90}}{D_{50}}\right)^{-0.856} \left(\frac{W_w}{W}\right)^{-0.751}$$

$$\frac{z_s}{y_w} = 0.540 \left(\frac{15}{0.3}\right)^{0.593} \left(\frac{0.7}{0.5}\right)^{-0.126} (19.3)^{0.544} \left(\frac{0.213}{0.043}\right)^{-0.856} \left(\frac{49.2}{62}\right)^{-0.751} = 7.96$$

$$z_s = 7.96\, y_w = 7.96 \times 0.3 = 2.4 \text{ m}$$

Estimate depth of scour pools below weirs as the maximum of the above three results.

$$z_s = 2.7 \text{ m} = 8.9 \text{ ft}$$

Step 4 Compute total scour for design using equation TS14B–1.

$$z_t = FS\left[z_{ad} + z_c + z_b + z_{bf} + z_s\right]$$

Use a factor of safety of 1.3.

$$z_t = 1.3\left[0.8 + 0.0 + 0.0 + 0.0 + 8.9\right] = 12.6 \text{ ft}$$

Compare with Blodgett (1986) (eq. TS14B–22).

$$z_t(\max) = K D_{50}^{-0.11}$$

$$z_t \max = 6.5(0.043)^{-0.115} = 9.3 \text{ ft}$$

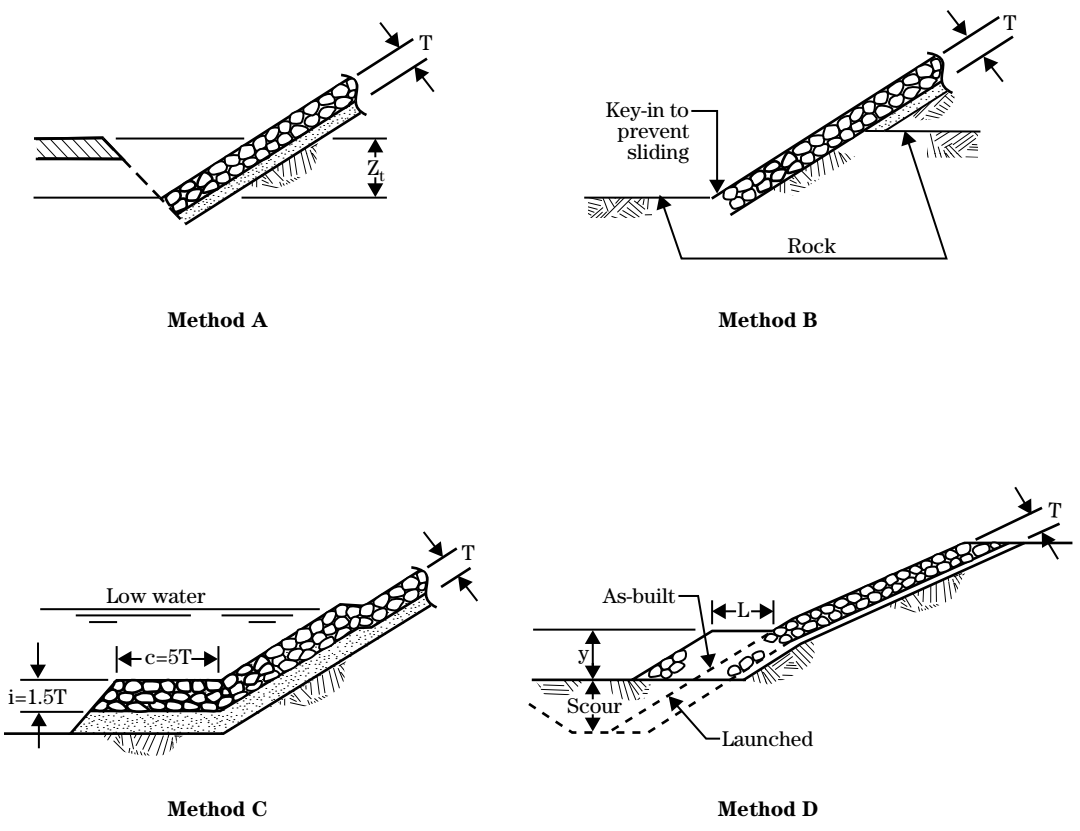
The predicted value of 12.6 feet is well within the scatter about Blodgett's relationship shown in figure TS14B–1. Predicted values of z_t, using the Lacey (1931) and Blench (1970) formulas, are much smaller: 1.4 feet and 3.3 feet, respectively. However, these values are close to the value of z_t mean (fig. TS14B–1) of 2 feet from Blodgett's formula.

Design features and measures to address scour

Structures may be designed to withstand scour in either of two ways (fig. TS14B–22 (USACE 1991b)). They may be extended down into the bed a sufficient distance (dig it in or key it in) to be beneath the projected total scour depth (method A, fig. TS14B–22) or until contact is made with a nonerodible material (method B, fig. TS14B–22). The key-it-in approach (method A) is most often used with armor revetment (Biedenharn, Elliott, and Watson 1997), but is difficult and costly to do in a flowing stream. Conventional excavation is usually not feasible in water depths >10 feet (3 m). Greater water depths usually require dredging or dewatering for construction.

Alternatively, additional loose material (stone) may be incorporated into the structure so that it will self-launch into the scour zone as scour occurs and inhibit deeper scour that would endanger the bank and the rest of the structure (methods C and D, fig. TS14B–22). Method C is recommended for situations where little scour is expected such as in straight, nonbraided reaches that are not immediately downstream from bends. Method D is more robust and is useful when water depths prohibit excavation for a method A type design. No excavation is needed for method D, as toe scour is a substitute for mechanical excavation when this method is used. The self-launching approach (method D) offers the advantage that it provides a built-in indicator of scour as it occurs. However, a self-launching toe requires more material.

Figure TS14B–22 Four methods for designing stone structures to resist failure due to bed scour



These approaches may be used with any type of stone structure. The volume of additional stone required at the toe of a revetment for method D is computed as follows (USACE 1991b):

Assume launch slope = 1V:2H
Revetment toe thickness after launching = 1.5 T_r, where T_r is the thickness of the bank revetment, feet (m), and therefore,

$$V_{stone} = 3.35 T_r z_t \qquad \text{(eq. TS14B–68)}$$

where:

V_{stone} = additional volume of stone added to toe for launching per unit streamwise length of revetment, ft^3/ft (m^3/m)

z_t = total projected scour depth, as before, ft (m)

Variations on the self-launching toe approach include windrow revetments (linear piles of riprap placed along the top bank) and trenchfill revetments (trenches excavated at the low water level and filled with stone).

With several possible choices of structures to counteract scour, designers should select a scour control strategy based on careful consideration of the possible modes of failure, their likelihood, the consequences of each failure mode, and the difficulty of detecting failures in time to correct them. A quantitative strategy for selecting scour control measures based on this approach is described by Johnson and Niezgoda (2004).

List of symbols

a_1 = morphological jump = $(S_o - S_{eq})L_s$ ft (m)

Δ = dune height, ft (m)

D_* = dimensionless sediment size

D_{50} = median bed-material size, mm or ft (m)

D_{90} = size larger than 90 percent of the bed material by weight, mm or ft (m)

D_{90} = size of bed material larger than 90% of the bed by weight, ft (m) (note units)

D_x = the smallest armor size or the size of the smallest nontransportable particle present in the bed material, ft (m)

D_c = diameter of the sediment particle, mm or ft (m)

D_m = mean bed-material particle size, mm or ft (m)

D_s = a characteristic sediment diameter, ft (m)

ΔS_g = change in relative submerged density of bed-material sediments ≅1.65

ΔV = change in volume of bed-material sediment stored or eroded, ft^3 (m^3)

e = porosity of the bed material

ϕ = angle of repose of bed sediment

FS = factor of safety

g = acceleration of gravity, 32.2 ft/s^2 (9.81 m/s^2)

γ_s = specific weight of sediment particles lb/ft^3 (N/m^3)

γ_w = specific weight of water, lb/ft^3 (N/m^3)

h_d = height of step crest above controlling bed elevation at downstream end of pool, ft (m)

h_d = vertical distance between water surface elevation upstream and downstream from the weir, ft (m)

H_s = specific energy of critical flow over the sill, ft (m)

φ = side slope of spur structure

L_r = reach length, ft (m)

L_c = length of spur crest measured perpendicular to flow direction, ft (m)

L_p = recommended length of protection, ft (m)

L_s = horizontal distance between sills, ft (m)

l_p = length of scour pool, ft (m)

$L_s max$ = horizontal distance between weir and deepest point of downstream scour hole, ft (m)

n = Manning's roughness coefficient

ν = kinematic viscosity of water, ft^2/s (m^2/s)

P_x = the fraction of bed material comprised of particles size D_a or larger

q = channel-forming or design discharge per unit width, $ft^3/s/ft$ ($m^3/s/m$)

θ = dimensionless Shields stress

θ_c = critical dimensionless shear stress or Shields constant

Q_d = design discharge, ft^3/s (m^3/s)

Q_s = sediment supply, ft^3/s (m^3/s)

q_s = sediment transport capacity in dimensions of volume per unit width per unit time, ft^2/s (m^2/s)

ρ = Density of water, 1.94 $slugs/ft^3$ (1,000 kg/m^3)

R = hydraulic radius, ft (m)

Rc = bend radius of curvature, ft (m)

S = average channel bed slope

S_e = energy slope

S_e = Slope of energy grade line of main channel, ft/ft (m/m)

S_e = energy slope, then the following analyses may be used to find S_{eq}

S_{eq} = equilibrium channel slope at which sediment particles of size D_c and larger will no longer move

S_{ex} = existing channel slope

S_g = specific gravity of the sediment

T = thickness of the active layer of the bed, ft (m)

τ_c = critical boundary shear stress, lb/ft^2 (N/m^2)

τ_c^* = critical shear stress for motion from the Shields diagram, lb/ft^2 (N/m^2)

τ_o = average bed shear stress, lb/ft^2 (N/m^2)

T_r = thickness of the bank revetment, ft (m)

τ_s^* = bed shear stress due to skin or grain friction, lb/ft^2 (N/m^2)

T_{ts} = dimensionless transport-stage parameter

u = mean streamwise velocity, ft/s (m/s)

U_* = shear velocity = $(gyS_e)^{0.5}$

V_c = critical velocity, ft/s (m/s)

V_{stone} = additional volume of stone added to toe for launching per unit streamwise length of revetment, ft^3/ft (m^3/m)

V_s = volume of scour hole, ft^3 (m^3)

Vs_{in} = volume of bed-material sediment supplied to the reach, ft^3 (m^3)

Vs_{out} = volume of bed-material sediment transported out of the reach, ft^3 (m^3)

W = average active channel width, ft (m)

ω = fall velocity of bed material based on the D_{50}, ft/s (m/s)

W = flow width at design discharge, ft (m)

W_{b1} = bottom width of the upstream channel, ft (m)

W_{b2} = bottom width of the contracted section, ft (m)

W_c = average channel width, ft (m)

W_f = flow width at design discharge, ft (m)

W_i = channel width at bend inflection point, ft (m)

W_w = width of weir crest, ft (m)

y = flow depth, ft (m)

y_c = mean water depth in the crossing upstream from the bend, ft (m)

y_{max} = maximum flow depth in the bend, ft (m)

y_t = tailwater depth above original ground surface, m

y_w = vertical distance between weir crest and upstream channel bed, ft(m)

z_{ad} = bed elevation changes due to reach-scale deposition (aggradation) or general scour (degradation), ft (m)

z_b = scour on the outside of bend, ft (m)

z_{bf} = bedform trough depth, ft (m)

z_c = clear-water contraction scour, ft (m)

z_s = depth of scour downstream from structure, ft (m), measured from the crest of the structure to the lowest point within the scour pool

z_s = local scour depth associated with a structure, ft (m)

z_t = total scour depth, ft (m)

Technical Supplement 14C

Stone Sizing Criteria

Issued August 2007

Cover photo: Stone may be needed as a foundation on which to implement other restoration features such as soil bioengineering practices. Stone may also be needed to form an erosion resistant layer. How large, how thick, and how deeply keyed-in are questions that are addressed in the design.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14C

Stone Sizing Criteria

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Technical Supplement 14C

Stone Sizing Criteria

Purpose

Many channel protection techniques involve rock or stone as a stand-alone treatment or as a component of an integrated system. Stone used as riprap can also be a component of many streambank soil bioengineering projects. Many Federal and state agencies have developed methods and approaches for sizing riprap, and several of those techniques are briefly described in this document. Stone sizing methods are normally developed for a specific application, so care should be exercised in matching the selected method with the intended use. While many of these were developed for application with stone riprap revetments, they are also applicable for other designs involving rock, as well.

Introduction

When the attacking forces of flowing water exceed the resisting forces of the existing channel material, channel protection is needed as part of a restoration design. Channel protection typically ranges from soil bioengineering treatments to more traditional armoring methods. Numerous methods have been developed for the design and sizing of riprap. Several common techniques for estimating the required stone size are briefly outlined in this document. The designer is encouraged to review the complete development of a selected method and assess the relevance of the assumptions behind that selected method to their application. In this document, the words rock and stone are used interchangeably.

Size is one of many considerations when designing riprap for use in protecting channel bed and banks. The designer must also address issues such as material strength, density, angularity, durability, length-to-width ratio, gradation, bedding, piping potential, and channel curvature. These important design and construction considerations are addressed in NEH654 TS14K.

Basic concepts

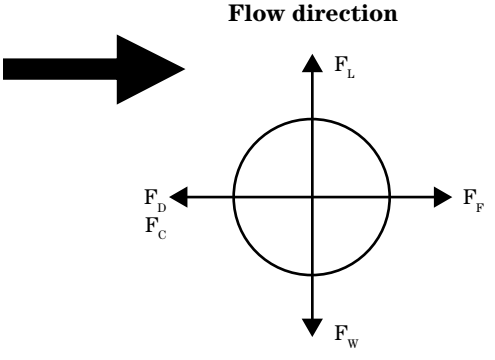
Description of forces on a stone

A rock will be stable until the lift and drag forces of moving water exceed a critical value or threshold. Therefore, for a given rock size subjected to a given force of moving water, there is some unit discharge where the rock will move and become unstable. Forces on a submerged stone, as indicated in figure TS14C–1, typically consist of the force exerted by the flowing water (F_F), drag force (F_D) associated with flow around the object (skin friction and form drag), lift force (F_L) associated with flow around the particle (pressure differences caused by streamline curvature and increased velocity around a particle), submerged weight of the stone (F_W), and resisting force due to the particle interlock and/or contact between stones (F_C).

While some methods are based on a particle force balance, all rock sizing methods are essentially empirical techniques. Field performance data, physical models, and theoretical developments have all contributed to the diverse set of approaches used to determine stable stone sizes for restoration designs.

Velocity-based approaches and boundary shear or stress-based approaches are the two prominent classes of methods that have been used to evaluate the erosion resistance of materials. While shear or stress-based approaches are considered more academically correct, velocity-based methods are still widely used. The design stress and the design discharge do not necessarily represent the same conditions.

Figure TS14C–1 Forces on a submerged stone



Flow conditions

The flow conditions associated with a particular application will have a major influence on selecting the right rock sizing method. While it is difficult to select a single criterion that separates rock sizing methods, high energy and low energy are used in this development. For example, a technique developed for the design of a riprap blanket revetment in a low-energy environment would not necessarily be suitable for estimating the minimum stone size in a high-energy environment, where the stone projects into the flow. Such applications, including instream habitat boulders, grade stabilization, and stream barbs, should be addressed with impinging flow design techniques. Table TS14C–1 lists some of the flow descriptors that can be associated with high- and low-energy flow conditions. Photographs of the different energy conditions where stone is applied as part of the solution are shown in figures TS14C–2 through 4. In figure TS14C–2, riprap is used to control a headcut. Riprap chutes can be used to control erosion from a headcut in a channel or in a side inlet to a channel. Riprap for this type of structure would fall in the steep-slope, high-energy design. Figure TS14C–3 shows riprap used to prevent erosion from flow from a side inlet to a channel. This structure also prevents a headcut from moving into the field. As illustrated in figure TS14C–4, if the toe of the slope is eroding, and it cannot be controlled with bioengineering alone, lining the toe of the slope with stone may be a solution. Riprap for this type of structure would fall in the mild slope, low-energy design.

The appropriate rock sizing method must consider the flow energy associated with the particular application. While there are exceptions, most rock sizing methods were developed for either a high- or low-energy flow condition.

Table TS14C–1 High-energy vs. low-energy conditions

High energy	Low energy
Supercritical flow	Subcritical flow
Steep slope	Mild slope
High turbulence	Low turbulence
Impinging flow	Parallel flow
Rapidly varied flow	Uniform or gradually varied flow
Unsteady flow	Steady flow

Figure TS14C–2 Riprap used to control a headcut

Figure TS14C–3 Riprap used to prevent erosion from flow from a side inlet to a channel

Figure TS14C–4 Toe of the slope lined with stone to control erosion

Sizing techniques

There are many techniques for sizing stone, and each method has advantages and disadvantages. Many techniques were derived under specific conditions and developed for particular applications. While this list is not complete and the description is not exhaustive, several commonly used methods are presented. The designer should review the applicability of a technique before choosing it to size stone for a particular project. Following is a brief description of several rock sizing techniques.

Isbash method

The Isbash formula (Isbash 1936) was developed for the construction of dams by depositing rocks into moving water. The Isbash curve should only be used for quick estimates or for comparisons. A coefficient is provided to target high- and low-turbulence flow conditions, so this method can be a high- or low-energy application. The equation is:

$$V_c = C \times \left(2 \times g \times \frac{\gamma_s - \gamma_w}{\gamma_w}\right)^{0.50} \times \left(D_{50}\right)^{0.50} \quad \text{(eq. TS14C–1)}$$

where:

V_c = critical velocity (ft/s)
C = 0.86 for high turbulence
C = 1.20 for low turbulence
g = 32.2 ft/s^2
γ_s = stone density (lb/ft^3)
γ_w = water density (lb/ft^3)
D_{50} = median stone diameter (ft)

A graphical solution is provided in figure TS14C–5 (ch. 16 of the Engineering Field Manual) This graph should be used only for quick estimates at a conceptual design level.

The U.S. Army Corps of Engineers (USACE) provides additional guidance for the use of the Isbash technique in EM 1110–2–1601. The required inputs are channel velocity, specific gravity of the stone, and a turbulence coefficient. The turbulence coefficient has two values that represent either high turbulence or low turbulence. The graphical solution for this is shown in figure TS14C–6(a) and (b).

Figure TS14C–5 Rock size based on Isbash curve

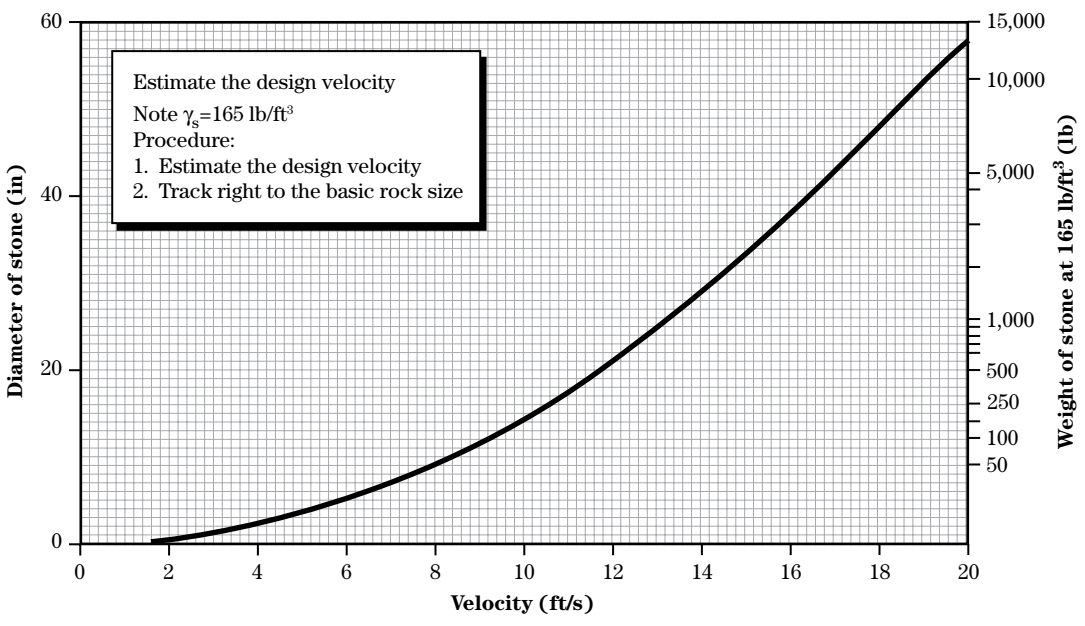
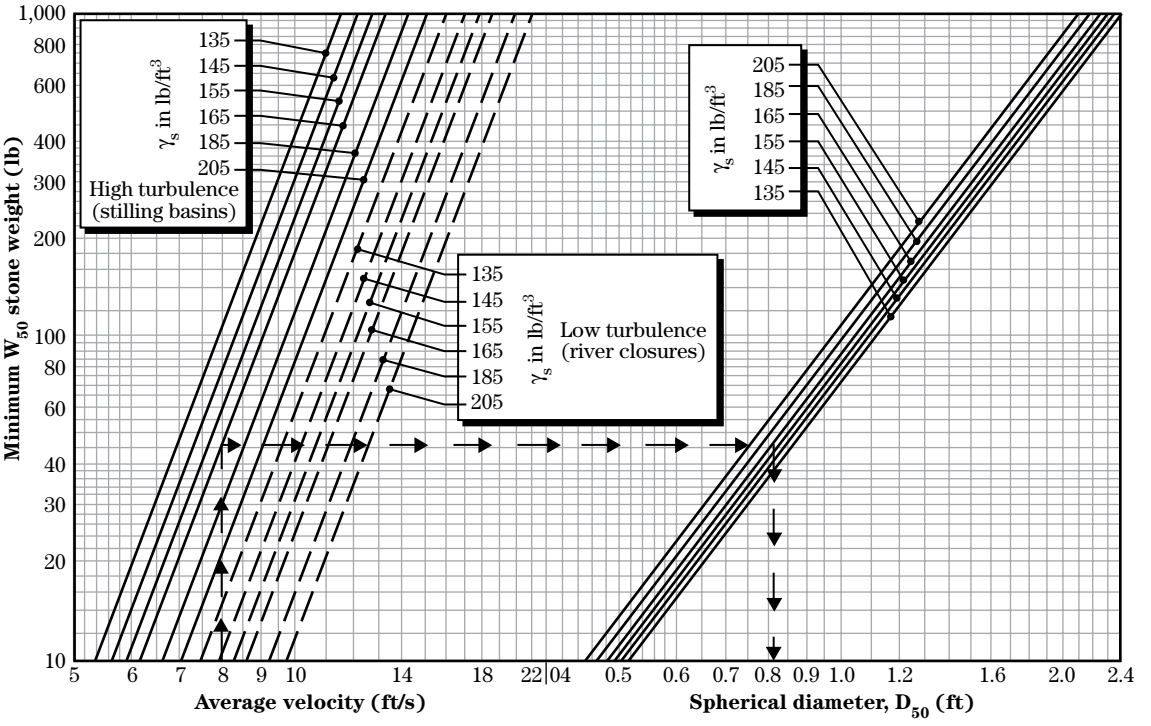


Figure TS14C–6 Graphical solution for Isbash technique

(a)



Basic equations :

$$V_c = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{\frac{1}{2}}\left(D_{50}\right)^{\frac{1}{2}}$$

$$D_{50} = \left(\frac{8W_{50}}{\pi\gamma_s}\right)^{\frac{1}{3}}$$

where:

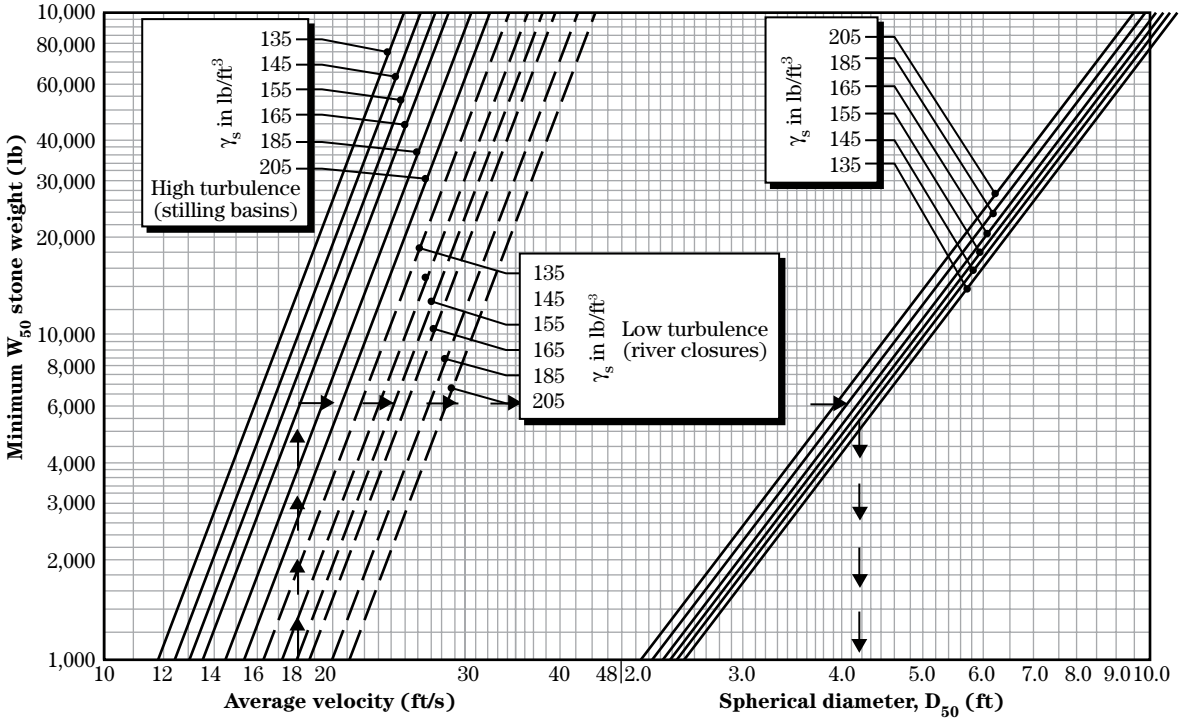
V	=	Velocity, ft/s
γ_s	=	Specific stone weight, lb/ft^3
γ_w	=	Specific weight of water, 62.5 lb/ft^3
W_{50}	=	Weight of stone, subscript denotes Percent of total weight of material containing stone of less weight
D_{50}	=	Spherical diameter of stone having the same weight as W_{50}
C	=	Isbash constant (0.86 for high turbulence level flow and 1.20 for low turbulence level flow)
g	=	Acceleration of gravity, ft/s^2

Stone stability velocity vs. stone diameter

Hydraulic design chart 712–1 (Sheet 1 of 2)

Figure TS14C–6 Graphical solution for Isbash technique—Continued

(b)



Basic equations :

$$V_c = C\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{\frac{1}{2}}\left(D_{50}\right)^{\frac{1}{2}}$$

$$D_{50} = \left(\frac{8W_{50}}{\pi\gamma_s}\right)^{\frac{1}{3}}$$

where:

V	=	Velocity, ft/s
γ_s	=	Specific stone weight, lb/ft^3
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W_{50}	=	Weight of stone, subscript denotes Percent of total weight of material containing stone of less weight
D_{50}	=	Spherical diameter of stone having the same weight as W_{50}
C	=	Isbash constant (0.86 for high turbulence level flow and 1.20 for low turbulence level flow)
g	=	Acceleration of gravity, ft/s^2

Stone stability velocity vs. stone diameter

Hydraulic design chart 712-1 (Sheet 2 of 2)

National Cooperative Highway Research Program Report 108

This method (Anderson, Paintal, and Davenport 1970) is suggested for design of roadside drainage channels handling less than 1,000 cubic foot per second and a maximum slope of 0.10 foot per foot. Therefore, this application can be used for high- or low-energy applications. Photo documentation shows that most of the research was done on rounded stones. This method will give more conservative results if angular rock is used.

$$\tau_o = \gamma R S_e \qquad \text{(eq. TS14C–2)}$$

$$\tau_c = 4D_{50} \qquad \text{(eq. TS14C–3)}$$

therefore,

$$D_{50} = \frac{\gamma R S_e}{4} \qquad \text{(eq. TS14C–4)}$$

τ_c = critical tractive stress
γ = 62.4 lb/ft^3
R = hydraulic radius (ft)
S_e = energy slope (ft/ft)
D_{50} = median stone diameter (ft)

A similar approach has been proposed by Newbury and Gaboury (1993) for sizing stones in grade control structures. This relationship is:

tractive force (kg/m^2) = incipient diameter (cm)

USACE—Maynord method

This low-energy technique for the design of riprap is used for channel bank protection (revetments). This method is outlined in USACE guidance as provided in EM 1110–2–1601, and is based on a modification to the Maynord equation:

$$D_{30} = FS \times C_S \times C_v \times C_T \times d \times \left[\left(\frac{\gamma_W}{\gamma_S - \gamma_w}\right)^{0.5} \times \frac{V}{\sqrt{K_1 \times g \times d}}\right]^{2.5} \qquad \text{(eq. TS14C–5)}$$

where:
D_m = stone size in ft; m percent finer by weight
d = water depth (ft)
FS = factor of safety (usually 1.1 to 1.5), suggest 1.2
C_s = stability coefficient Z=2 or flatter C=0.30, (0.3 for angular rock, 0.375 for rounded rock)
C_v = velocity distribution coefficient (1.0 for straight channels or inside of bends, calculate for outside of bends)
C_T = thickness coefficient (use 1.0 for 1 D_{100} or 1.5 D_{50}, whichever is greater))
γ_w = specific weight of water (lb/ft^3)
γ_s = specific weight of stone (lb/ft^3)
V = local velocity; if unknown use 1.5 $V_{average}$
g = 32.2 ft/s^2
K_1 = side slope correction as computed below

$$K_1 = \sqrt{1 - \frac{\sin^2\theta}{\sin^2\phi}} \qquad \text{(eq. TS14C–6)}$$

where:
θ = angle of rock from the horizontal
ϕ = angle of repose (typically 40°)

Note that the local velocity can be 120 to 150 percent of the average channel velocity or higher. The outside bend velocity coefficient and the side slope correction can be calculated:

$$C_V = 1.283 - 0.2\log\left(\frac{R}{W}\right) \qquad \text{(eq. TS14C–7)}$$

where:
R = centerline bend radius
W = water surface width

In the analysis used to develop this formula, failure was assumed to occur when the underlying material became exposed. It should be noted that while many of the other techniques specify a D_{50}, Maynord (1992) specifies a D_{30} which will typically be 15 percent smaller than the D_{50}. This assumes a specific gradation of:

$$1.8D_{15} < D_{85} < 4.6D_{15} \qquad \text{(eq. TS14C–8)}$$

The USACE developed this method for the design of riprap used in either constructed or natural channels which have a slope of 2 percent or less and Froude numbers less than 1.2. As a result, this technique is not appropriate for high-turbulence areas.

Maynord's side-slope and invert equation is for cases where the protective blanket is constructed with a relatively smooth surface and has no significant projections. It is appropriate for use to size stone-toe protection. However, it has been suggested that with some adjustment to the coefficients (typically using a velocity coefficient of 1.25 and a local velocity equal to 160% of the channel velocity), Maynord's method can

be used for exposed boulders or stones exposed to impinging flow.

U.S. Bureau of Reclamation method

This high-energy technique is outlined in U.S. Bureau of Reclamation (USBR) EM–25 (Peterka 1958) and was developed for sizing riprap below a stilling basin. It was empirically developed using 11 prototype installations with velocities ranging from 1 foot per second to 20 foot per second. The formula is:

$$D_{50} = 0.0122V^{2.06} \qquad \text{(eq. TS14C–9)}$$

where:
D_{50} = median stone diameter (ft)
V = average channel velocity (ft/s)

U.S. Geological Survey method (Blodgett 1981)

This technique is based on analysis of field data of 39 large events from sites in Arizona, Washington, Oregon, Nevada, and California. Riprap protection failed in 14 of the 39 cases. An envelope curve was empirically developed to represent the difference between sites that performed without damage and those that were damaged by particle erosion. The formula is:

$$D_{50} = 0.01V^{2.44} \qquad \text{(eq. TS14C–10)}$$

where:
D_{50} = median stone diameter (ft)
V = average channel velocity (ft/s)

This method typically provides overly conservative results.

Tillatoba model study

This study (Blaisdell 1973) provides an equation for sizing stone to remain stable in the turbulent flow found below stilling basins. This high-energy technique results in an estimate for D_{50}.

$$D_{50} = 0.00116\frac{V^3}{\sqrt{d}} \qquad \text{(eq. TS14C–11)}$$

where:
V = velocity (ft/s)
d = flow depth (ft)
D_{50} = stone diameter (ft)

USACE steep slope riprap design

This high-energy technique is outlined in standard USACE guidance as provided in EM 1110–2–1601. It is designed for use on slopes from 2 to 20 percent. However, the side slopes should be 1V:2.5H or flatter. A typical application would be a rock-lined chute. The formula is:

$$D_{30} = \frac{1.95S^{0.555}(Cq)^{\frac{2}{3}}}{g^{\frac{1}{3}}} \qquad \text{(eq. TS14C–12)}$$

where:
D_{30} = stone size; m percent finer by weight
S = channel slope
q = unit discharge ($q = Q/b$, where b = bottom width of chute and Q is total flow)
C = flow concentration factor (usually 1.25, but can be higher if the approach is skewed)
g = gravitational constant

This equation is applicable to thickness = 1.5 D_{100}, angular rock, unit weight of 167 pounds per cubic foot, D_{85}/D_{15} from 1.7 to 2.7, slopes from 2 to 20 percent, and uniform flow on a downslope with no tailwater. This equation typically predicts conservative sizes.

USACE habitat boulder design

This technique is outlined in USACE guidance provided in EMRRP–SR–11. It is developed for sizing boulder clusters in a channel for habitat enhancement. This high-energy relationship is an incipient motion relation for fully immersed boulders in turbulent flow on a flat bed. This method is for impinging flow. The formula is:

$$D = \frac{18(\text{depth})S_f}{(SG-1)} \qquad \text{(eq. TS14C–13)}$$

where:
D = minimum stone size
depth = channel depth
S_f = channel friction slope
SG = specific gravity of the stone

This equation has also been used to size stones for use in low instream weirs. However, estimating the friction slope across a drop can be difficult.

Abt and Johnson (1991)

Abt and Johnson (1991) conducted near-prototype flume studies to determine riprap stability when subjected to overtopping flows such as in spillway flow or in sloping loose-rock grade control structures. Slopes varied from 2 to 20 percent. Riprap design criteria for overtopping flows were developed for two conditions: stone movement and riprap layer failure. Criteria were

developed as a function of median stone size, unit discharge, and embankment slope. The equation is:

$$D_{50} = \left(q_{design}\right)^{0.56} \times S^{0.43} \times 5.23 \qquad \text{(eq. TS14C–14)}$$

where:
D_{50} = stone size in inches; m percent finer by weight
q_{design} = unit discharge ($ft^3/s/ft$)
S = channel slope (ft/ft) and S between 0.02 and 0.20 ft/ft

$$\left(q_{design}\right) = \frac{(q_{failure})}{0.74} = 1.35 q_{failure} \qquad \text{(eq. TS14C–15)}$$

Stone movement occurred at approximately 74 percent of the flow, causing layer failure. It was determined from testing that rounded stone should be oversized by approximately 40 percent to provide the same protection as angular stone.

ARS rock chutes

This design technique (Robinson, Rice, and Kadavy 1998) is primarily targeted at high-energy applications. Loose riprap with a 2 D_{50} blanket thickness composed of relatively uniform, angular riprap was tested to overtopping failure in models and field scale structures. This method applies to bed slopes of 40 percent and less. This technique can be used for low slope, and thus, low-energy applications, but it is particularly useful for slopes greater than 2 percent. A factor of safety appropriate for the project should be applied to the predicted rock size. The equations are:

for S <0.1

$$D_{50} = 12\left(1.923 q S^{1.5}\right)^{0.529} \qquad \text{(eq. TS14C–16)}$$

0.10<S<0.40

$$D_{50} = 12\left(0.233 q S^{0.58}\right)^{0.529} \qquad \text{(eq. TS14C–17)}$$

where:
D_{50} = median stone size (in)
q = highest stable unit discharge ($ft^3/s/ft$)
S = channel slope (ft/ft)

A spreadsheet program (Lorenz, Lobrecht, and Robinson 2000) is available to assist in sizing riprap on steep slopes. A screen capture of this spreadsheet program is shown in figure TS14C–7.

This method is best used in steep slopes for grade control, embankment overtopping, or on side inlets from fields to a major drainage outlet. The spreadsheet provides much additional information related to rock chutes such as guidance on inlet and outlet conditions, quantity estimates, and hydrology.

California Department of Transportation RSP

This technique was developed by the California Department of Transportation (CALTRANS) for designing rock slope protection (RSP) for streams and riverbanks. Unlike most of the other available techniques, it results in a recommended minimum weight of the stone. The equation is:

$$W = \frac{0.00002}{\left(G_S - 1\right)^3} \times \frac{VM \times V^6 \times G_S}{\sin^3\left(r - a\right)} \qquad \text{(eq. TS14C–18)}$$

where:
W = minimum rock weight (lb)
V = velocity (ft/s)
VM = 0.67 if parallel flow
VM = 1.33 if impinging flow
G_S = specific gravity of rock (typically 2.65)
r = angle of repose (70° for randomly placed rock)
a = outside slope face angle to the horizontal (typically a maximum of 33°)

The weight indicated by this method should be used in conjunction with standard CALTRANS specifications and gradations.

Far West states (FWS)—Lane's Method

Vito A. Vanoni worked with the Northwest E&WP Unit to develop the procedure from the ASCE paper entitled "Design of Stable Alluvial Channels" (Lane 1955a). The equation is:

$$D_{75} = \frac{3.5}{C \times K} \times \gamma_w \times D \times S_f \qquad \text{(eq. TS14C–19)}$$

where:
D_{75} = stone size, (in)
C = correction for channel curvature
K = correction for side slope
S_f = channel friction slope (ft/ft)
d = depth of flow (ft)
γ_w = density of water

This is generally considered to be a conservative technique. It assumed that the stress on the sides of the channel were 1.4 times that of the bottom. This

Figure TS14C–7 Rock chute spreadsheet

Rock Chute Design Data

(Version 4.01 - 04/23/03, Based on Design of Rock Chutes by Robinson, Rice, Kadavy, ASAE, 1998)

Project: *Spillway protection* **County:** *Woodbury*
Designer: *Jim Villa* **Checked by:** _____
Date: 3/30/2006 **Date:** _____

Input Channel Geometry

→ Inlet Channel	→ Chute	→ Outlet Channel
Bw = *20.0* ft.	Bw = *20.0* ft.	Bw = *40.0* ft.
Side slopes = *4.0* (m:1)	Factor of safety = *1.20* (F_s)	Side slopes = *4.0* (m:1)
n-value = *0.035*	Side slopes = *4.0* (m:1) → *2.0:1 max.*	n-value = *0.045*
Bed slope = *0.0060* ft./ft.	Bed slope (5:1) = *0.200* ft./ft. → *2.5:1 max.*	Bed slope = *0.0050* ft./ft.
Freeboard = *0.5* ft.	Outlet apron depth, d = *1.0* ft.	Base flow = *0.0* cfs

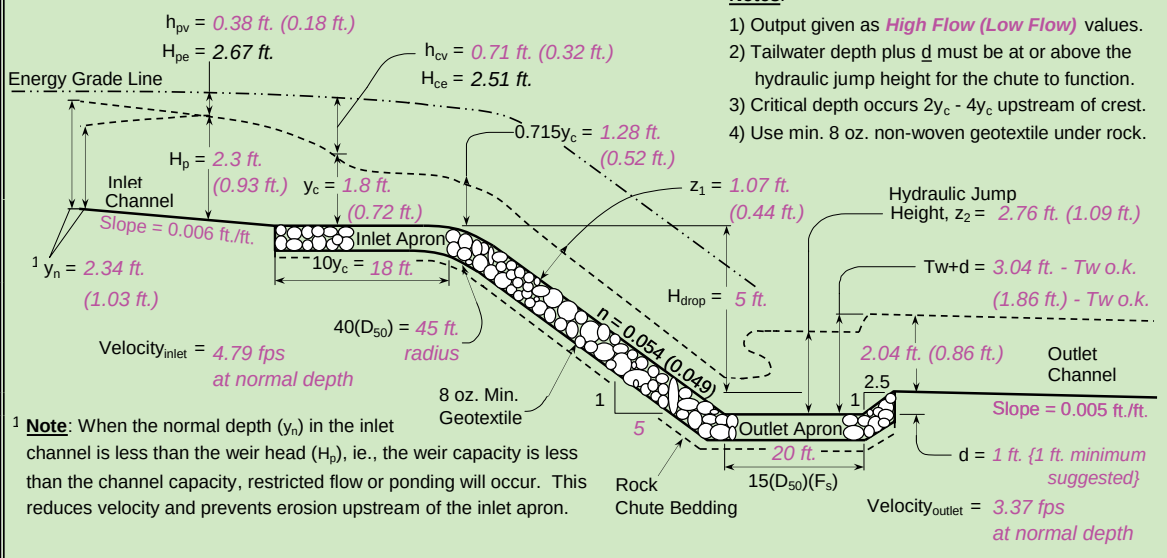
Design Storm Data (Table 2, NHCP, NRCS Grade Stabilization Structure No. 410)

Drainage area = *450.0* acres Rainfall = ○ 0 - 3 in. ◉ 3 - 5 in. ○ 5+ in.
Apron elev. --- Inlet = *105.0* ft. --- Outlet = *99.0* ft. --- (H_{drop} = 5 ft.)
Chute capacity = Q5-year
Total capacity = Q10-year
Minimum capacity (based on a 5-year, 24-hour storm with a 3 - 5 inch rainfall)
Q_{high} = *330.0* cfs High flow storm through chute → Tw (ft.) = *Program* 0.20
Q_{low} = *75.0* cfs Low flow storm through chute → Tw (ft.) = *Program*

***Note**: The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.*
***Input tailwater (Tw)**:*

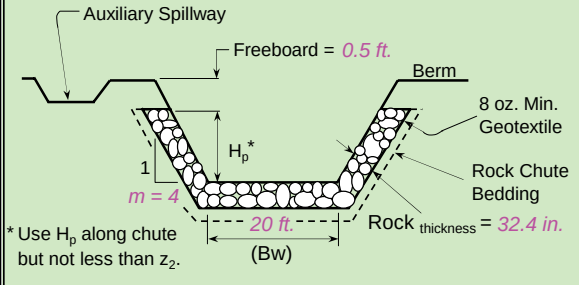
Profile and Cross Section (Output)

Notes:
1) Output given as ***High Flow (Low Flow)*** values.
2) Tailwater depth plus d must be at or above the hydraulic jump height for the chute to function.
3) Critical depth occurs $2y_c$ - $4y_c$ upstream of crest.
4) Use min. 8 oz. non-woven geotextile under rock.



[1] **Note**: When the normal depth (y_n) in the inlet channel is less than the weir head (H_p), ie., the weir capacity is less than the channel capacity, restricted flow or ponding will occur. This reduces velocity and prevents erosion upstream of the inlet apron.

Profile Along Centerline of Chute



* Use H_p along chute but not less than z_2.

Typical Cross Section

q_t =	*13.65 cfs/ft.*	Equivalent unit discharge
F_S =	*1.20*	Factor of safety (multiplier)
z_1 =	*1.07 ft.*	Normal depth in chute
n-value =	*0.054*	Manning's roughness coefficient
$D_{50}(F_s)$ =	*16.2 in. (309 lbs. - 50% round / 50% angular)*	
$2(D_{50})(F_s)$ =	*32.4 in.*	Rock chute thickness
Tw + d =	*3.04 ft.*	Tailwater above outlet apron
z_2 =	*2.76 ft.*	Hydraulic jump height
***** The outlet**	***will***	***function adequately***

High Flow Storm Information

is about 1.8 times the actual stress on the sides of a straight channel. It is very close to the stresses on the sides in a curved channel reach. The curved corrections included in the procedure only make the conservative answer even more conservative. In addition, it was developed for stones with a specific gravity of 2.56. However, it has been successfully applied on many projects. This procedure may be used with figure TS14C–8 and is:

Step 1 Enter figure TS14C–8 with energy slope (channel grade) and flow depth.

Step 2 Track right to side slope.

Step 3 Track up to ratio of curve radius to water surface width.

Step 4 Track right to estimate required riprap size.

Figure TS14C–8 Lane's method

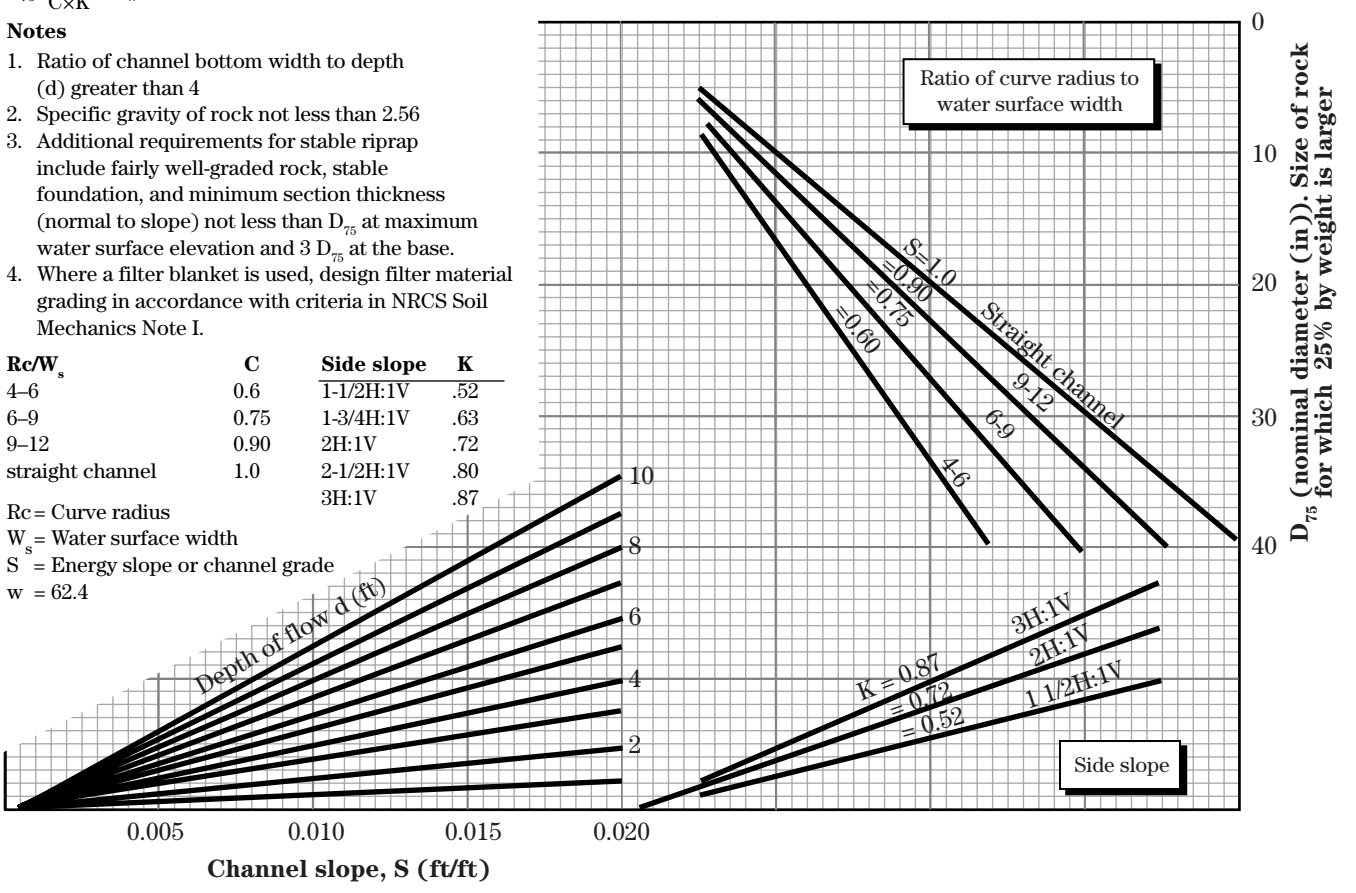
$$D_{75} = \frac{3.5}{C \times K} \times \gamma_w \times d \times S$$

Notes

1. Ratio of channel bottom width to depth (d) greater than 4
2. Specific gravity of rock not less than 2.56
3. Additional requirements for stable riprap include fairly well-graded rock, stable foundation, and minimum section thickness (normal to slope) not less than D_{75} at maximum water surface elevation and 3 D_{75} at the base.
4. Where a filter blanket is used, design filter material grading in accordance with criteria in NRCS Soil Mechanics Note I.

Rc/W_s	C	Side slope	K
4–6	0.6	1-1/2H:1V	.52
6–9	0.75	1-3/4H:1V	.63
9–12	0.90	2H:1V	.72
straight channel	1.0	2-1/2H:1V	.80
		3H:1V	.87

Rc = Curve radius
W_s = Water surface width
S = Energy slope or channel grade
w = 62.4



U.S. Department of Transportation Federal Highway Administration techniques

Several additional computational techniques for designing riprap are available from the U.S. Department of Transportation Federal Highway Administration (FHWA). While these are not described in detail, a brief description of each is provided in table TS14C–2.

Review the references (FHWA HEC 1987, 1988, 2001a, 2001b) to obtain the design relationships and application manuals for these methods.

Summary guide of selected techniques

Attributes of selected methods are summarized in table TS14C–3 to allow the user to quickly select a method.

The designer should not be surprised if the different techniques produce different answers. The user needs to recognize the limits and applicability of each technique and match it to the site and project conditions.

Table TS14C–2 Federal Highway Administration techniques

HEC–11	This technique was developed for use on natural streams or rivers with a flow greater than 50 ft^3/s. It is limited to straight or mildly curving reaches with relatively uniform cross sections. This method calculates a D_{50} based on average channel velocity, side slope, riprap angle of repose, specific gravity of the stone, and average channel depth
HEC–15	This technique was developed for use on small, constructed channels with a flow less than 50 ft^3/s
HEC–18	This technique was developed for design of stone at bridge piers and abutments

Table TS14C–3 Summary of techniques

Technique	High or low energy	Slopes	Typical application(s)
Isbash	Both	Not specified	Rock revetment, stilling basins, river closures
108 Report	Both	<10%	Quick assessments for stable stone requirements
Maynord	Low	<2%	Rock revetment, bank protection, stone toe
Abt and Johnson	High	2% to 20%	Overtopping, grade protection
ARS – rock chute	High	2% to 40%	Overtopping, rock chutes, grade protection
USBR	High	Not specified	Riprap below a stilling basin
USGS Blodgett	Both	Not specified	Riprap stability
USACE Steep Slope Riprap	High	2% to 20%	Rock chutes, grade protection
USACE Habitat Boulder	High	Not specified	Instream boulders for habitat enhancement
CALTRANS RSP	Low	<2%	Rock revetment, bank protection, stone toe
Lane's (FWS)	Low	<2%	Stone bank protection, stream barbs with adjustments

Factor of safety

Stone sizing should be approached with care because rock treatments can be expensive and can give a false sense of security if not applied appropriately. A factor of safety is often advisable to account for unknowns and uncertainty. In some cases, the factor of safety is part of the sizing formulas provided. Where a factor of safety is not built into the procedure, the designer should multiply the resulting size by an appropriate value. Appropriate engineering judgment should be applied when assigning a factor of safety. Maynord (1992) suggests a minimum factor of safety of 1.1. Typically, a factor of safety will range from 1.1 to 1.5. The risk and uncertainty associated with a project should be reflected in the factor of safety.

Example calculations

Example calculations are presented for selected methods to illustrate the variability associated with rock sizing methods. The examples may also provide a new user with confirmation that they are correctly applying a method.

Example problem: Mild slope

Problem: For the following flow conditions, determine the required rock size for stone toe protection.

G_s = 2.65 or γ_s=165.36 lb/ft^3
Width = 40 ft
n = 0.045
Slope = 0.01 ft/ft
Depth = 6 ft

Solution: Solve relevant hydraulic parameters

Vel = 9.1 ft/s
Q = 2,200 ft^3/s
Y_{crit} = 4.54 ft

The riprap size determined from several methods is:

Isbash	D_{50} = 6.5 in
Maynord	D_{30} = 4.6 in, D_{50}= 5.5 in
Lane's (FWS)	D_{75} = 15 in, D_{50} = 12.7 in
Abt and Johnson	D_{50} = 8.1 in
ARS rock chute	D_{50} = 3.6 in

Discussion: The computed critical depth indicates that this is a subcritical flow. The design calls for a revetment-type protection, so the stones are not projecting into the flow. Therefore, this is a low-energy flow condition. The Isbash (1936) and the Maynord (1992) methods both indicate a D_{50} of about 5.5 to 6.5 inches. These methods were developed for conditions that are similar to those in the problem statement. Therefore, a stone size of 6 inches with an appropriate factor of safety should be acceptable.

Lane's (1955a) FWS method provides a conservative estimate of 12.7 inches. While this technique is used in similar situations, a conservative answer is expected. The Abt and Johnson (1991) method and the ARS method (Robinson, Rice, and Kadavy 1998) were developed for steeper high-energy flow conditions (>2%); therefore, use of these methods would not be advisable for this application.

Example problem: Steep slope

Problem: For the following flow conditions, determine the required rock size for a rock chute.

G_s = 2.65 or γ_s=165.36 lb/ft^3
Width = 40 ft
n = 0.045
Slope = 0.06 ft/ft
Depth = 3.5 ft

Solution: Solve relevant hydraulic parameters

Vel = 16.7 ft/s
Q = 2,340 ft^3/s
Y_{crit} = 4.7 ft

The riprap size determined from several methods is:

Isbash	D_{50} = 1.6 ft
Maynord	D_{30} = 1.6 ft, D_{50} = 1.9 ft
Lane's (FWS)	D_{75} = 3.7 ft, D_{50} = 3.2 ft
Abt and Johnson	D_{50} = 1.3 ft
ARS rock chute	D_{50} = 1.1 ft

Discussion: The computed critical depth indicates that this is a supercritical flow. While similar in prediction, the Isbash and the Maynord (1992) methods were not developed for conditions that are described in the problem statement. The Abt and Johnson (1991), as well as the ARS rock chute methods (Robinson, Rice, and Kadavy 1998), were derived for similar conditions to the problem statement. Therefore, the 1.1 to 1.3 foot D_{50} riprap with an appropriate factor of safety should be acceptable.

Conclusion

Rock is often used where long-term durability is needed, velocities are high, periods of inundation are long, and there is a significant threat to life and property. Whether a streambank project involves the use of rock as part of a stand-alone treatment or as a component of an integrated system, the determination of the required stone size requires engineering analysis. Stone sizing should be approached with care because rock treatments can be expensive and can give a false sense of security if not applied appropriately. Since stone sizing methods are normally developed for a specific application, care should be exercised matching the selected method with the project purpose and site condition. Therefore, the intended application should dictate which rock sizing technique is used. By using several methods, the designer will often see a convergence of rock sizes for a given application.

Technical Supplement 14D

Geosynthetics in Stream Restoration

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Inert or manmade materials can be used in restoration designs where immediate stability is required and can be used in concert with vegetation.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14D

Geosynthetics in Stream Restoration

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Technical Supplement 14D

Geosynthetics in Stream Restoration

Purpose

A variety of geosynthetic materials may be used for various functions and applications in stream restoration and stabilization projects. This technical supplement is intended to provide field staffs an understanding of some of the basic principles and applications of geosynthetic materials.

Introduction

A geosyntethic material is defined as a planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as part of a manmade project, structure, or system (American Society for Testing and Materials International (ASTM) D4439). Geosynthetics used in stream restoration and stabilization include geotextiles, geogrids, geonets, geocells, and rolled erosion control products.

Materials

Selection of a geosynthetic material appropriate for a project requires an understanding of the different types that are available, as well as their performance criteria and range of applications. Five types of geosynthetic materials are described here.

Geotextile

A geotextile is defined as a permeable geosynthetic comprised solely of textiles (ASTM D4439). A geotextile may be woven or nonwoven and may be composed of monofilament yarns or monofilament plastic (fig. TS14D–1). A nonwoven geotextile may be needle-punched (fig. TS14D–2), heat bonded (fig. TS14D–3), or resin bonded.

Geogrid

A geogrid is defined as a geosynthetic formed by a regular network of integrally connected elements with apertures greater than a fourth inch to allow interlocking with surrounding soil, rock, earth, and other sur-

Figure TS14D–1 Monofilament woven geotextile

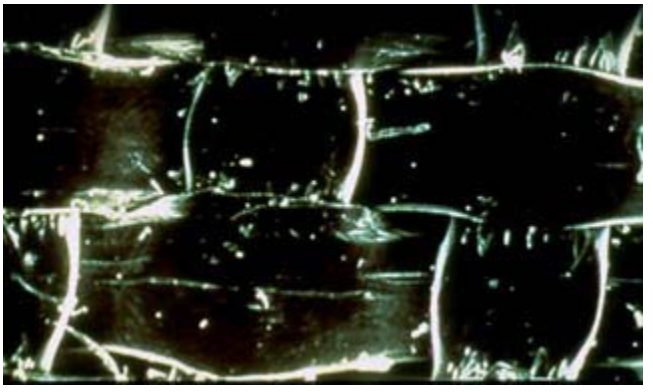


Figure TS14D–2 Needle-punched nonwoven geotextile

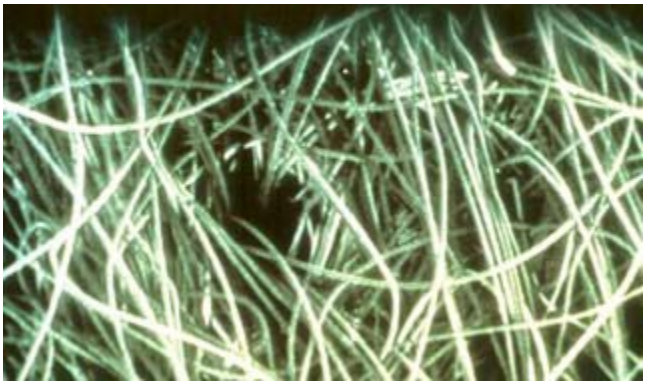
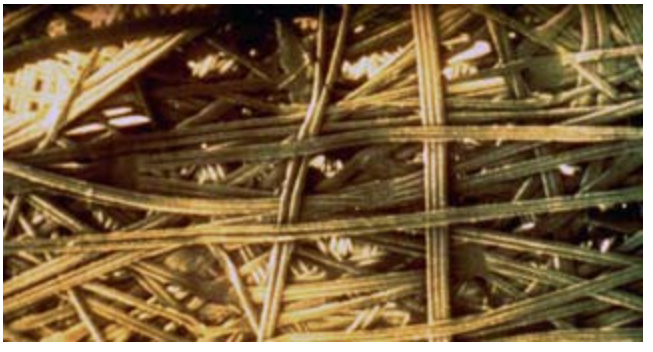


Figure TS14D–3 Heat-bonded nonwoven geotextile



rounding materials to function primarily as reinforcement (ASTM D4439). A geogrid may be biaxial (fig. TS14D–4), or uniaxial (fig. TS14D–5).

Geonet

A geonet is defined as a geosynthetic consisting of integrally connected parallel sets of ribs overlying similar sets at various angles for planar drainage of liquids and gases (ASTM D4439). A typical geonet is shown in figure TS14D–6.

Geocell

A geocell is defined as a product composed of polyethylene strips, connected by a series of offset, full-depth welds to form a three-dimensional honeycomb system (ASTM D4439). Geocells (fig. TS14D–7) are available in a variety of depths from 4 inches to 9 inches.

Figure TS14D–4 Biaxial geogrid

Figure TS14D–5 Uniaxial geogrid

Figure TS14D–6 Geonet

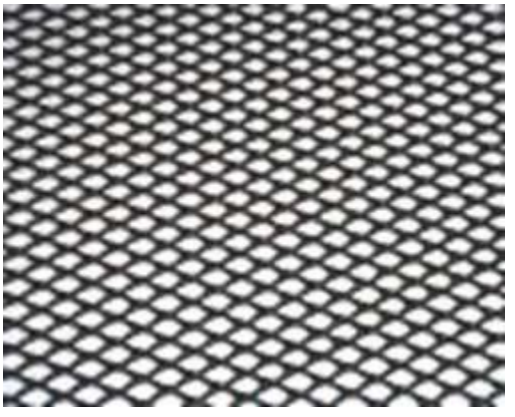


Figure TS14D–7 Geocell

Rolled erosion control products

Rolled erosion control products consist of both erosion control blankets (ECB) used for temporary erosion protection and turf reinforcement mats (TRM) for more permanent erosion protection. An ECB is shown in figure TS14D–8 and two TRMs are shown in figure TS14D–9.

Functions

Geosynthetics may provide one or more of the following functions on a stream restoration or stabilization project.

Drainage

Geosynthetics used for drainage are intended to act as a conduit for fluid (typically water) within the plane of the fabric. Nonwoven geotextiles, geonets, or a composite of geotextiles and geonets are often used for this function.

Figure TS14D–8 Erosion control blanket

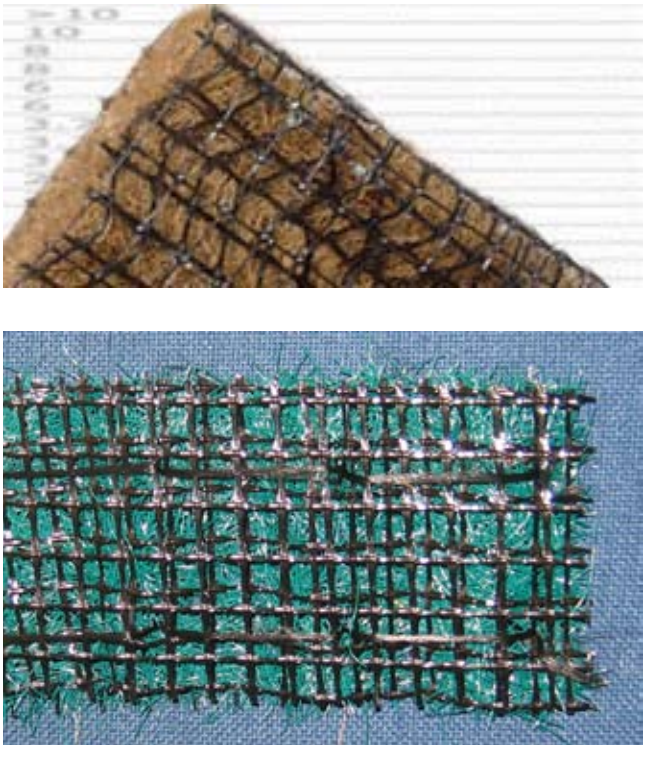
Filtration

Filtration is the most common use of geosynthetics in stream restoration and stabilization projects. A geosynthetic used for filtration is intended to retain the particles of the filtered (protected) soil, while allowing a fluid to flow through the plane of the fabric. Woven and nonwoven needle-punched geotextiles are used for this function.

Separation

The objective of geosynthetics used for the separation function is to prevent two different materials from mixing and compromising the performance of one or both of these materials. Woven and nonwoven geotextiles may both be used for this function. Heat-bonded nonwoven geotextiles offer an economical geosynthetic separator.

Figure TS14D–9 Turf reinforcement mats



Reinforcement

Geosynthetics used as reinforcement strengthen the soil mass by interaction with soil, creating frictional or adhesion forces. The geosynthetic reinforcement provides resistance to tensile forces which cannot be otherwise carried by an unreinforced soil mass. High-strength woven geotextiles and geogrids are used for this function.

Erosion control

In erosion control, geosynthetics protect the soil surface from the tractive forces of moving water. They may also provide additional strength to the root system of vegetation. Geotextiles, ECBs, and TRMs may be used for this function.

Applications

Geosynthetics may be used in a variety of applications for streambank restoration and stabilization.

- separation and/or filtration beneath erosion protection materials (fig. TS14D–10)
- reinforcement of steep streambank slopes (fig. TS14D–11)
- mechanically stabilized earth walls (fig. TS14D–12)
- earth retaining structures (fig. TS14D–13)
- erosion protection (fig. TS14D–14)

Geotextile filter

Nonwoven needle-punched geotextiles are typically less costly than woven geotextiles. Nonwoven geotextiles have typically been used by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) beneath erosion protection materials to serve either filtration or separation functions. The U.S. Army Corps of Engineers (USACE) has traditionally used woven geotextiles for these functions. A heat-bonded or resin-bonded nonwoven geotextile should not be used for geotextiles serving as a filter. The permeability of heat-bonded and resin-bonded geotextiles is too low to allow adequate seepage and dissipation of hydrostatic pressure.

A woven geotextile is recommended when water will frequently flow through the geotextile and the retained soil particles have the potential to move within the soil structure towards the geotextile. In this condition, a nonwoven geotextile has a greater potential for clogging since it will allow very few particles to filter through the geotextile. If soil particles have the potential to move within the soil structure, a woven geotextile will often allow fine sand and silt particles to pass through the geotextile until a natural graded filter is developed within the soil structure behind the geotextile. To retain fine sand and silt soil particles, a granular filter of ASTM C33 sand overlain by a properly sized geotextile is often used.

Recommended geotextile properties for geotextiles providing filtration are provided in the Guide for the Use of Geotextile (USDA Soil Conservation Service (SCS) 1991). Recommended geotextile properties for geotextiles providing drainage, separation, and filtration beneath erosion protection are provided in Geotextile Specification for Highway Applications (American Association of State Highway Transportation Officials (AASHTO) 2000). An example design of a geotextile providing filtration beneath rock riprap is provided later in this technical supplement.

It is important to note that some soil bioengineering techniques do not function well under geotextiles, and placing holes through the geotextile may provide a seepage path that would weaken the structure. This may require a trade-off analysis to balance the advantages of incorporating soil bioengineering techniques against the advantages of an intact filter geotextile. Finally, it should be noted that some streambanks may have sufficient gravel or clay content (PI>15), precluding the need for either bedding or geotextiles.

Reinforced slopes

The reinforced slope obtains its internal stability from the tensile strength of the geosynthetic reinforcement layers. The reinforced slope design may be completed with guidance provided by USACE (1995b), U.S. Department of Transportation (DOT) Federal Highway Administration (FHWA) 2001c), or Designing with Geosynthetics

Figure TS14D–10 Separation and/or filtration beneath erosion protection material

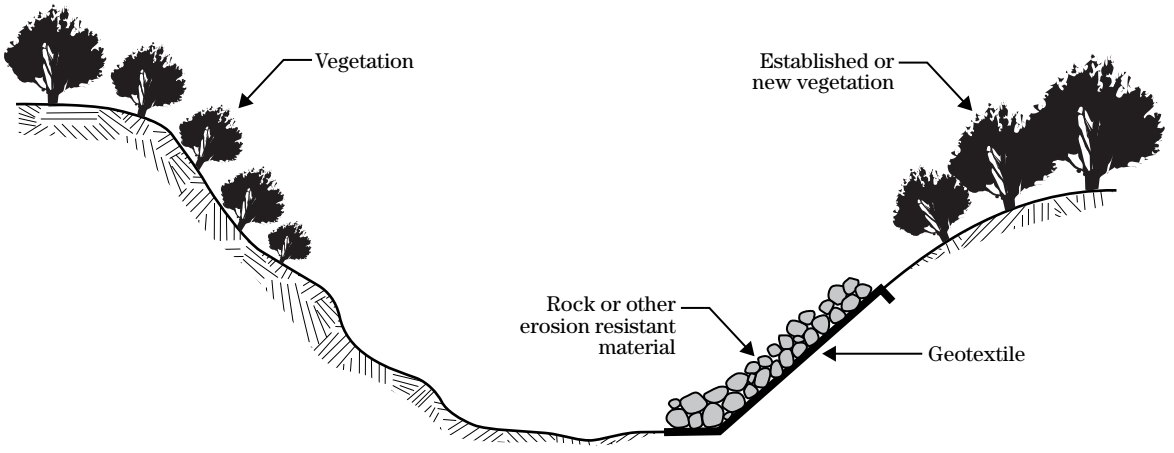


Figure TS14D–11 Reinforcement of steep streambank slope

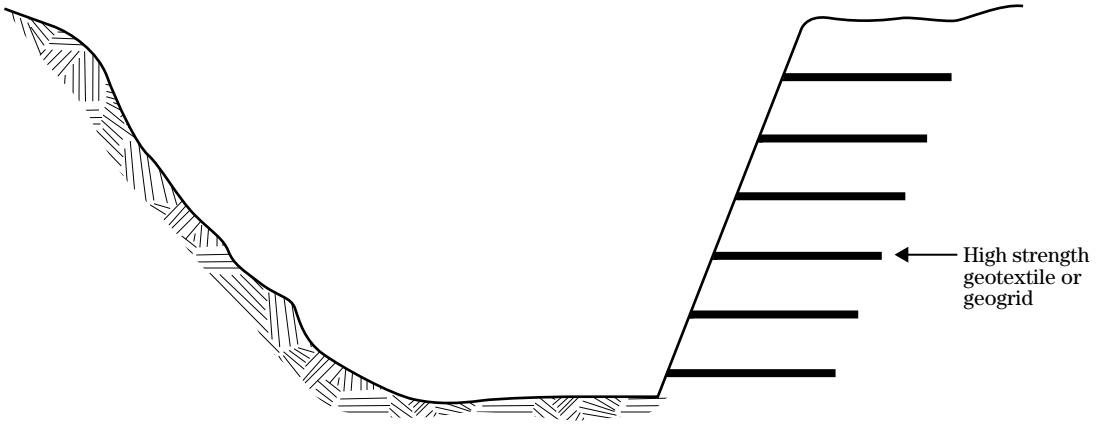


Figure TS14D–12 Mechanically stabilized earth wall

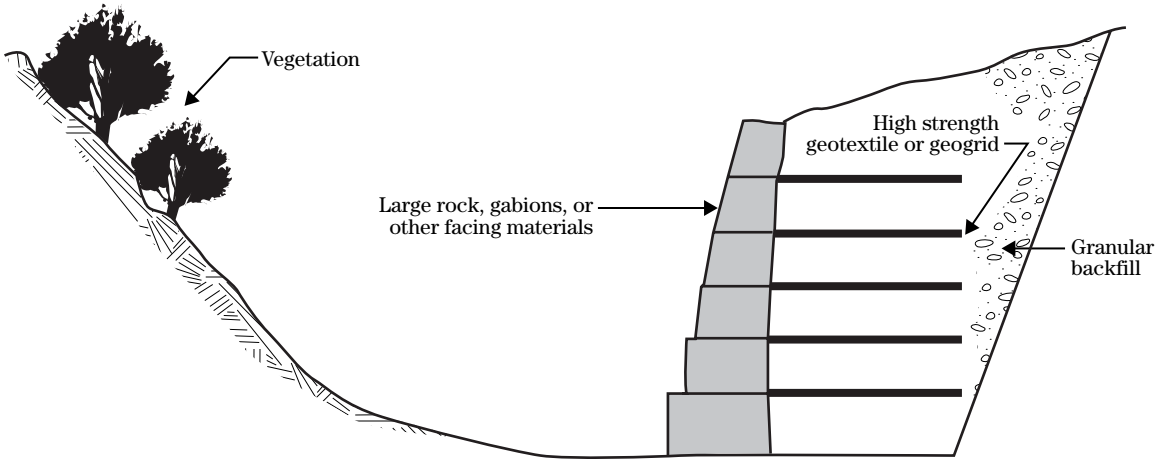


Figure TS14D–13 Earth retaining structure

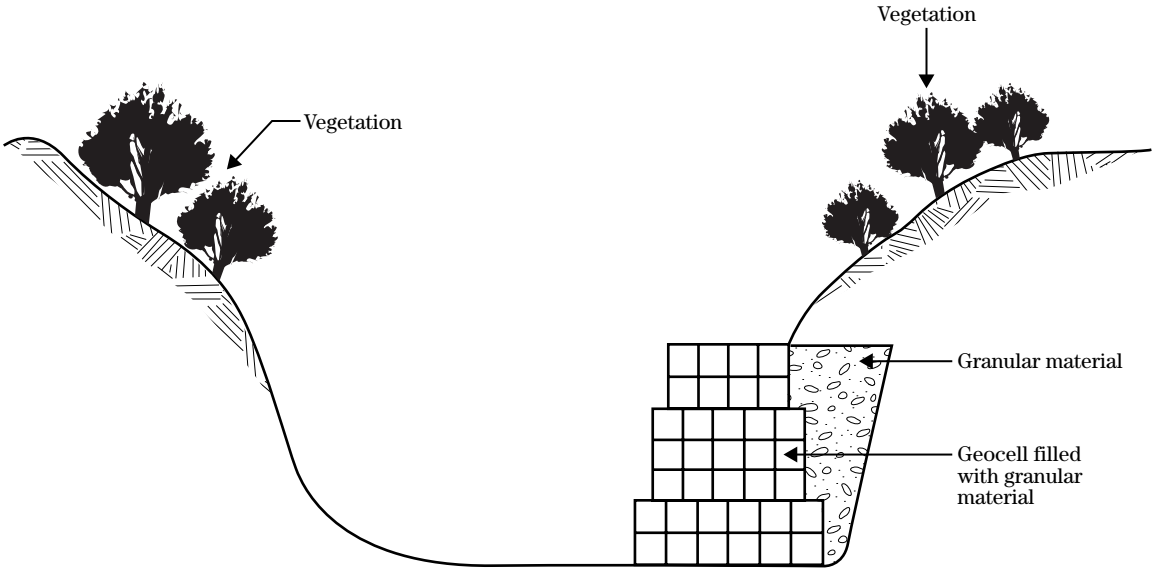
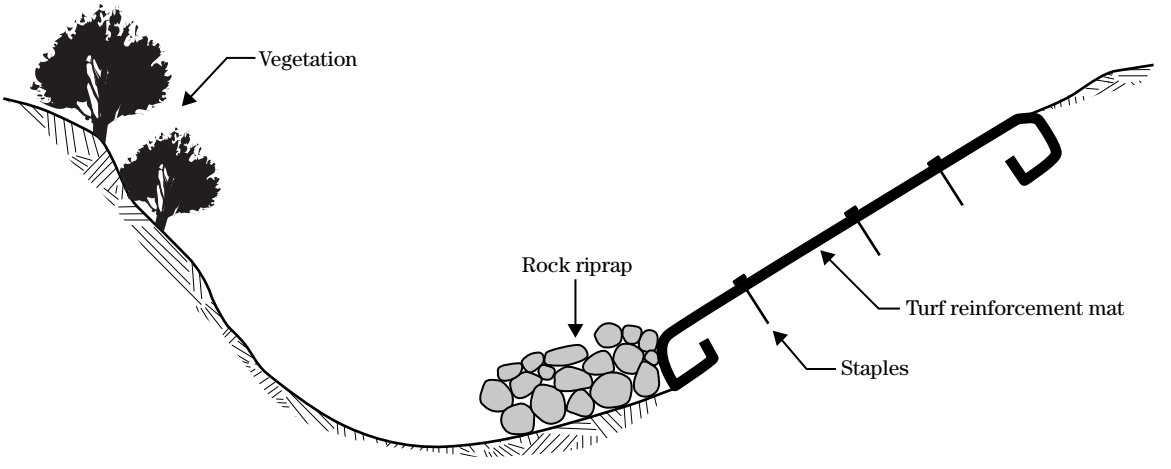


Figure TS14D–14 Erosion protection



(Koerner 1998). The computer program ReSlope may also aid in the design.

Once the internal stability of the slope is satisfied, the external stability must be evaluated, including an analysis of sliding, overturning, bearing capacity, and settlement.

The global stability of reinforced slopes must be analyzed with the appropriate slope stability analysis method. Slope stability analyses are typically performed with computer software such as SLOPE/W, PCSTABLE, or UTEXAS software.

A photograph of a reinforced soil slope with a rock facing is shown in figure TS14D–15.

Mechanically stabilized earth walls

A mechanically stabilized earth (MSE) wall must be designed for external, internal, and local stability. The external stability analyses include sliding, overturning, bearing capacity, and settlement. The internal stability analyses include geosynthetic pullout, tensile strength of the geosynthetic, and internal sliding. Local stability analyses include an analysis of the facing connection to the geosynthetic and bulging of the facing. A photograph of an MSE wall that is under construction is shown in figure TS14D–16.

Guidance for MSE wall design is provided by FHWA (FHWA 2001c), Designing with Geosynthetics (Koerner 1998), or National Concrete Masonry Association (NCMA) 1997). A computer program entitled MSEW may also aid in the design.

Figure TS14D–15 Reinforced soil slope with rock face (*Photograph courtesy of Frank Cousin, NRCS MI*)

Figure TS14D–16 MSE wall under construction

The global stability of the MSE wall, retained soil, and soil foundation must be analyzed, just as reinforced slope design.

Earth retaining structures

An earth retaining structure must be designed for external stability and internal stability. The external stability analyses include an analysis of sliding, overturning, bearing capacity, and settlement. In a geocell wall, the internal stability analysis includes an analysis of the friction between each geocell layer.

The global stability of earth retaining structures must be analyzed just as reinforced slopes. A photograph of a geocell earth retaining structure is shown in figure TS14D–17.

Erosion protection

Selection and installation of an ECB or TRM is a function of the hydraulic characteristics of the site, streambank slopes, and expected lift of the product. ECBs are used for temporary erosion protection until adequate vegetation can be established. TRMs are considered permanent erosion protection and are designed to reinforce the soil surface and root system of the vegetation.

Figure TS14D–17 Geocell earth retaining structure (*Photograph courtesy of Carl Gustafson, NRCS MA*)

Example: Geotextile filter calculations

***Problem*:** A streambank stabilization project will include a rock chute constructed on soil with the gradation in table TS14D–1.

Using design criteria for a ***woven geotextile*** in Design Note 24, Guide for the Use of Geotextiles (USDA SCS 1991) (table TS14D–2), determine the geotextile filter requirements.

Solution: Soil contains 15 to 50 percent finer than the # 200 sieve, so:

Apparent opening size (AOS) $<D_{85}$
Percent open area (POA) >4%
Permeability, $K_{geotextile} > 10\ K_{soil}$

D_{85}= 0.150 mm, so AOS ≤0.15 mm (#100 sieve)

Percent open area (POA) >4%
The soil contains 25 percent finer than the #200 sieve with an estimated K_{soil} = 0.004 cm/s, so $K_{geotextile}$ >0.04 cm/s

Using design criteria for a nonwoven geotextile in Design Note 24, Guide for the Use of Geotextiles (USDA SCS 1991) (table TS14D–3).

AOS ≤0.425 mm (#40 sieve)
A mechanically bonded needle-punched nonwoven geotextile is required.

Using design criteria from the AASHTO M–288 Geotextile Specification for Highway Applications (AASHTO 2000)

Since this is a permanent erosion control (AASHTO M–288) (table TS14D–4), use Class 2 for woven geotextiles and Class 1 for woven geotextiles.

Soil contains 25 percent finer than the #200 sieve so:
Permittivity= 0.2 s^{-1}
AOS ≤0.25 mm (#60 sieve)
Woven slit film geotextiles are not allowed.

A summary of the design using the three criteria is shown in table TS14D–5.

Table TS14D–1 Example problem soil gradation

Soil sample gradation	
Size	**% Finer**
#40 (0.42 mm)	100
#60 (0.25 mm)	98
#140 (0.105 mm)	60
#200 (0.074 mm)	25
0.005 mm	4

Table TS14D–2 Requirements for woven geotextiles

Property	Test method	Class I	Class II and III	Class IV 3/
Tensile strength (lb)1/	ASTM D4632 Grab test	200 min. in any principal direction	120 min. in any principal direction	180 min. in any principal direction
Bursting strength (lb/in^2)1/	ASTM D3786 Diaphragm tester	400 min.	300 min.	NA
Elongation at failure (%)	ASTM D4632 Grab test	<50	<50	<50
Puncture (lb)	ASTM D4833	90 min.	60 min.	60 min.
Ultraviolet light (percent residual tensile strength)	ASTM D4355 150-hr exposure	70 min.	70 min.	70 min.
Apparent opening size (AOS)	ASTM D4751	As specified, or a min. #100 2/	As specified, or a min. #100 2/	As specified, or a min. #100 2/
Percent open area (%)	CWO–02215–86	4.0 min.	4.0 min.	1.0 min.
Permittivity (1/s)	ASTM D4491	0.10 min.	0.10 min.	0.10 min.

1/ Minimum average roll value (weakest principal direction)
2/ U.S. standard sieve size
3/ Heat-bonded or resin-bonded geotextile may be used for Class IV only and are particularly well suited for this use. Needle-punched geotextiles are required for all other classes.

Table TS14D–3 Requirements for nonwoven geotextiles

Property	Test method	Class I	Class II	Class III	Class IV 3/
Tensile strength (lb)1/	ASTM D4632 Grab test	180 min.	120 min.	90 min.	115 min.
Bursting strength (lb/in^2)1/	ASTM D3786 Diaphragm tester	320 min.	210 min.	180 min.	WA
Elongation at failure (%)	ASTM D4632 Grab test	>50	>50	>50	>50
Puncture (lb)	ASTM D4833	80 min.	60 min.	40 min.	40 min.
Ultraviolet light (percent residual tensile strength)	ASTM D4355 150-hr exposure	70 min.	70 min.	70 min.	70 min.
Apparent opening size (AOS)	ASTM D4751	As specified, max. #40 2/	As specified, max. #40 2/	As specified max. #40 2/	As specified, max. #40 2/
Permittivity (1/s)	ASTM D4491	0.70 min.	0.70 min.	0.70 min.	0.70 min.

1/ Minimum average roll value (weakest principal direction)
2/ U.S. standard sieve size
3/ Heat-bonded or resin-bonded geotextile may be used for Class IV only and are particularly well suited for this use. Needle-punched geotextiles are required for all other classes.

Table TS14D–4 Default geotextile class and design class for the subsurface drainage, permanent erosion control, separation, and stabilization applications

Application class	Default class	Design
Subsurface drainage	Class 2	Class 3
Permanent erosion control	Class 2 for woven monofilaments	Class 2
	Class 1 for all others	
Separation	Class 2	Class 3
Stabilization	Class 1	Class 2 or 3

Table TS14D–5 Summary of design solutions for example problem

Property	NRCS DN 24 – Woven	NRCS DN 24 – Nonwoven	AASHTO M–288
AOS	0.155 mm (#100 Sieve)	0.425 mm (#40 Sieve)	0.25 mm (#60 Sieve)
Permeability	0.04 cm/s	NA	NA
Permittivity	NA	NA	0.2 s^{-1}

Technical Supplement 14E

Use and Design of Soil Anchors

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Anchoring materials into the streambed and bank can be a significant challenge due to the variable hydraulic forces and the variable earth material strengths.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14E

The Use and Design of Soil Anchors

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Technical Supplement 14E

The Use and Design of Soil Anchors

Purpose

The success of a soil bioengineering project that uses large woody material (LWM) structures depends on proper anchoring design. This technical supplement presents three of the more common anchoring methods used: driven soil anchors, screw-in soil anchors, and cabling to boulders or bedrock. Also covered is a method for estimating the pullout capacity required of the anchor and another method for connecting of the anchor to a LWM structure. Selecting the anchoring method and sizing the anchor require information about the expected streamflows and soil characteristics. The required pullout capacity per anchor can be estimated from the streamflow information, and the anchor type and method can be selected from the soil information. Once the anchor has been installed, the LWM structure must be firmly held into place by the anchor. This requires applying tension to the wire rope that connects the anchor to the LWM structure. An effective method for achieving this is described.

Introduction

Anchoring is required to hold LWM structures and brush revetments against streambanks and streambeds. During high flows, material placed in the streambed or on the streambank will be subject to drag forces, buoyancy forces, and, possibly, impact forces. Proper anchoring is required to resist these forces and firmly hold the structure in place. Since impact forces are difficult to predict, the factor of safety used in the calculations is assumed to be sufficient to account for impact forces.

Failure of an anchoring system on a LWM structure could cause damage to the embankment and downstream structures. Undersized anchors and loose connections contribute to the majority of failures. A proper connection is required between an anchor and a LWM structure to firmly hold the structure in place.

Calculating the forces acting on a LWM structure

Before the anchor method and anchor size can be selected, an estimation of the needed pullout capacity per anchor must be calculated. A simplified method for estimating the forces acting on a LWM structure is provided in this technical supplement. This approach uses project-specific information about soil characteristics, stream velocity at a flow that submerges the structure, and debris load. Much of the information used in this approach will be difficult to obtain or approximate. As a result, a factor of safety is used to account for the lack of data. The designer must consider the impact of an anchor failure when determining the factor of safety.

Soil anchor types

Soil anchors are an effective way to anchor LWM structures. The two types described here are driven anchors and screw anchors. Both anchors are available in different configurations and sizes, with various holding capacities. The anchors can be installed manually in certain soil conditions and have pullout capacities of up to 5,000 pounds. Much greater pullout capacities can be obtained with both anchor types, but a mechanical means of installation is required. Estimates of pullout capacities for anchors in different classes of soils are available in tables published by the manufacturers.

Driven anchors

Driven-type soil anchors are available in different configurations and sizes. They are pushed vertically into the soil to the recommended depth and then are locked into a horizontal position.

Information and supply can be obtained from vineyard, landscape, and utility supply companies. Some of the more common trade names are:

- Duckbill®
- Platipus Stealth®

- Manta Ray®
- Platipus Bat®
- Stingray®

The Duckbill® and the Platipus Stealth® (fig. TS14E–1) are similar in that they are cylindrical-shaped anchors with approximately equal pullout capacities. They are referred to as low-capacity anchors in this technical supplement. The Manta Ray® and Platipus Bat® also can be grouped as similar anchors since they have similar shape and pullout capacities. They are referred to as medium-capacity anchors in this technical supplement. In easy-to-penetrate soils such as wet silts and clays, the Manta Ray® and Platipus Bat® anchors can be installed using a jackhammer, but in most other soils, installation will require heavy equipment.

Stingray® anchors are referred to as high-capacity anchors in this technical supplement. They are more difficult to install, but achieve considerably greater pullout capacities. The Stingray® anchors require heavy equipment for installation.

The pullout capacity of specific driven anchors can be determined from manufacturer tables. Various manufacturer tables are provided at the end of this technical supplement as a guide for anchor selection.

Normally, wherever a stake can be driven or a hole can be drilled, a driven-type anchor can be installed. The anchor is driven by using a drive rod (fig. TS14E–2) to push the anchor to the specified depth into the soil. Note that the bar in figure TS14E–2 has a tapered end, so it is easily removable from the soil anchor. It is important that the soil anchor be driven as close as possible to parallel with the direction of the pull force.

Multiple methods can be used to provide the force needed to push the anchor into the soil. A smaller

Figure TS14E–1 Platipus Stealth® anchor

Figure TS14E–2 Drive rod being inserted into Duckbill® anchor prior to installation

anchor can be driven with a sledgehammer or a post-driver in easy-to-penetrate soils (fig. TS14E–3).

In soils that are harder to penetrate, such as compacted gravels, a jackhammer is effective. Figure TS14E–4 shows a 30-pound jackhammer being used to drive a Duckbill® model 88 anchor into such soils. On this particular job, the manual method of using a sledgehammer was tried without success, but the 30-pound jackhammer was very effective. In soils and soft rock that are very hard to penetrate, a pilot hole can be drilled to assist the installation of a cylindrically shaped soil anchor. Manufacturer specifications should be reviewed for size of pilot holes for the anchor being used.

If greater holding capacities are required, a plate-type anchor, such as a Manta Ray® soil anchor or similar, can be used. In easy-to-penetrate soils, Manta Ray® anchors can be driven with a jackhammer. In medium to hard soils, larger equipment, such as a backhoe with a vibratory plate attachment or a rock breaker attachment, is necessary. Once the soil type and required holding capacity are known, manufacturer data should be used to determine the appropriate size for this type of anchor.

Once a driven-type soil anchor has been pushed to the specified depth, it must be locked into place. This is done by applying tension to the anchor cable. As the anchor cable is pulled up, the bill of the flat part of the anchor catches the edge of the pilot or drive hole. This causes the anchor plate to rotate 90 degrees from its driven position. The anchor now presents its maximum surface area against the pulling forces.

In the locked position, the anchor is capable of obtaining its ultimate holding capacity for the particular soil and depth. In easy-to-penetrate soils, small anchors can be locked using a lever mechanism, such as the

Figure TS14E–3 Post driver being used to install soil anchors

Figure TS14E–4 Driving soil anchor with a 30-lb jackhammer

drive bar, to pry the anchor into the locked position. In soils that are harder to penetrate, a Hi-Lift® jack (fig. TS14E–5) can be used to lock the anchor. Figure TS14E–6 shows a Hi-Lift® jack being used to lock a Duckbill® anchor. Larger anchors require an anchor-locking base with a hydraulic ram system that is made specifically for locking the anchor into position and proof-testing the holding capacity of the anchor. The proof-tested holding capacity should be compared with design values to assure adequate anchorage.

Screw-in anchors

Screw-in soil anchors (fig. TS14E–7) are another option for anchoring LWM. Screw-in anchors can be used in loose to medium dense, fine to coarse sand and sandy gravels, and firm to very stiff silts and clays. They can have a single helical disk or multiple disks that, when rotated, will auger itself into the soil. These anchors are available in multiple sizes. Smaller screw-in soil anchors, like the ones that can be purchased at a farm supply store, can be installed in silty clay soils without rocks by manually screwing them in, using a cross bar. These manually installed anchors can achieve pull-out capacities of up to 4,000 pounds. Larger screw-in soil anchors require heavy equipment for installation. Drilling attachments for tractors, backhoes, and boom trucks are commonly used to install large screw-in soil anchors. The anchor must be installed parallel with the direction of pull.

Figure TS14E–5 Hi-Lift® jack

Figure TS14E–6 Hi-Lift® jack being used to load-lock a Duckbill® anchor

Figure TS14E–7 Screw-in anchor

Cabling (wire rope) to boulders or bedrock

Boulders or bedrock, when available, can be used to anchor structures. Boulders may exist onsite or be incorporated into the design for bank toe stabilizing. Whichever the case, it is possible to strategically place the boulders so that they can be used as anchors. Figure TS14E–8 shows boulders being used for bank toe stabilization, as well as anchors for a brush revetment.

Cabling to boulders or bedrock requires drilling a hole in the rock and using epoxy to secure an eyebolt (fig. TS14E–9) or the ends of wire rope (fig. TS14E–10) into the rock. Follow the epoxy manufacturers specifications for hole diameter, depth, and time required for the epoxy to set. The hole must be free of dust and debris, and the eyebolt or wire rope must be free of any dust, dirt, and lubrication to allow a proper bond.

Wire rope

Wire rope is typically used to attach LWM structures to the anchors. It comes in a range of sizes, constructions, and materials. The characteristics that are generally most essential in soil bioengineering projects are the breaking strength, flexibility, and corrosion-resistance. Wire rope must be flexible enough to make a tight wrap around a LWM structure. In soil bioengineering projects, the wire rope will be exposed to the weather with portions of the wire rope at times submerged in water or buried in the soil. Using galvanized or stainless steel wire rope can provide added corrosion resistance.

Figure TS14E–9 Eyebolt anchored in boulder with epoxy

Figure TS14E–8 Boulders serve dual purpose: to stabilize the toe and secure brush revetment

Figure TS14E–10 Wire rope anchored in boulder with epoxy

Once the total force per anchor (F_t/Anchor) has been calculated, the breaking strength required of the wire rope can be obtained by multiplying the force per anchor by a minimum factor of safety (FS) of 2 to determine the minimum breaking strength required from the wire rope. A factor of safety of 2 is used to account for corrosion and wear over time, as well as impact forces. A minimum of 1/8-inch-diameter wire rope should be used. However, the designer should not necessarily select the thickest cable available because too thick of a cable may not be flexible enough to secure tightly for some applications.

Connectors and tensioning

Proper tensioning of the wire rope to the LWM is essential. Many problems can result from a loose connection between the anchor and LWM such as oscillating forces resulting in the anchor pulling out, increased erosion of the bank or streambed, or the LWM breaking loose from the wire rope.

An effective method for tensioning wire rope around LWM uses ratcheting type cable clamps (fig. TS14E–11) and a special tensioning tool (fig. TS14E–12). Two pieces of wire rope connected to Duckbill® anchors are connected together with a Gripple® wire rope grip One such type is manufactured by Gripple®. The ratcheting type cable clamp is used for connecting two pieces of wire rope or a single piece that is looped back through the wire rope grip. The wire rope grip allows the wire rope to pass through the wire rope grip in one direction only. With the use of the tensioning tool the wire rope is pulled through the wire rope grip, applying tension to the wire rope. Wire rope ratcheting type cable clamps can be obtained in different sizes with working load limits up to 4,000 pounds. Wire clamps can be added if the design indicates that the wire rope grip capacity will be exceeded or as an added precaution after the wire rope has been tensioned.

Figure TS14E–11 Ratcheting-type cable clamp—allows tension to be applied between two cables

Figure TS14E–12 Gripple® wire rope grip and tensioning tool being used to tension down a brush spur

Method for calculating forces acting on a LWM structure

This technical supplement provides a simplified method for calculating forces on a LWM structure. A more detailed approach is provided in technical supplement 14J of this handbook. The resulting calculation can be used to select the appropriate soil anchor. It should be noted that this simplification may not be applicable in all situations, and a more involved analysis may be necessary.

The forces acting on a LWM structure include the drag force from the water flow, a buoyancy force, and impact forces from debris. Since impact forces are less predictable, the equation includes potential impact forces by increasing the debris or increasing the factor of safety.

Drag force

The following empirical equation, based on Stoke's Law (Stokes 1851), can be used to estimate the drag force (F_d) in pounds on the LWM structure:

$$F_d = 0.95(A)(v)^2(D)(K) \qquad \text{(eq. TS14E–1)}$$

where:

A = surface area (ft^2) of the LWM structure that is perpendicular to the flow and exposed to the current. This area should include the areas of voids that could potentially fill with debris.

Many LWM structures will have irregular surface areas; for example, full size trees with branches still attached, rootwads, or multiple trees and brush attached together to create one structure. The following methods can be used to account for the irregular, semipermeable areas, each of which requires an estimation of the void areas.

Method 1—First, estimate the surface area of the whole structure including the voids. Then, estimate the percent of the area that is voids that is not anticipated to plug or fill with debris, and subtract it from the surface area of the structure. If this method is used, the permeability coefficient (K) should be 1.0.

Method 2—First, estimate the surface area of the whole structure including the voids, and use that as the surface area (A). Then, use the permeability coefficient (K) to account for the voids in the structure.

v = expected stream velocity (ft/s)
D = estimated debris increase factor

The debris increase factor is generally between 1 and 1.5. Estimating this factor requires engineering judgment from observation of the debris load on existing stationary objects within the stream and potential for the addition of debris from the streambanks and tributaries. Take notice of the debris load on bridge columns and/or abutments, fallen trees that extend into the stream or have lodged within the stream, or any other stationary object within the stream that could catch debris. Figure TS14E–13 shows an example of a stream with potential for additional debris load on a LWM structure. From these observations and considering the potential damage if an anchor failed, estimate the percent increase in surface area that is perpendicular to the flow, and use that as the debris increase factor.

K = permeability coefficient

This factor is figured by estimating the percentage of voids in the surface area that are not anticipated to plug/fill with debris. Use conservative judgments when making this estimate. If method 1 is used to calculate the surface area, the permeability coefficient (K) is 1.0.

Figure TS14E–13 Debris lodged against rootwads

Buoyancy force

The buoyancy force (F_b) can be estimated by:

$$F_b = V\left(\gamma_w - \gamma_{(LWM)}\right) \qquad \text{(eq. TS14E–2)}$$

where:
V = volume (ft^3) of LWM submerged
γ_W = density of water (62.4 lb/ft^3)
$\gamma_{(LWM)}$ = density of LWM (lb/ft^3) (calculated from the following equation)
$\gamma_{(LWM)} = G_S(\gamma_W)(\omega)$

where:
G_S = specific gravity of wood
ω = (1+moisture content, as a decimal)

The unit density (γ) of the LWM can be calculated from the specific gravity of the wood (G_S) and the expected moisture content (ω). The average moisture content of wood that has been air dried for an extended period is 12 percent. For LWM structures using a moisture content of 12 percent would be a good conservative estimate. The specific gravity for different species of wood in the United States is given in table TS14E–5. The USDA Forest Service compiled these tables at their Forest Service Laboratory. Typical unit densities for wood with 12 percent moisture content range from 25 pounds per cubic foot to 40 pounds per cubic foot.

Once the drag force and the buoyancy force have been calculated, the total force per anchor (F_t/Anchor) is calculated using the following equation:

$$\frac{F_t}{\text{anchor}} = \frac{FS\left(F_d + F_b\right)}{A_n} \qquad \text{(eq. TS14E–3)}$$

where:
FS = factor of safety
A_n = number of anchors

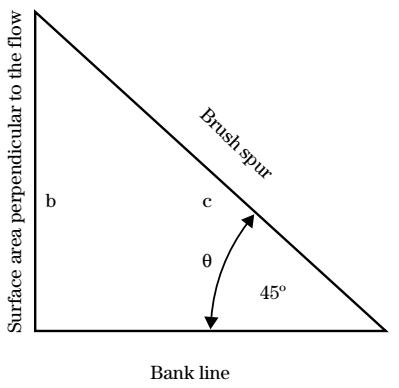
The factor of safety used depends on the potential damages that would occur if an anchor were to fail, as well as the level of confidence in the design assumptions such as potential impact loads from debris and extent of soils information available. Factors of safety for LWM structures typically range from 1.5 (when limited impact loads are expected and soil characteristics are known) to 3.0 (when impact loads are unknown, and/or the soil characteristics are unknown).

Example calculation

Problem:
Brush spurs made from willow brush are designed for a soil bioengineering project to deflect the water flow away from a streambank toe and facilitate the accumulation of sediment between the spurs. The spurs are 20 feet long, 3 feet high, and 3 feet wide and are placed at a 45-degree angle from the streambank, pointing in the upstream direction (fig. TS14E–14). The stream velocity for flow above the brush spur was measured at 4 feet per second. Estimate the force per anchor during a storm event that completely submerges the brush spurs.

Solution:
Estimate the drag force acting on the structure using equation TS14E–1.

Solve for the surface area (A) perpendicular to the flow:



$A = \text{length} \times \text{height}$

$$\sin\theta = \frac{\text{opp}}{\text{hyp}} = \frac{b}{c}$$

$b = 0.707 \times 20\ \text{ft} = 14.1\ \text{ft}$

$A = 14.1\ \text{ft} \times 3\ \text{ft (height, given)} = 42.4\ \text{ft}$

v = given as 4 ft/s

$D = 1.25$ (After observation of debris build up on stationary objects within the stream and its tributaries)

$F_d = 0.95\left(42.4\ \text{ft}^2\right)\left(4\ \text{ft/s}\right)^2\left(1.25\right) \times 1 = 802\ \text{lb}$

$K = 1$ (brush spur is well compacted, making it fairly impervious)

Estimate the buoyancy force acting on the structure using equation TS14E–2.

First estimate the density (γ) of the wood using the following equation.

$$\gamma_{(LWM)} = G_S(\gamma_W)(\omega)$$

ω = + 12% = 1.12 (12% is the typical air dried moisture content)
G_S = 0.39 (table TS14E–5)
γ_W = 62.4 lb/ft^3
$\gamma_{(LWM)}$ = 0.39(62.4 lb/ft^3)(1.12)= 27.3 lb/ft^3

Estimate the volume (V) by assuming 60 percent of the brush spur is wood:

V = 20 ft(3 ft)(3 ft)(0.60)= 108 ft^3

So, the buoyancy force (F_b) is:

F_b = 108(62.4 – 27.3) = 3,791 lb

Estimate the total force per anchor (F_t/anchor) using equation TS14E–3.

$$\frac{F_t}{\text{anchor}} = \frac{FS\left(F_d + F_b\right)}{A_n}$$

FS = 1.5
A_n = 6 anchors
F_t/anchor = $1.5\left(802\text{ lb} + 3{,}791\text{ lb}\right) \div 6 = 1{,}148$ lb/anchor

Anchor manufacturer data

The anchors in table TS14E–1 (Foresight Products 2001) are rated in an average soil condition (class 5). Soil classes are listed in table TS14E–2 (A.B. Chance Company). A torque probe can be used for quick soil classification in the field. A core sampler could also be used to obtain *in-situ* soil samples, but they are expensive and take time to obtain test results. Higher capacities can be expected in the numerically lower classes and less capacity in the higher number classes. If the soil is something other than a class 5, the rated capacity can be calculated by dividing the actual, if known, or the average probe value for that particular soil by the average probe value for a class 5 soil and multiplying times the rated capacity given in tables TS14E–1, TS14E–3 (Foresight Products 2001), or TS14E–4 (Foresight Products 2001). Generally, resistance to driving an anchor is a good indicator of its pullout capacity, but proof-loading is the only way to ensure the exact pullout capacity of any soil anchor.

Figure TS14E–14 Example problem, plan view

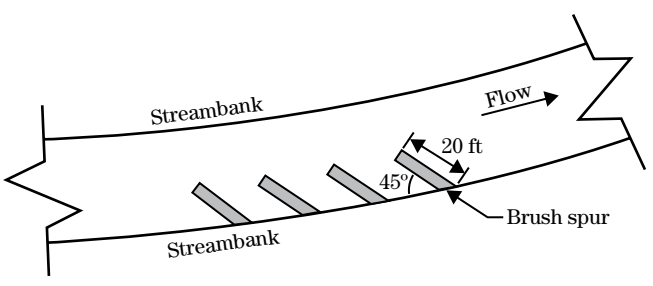


Table TS14E–1 Duckbill® specifications (rated for class 5 soils, see table TS14E–2)

Duckbill model no.	Rated capacity (lb)	Drive rod diameter (in)	Normal depth of installation
40	300	1/4	20 in
68	1,100	1/2	2 1/2 ft
88	3,000	3/4	3 1/2 ft
138	5,000	1	5 ft

Table TS14E–2 Soil classification

Class	Description	Probe value
1	Solid bedrock	—
2	Dense clay; compact gravel dense fine sand; laminated rock; slate; schist; sand stone	Over 600 in/lb
3	Shale; broken bedrock; hardpan; compacted gravel clay mixture	500–600 in/lb
4	Gravel; compacted gravel and sand; claypan	400–500 in/lb
5	Medium-firm clay; loose standard gravel; compacted coarse sand	300–400 in/lb
6	Medium-firm clay; loose course sand; clayey silt; compact fine sand	200–300 in/lb
7	Fill; loose fine sand; wet clays; silt	100–200 in/lb
8	Swamp; marsh; saturated silt; humus	Under 100 in/lb

Table TS14E–3 Manta Ray® ultimate holding capacity

Soil description	Blow count (N)	MR–88 ultimate= 10 kips	MR–4 ultimate= 16 kips	MR–3 ultimate= 20 kips	MR–2 ultimate= 40 kips	MR–1 ultimate= 40 kips	MR–SR ultimate= 40 kips	MK–B ultimate= 40 kips
Very dense and/or cemented sands; coarse gravel and cobbles	60+	10 (1,3)	16 (1,3)	20 (1,3)	28–40 (1,3,4)	40 (1,3,)	40 (1,3,5)	40 (1,3,5)
Dense, fine, compacted sands; very hard silts or clays	45–60	6–10 (2,3,4)	9–16 (2,3,4)	17–20 (2,3,4)	21–28 (2,4)	36–40 (1,3,4)	40 (1,3)	40 (1,3,5)
Dense clays, sands and gravels; hard silts and clays	35–50	4–6 (4)	6–9 (4)	12–18 (2,4)	15–22 (2,4)	24–36 (2,4)	32–40 (2,3,4)	40 (1,3)
Medium-dense, sandy gravel, stiff to hard silts and clays	24–40	3–4 (4)	4.5–5.5 (4)	9–14 (4)	12–18 (4)	18–20 (2,4)	24–34 (2,4)	32–40 (2,3,4)
Medium-dense, coarse sand and sandy gravel, stiff to very stiff silts and clays	14–25	2–3 (4)	3.5–4.5 (4)	7–9 (4)	9–12 (4)	15–20 (4)	18–24 (4)	24–32 (2,4)
Loose to medium-dense, fine to coarse sand; firm to stiff clays and silts	7–14	1.5–2.5 (4)	2.5–4 (4)	5–8 (4)	7–10 (4)	10–15 (4)	14–18 (4)	20–24 (4)
Loose fine sand; alluvium; soft clays; fine, saturated, silty sand	4–8	0.9–1.5 (4,6)	1.5–2.5 (4,6)	3–5 (4,6)	5–8 (4,6)	8–12 (4,6)	9–14 (4,6)	13–20 (4,6)

1 = Drilled pilot hole required for efficient installation
2 = Ease of installation may be improved by drilling a pilot hole
3 = Holding capacity limited by ultimate strength of anchors
4 = Holding capacity limited by soil structure
5 = Not recommended in these soils
6 = Wide variation in soil properties reduces prediction accuracy. Preconstruction field test is recommended.

Table TS14E–4 Stingray® ultimate holding capacity

Description	Blow count (N)	SR–1 ultimate = 100 kips	SR–2 ultimate = 100 kips	SR–3 ultimate = 100 kips
Very dense and/or cemented sands; coarse gravel and cobbles	60+	65–89 (1,3)	89–100 (1,3)	100 (1,3,5)
Dense, fine, compacted sand; very hard silts and clays	45–60	58–65 (2, 4)	79–89 (2,4)	100 (2,3)
Dense clays, sands and gravel; hard silts and clays	35–50	39–58 (4)	62–79 (2,4)	85–100 (2,3,4)
Medium dense sandy gravel; very stiff to hard silts and clays	24–40	29–41 (4)	46–66 (4)	63–90 (4)
Medium dense coarse sand and sandy gravel; stiff to very stiff silts and clays	14–25	24–32 (4)	31–48 (4)	48–63 (4)
Loose to medium-dense, fine to coarse sand; firm to stiff clays and silts	7–14	16–24 (4)	27–36 (4)	37–48 (4)
Loose, fine sand; alluvium; soft-firm clays; varied clays; fill	4–8	13–19 (4,6)	19–28 (4,6)	24–37 (4)

1 = Drilled hole required to install
2 = Installation may be difficult; pilot hole may be required
3 = Holding capacity limited by structural rating of anchors
4 = Holding capacity limited by soil structure
5 = Not recommended in these soils
6 = Wide variation in soil properties reduces prediction accuracy. Preconstruction field test recommended

Specific gravity of wood

Tabls TS14E–5 provides a summary of specific gravities for some commercially important wood grown in the United States. The designer may want to adjust these values based on age or condition of the wood used in the project or to provide for a factor of safety.

Conclusion

Proper anchoring of LWM structures is essential to the success of a soil bioengineering project. Choosing the most applicable anchoring method depends on the pullout capacity required of the anchor, site conditions such as streambed and streambank soil characteristics, site access for construction equipment, and material availability.

Site access or equipment availability may be the deciding factor in the anchor method selected. Manual installation may be possible for some projects, but much greater pullout capacities can be achieved from an anchor that requires some type of mechanical installation. For example, driven anchors that require a jackhammer for installation can achieve much greater pullout capacities than ones that can be manually driven. In most locations, a jackhammer and compressor can be rented fairly inexpensively and can greatly decrease the effort and time required to install a driven anchor. Once the anchor has been selected, it is essential that the LWM structure be properly tensioned to the anchor to prevent movement.

Table TS14E–5 Specific gravity values for some commercially important woods grown in the United States

Hardwood

Common species	Moisture content	Specific gravity[1]
Alder, red	Green	0.37
	12%	0.41
Ash		
Black	Green	0.45
	12%	0.49
Blue	Green	0.53
	12%	0.58
Green	Green	0.53
	12%	0.56
Oregon	Green	0.5
	12%	0.55
White	Green	0.55
	12%	0.6
Aspen		
Bigtooth	Green	0.36
	12%	0.39
Quaking	Green	0.35
	12%	0.38
Basswood		
American	Green	0.32
	12%	0.37
Beech		
American	Green	0.56
	12%	0.64
Birch		
Paper	Green	0.48
	12%	0.55
Sweet	Green	0.60
	12%	0.65
Yellow	Green	0.55
	12%	0.62
Butternut	Green	0.36
	12%	0.38

Common species	Moisture content	Specific gravity[1]
Cherry		
Black	Green	0.47
	12%	0.50
Chestnut		
American	Green	0.40
	12%	0.43
Cottonwood		
Balsam, Poplar	Green	0.31
	12%	0.34
Black	Green	0.31
	12%	0.35
Eastern	Green	0.37
	12%	0.40
Elm		
American	Green	0.46
	12%	0.50
Rock	Green	0.57
	12%	0.63
Slippery	Green	0.48
	12%	0.53
Hackberry	Green	0.49
	12%	0.53
Hickory, Pecan		
Bitternut	Green	0.60
	12%	0.66
Nutmeg	Green	0.56
	12%	0.60
Pecan	Green	0.60
	12%	0.66
Water	Green	0.61
	12%	0.62

Table TS14E–5 Specific gravity values for some commercially important woods grown in the United States—Continued

Common species	Moisture content	Specific gravity[1]
Hickory, True		
Mockernut	Green	0.64
	12%	0.72
Pignut	Green	0.66
	12%	0.75
Shagbark	Green	0.64
	12%	0.72
Shellbark	Green	0.62
	12%	0.69
Honeylocust	Green	0.60
	12%	—
Locust		
Black	Green	0.66
	12%	0.69
Magnolia		
Cucumbertree	Green	0.44
	12%	0.48
Southern	Green	0.46
	12%	0.50
Maple		
Bigleaf	Green	0.44
	12%	0.48
Black	Green	0.52
	12%	0.57
Red	Green	0.49
	12%	0.54
Silver	Green	0.44
	12%	0.47
Sugar	Green	0.56
	12%	0.63
Oak, Red		
Black	Green	0.56
	12%	0.61
Cherrybark	Green	0.61
	12%	0.68
Laurel	Green	0.56
	12%	0.63
Northern Red	Green	0.56
	12%	0.63
Pin	Green	0.58
	12%	0.63
Scarlet	Green	0.60
	12%	0.67
Southern Red	Green	0.52
	12%	0.59
Water	Green	0.56
	12%	0.63
Willow	Green	0.56
	12%	0.69
Oak, White		
Bur	Green	0.58
	12%	0.64
Chestnut	Green	0.57
	12%	0.66
Live	Green	0.80
	12%	0.88
Overcup	Green	0.57
	12%	0.63
Post	Green	0.60
	12%	0.67
Swamp Chestnut	Green	0.60
	12%	0.67
Swamp White	Green	0.64
	12%	0.72
White	Green	0.60
	12%	0.68
Sweetgum	Green	0.46
	12%	0.52
Sycamore		
American	Green	0.46
	12%	0.49
Tanoak	Green	0.58
	12%	—

Table TS14E–5 Specific gravity values for some commercially important woods grown in the United States—Continued

Common species	Moisture content	Specific gravity[1]
Tupelo		
Black	Green	0.46
	12%	0.50
Water	Green	0.46
	12%	0.50
Walnut		
Black	Green	0.51
	12%	0.55
Willow		
Black	Green	0.36
	12%	0.39

Softwood

Common species	Moisture content	Specific gravity[1]
Baldcypress	Green	0.42
	12%	0.46
Cedar		
Atlantic White	Green	0.31
	12%	0.32
Eastern redceder	Green	0.44
	12%	0.47
Incense	Green	0.35
	12%	0.37
Northern White	Green	0.29
	12%	0.31
Port-Orford	Green	0.39
	12%	0.43
Western redceder	Green	0.31
	12%	0.32
Yellow	Green	0.42
	12%	0.44
Douglas-fir[2]		
Coast	Green	0.45
	12%	0.48
Interior West	Green	0.46
	12%	0.50
Interior North	Green	0.45
	12%	0.48
Interior South	Green	0.43
	12%	0.46
Fir		
Balsam	Green	0.33
	12%	0.35
California Red	Green	0.36
	12%	0.38
Grand	Green	0.35
	12%	0.37
Noble	Green	0.37
	12%	0.39
Pacific Silver	Green	0.40
	12%	0.43
Subalpine	Green	0.31
	12%	0.32
White	Green	0.37
	12%	0.39
Hemlock		
Eastern	Green	0.38
	12%	0.40

Table TS14E–5 Specific gravity values for some commercially important woods grown in the United States—Continued

Common species	Moisture content	Specific gravity[1]
Mountain	Green	0.42
	12%	0.45
Western	Green	0.42
	12%	0.45
Larch		
Western	Green	0.48
	12%	0.52
Pine		
Eastern White	Green	0.34
	12%	0.35
Jack	Green	0.40
	12%	0.43
Loblolly	Green	0.47
	12%	0.51
Lodgepole	Green	0.38
	12%	0.41
Longleaf	Green	0.55
	12%	0.59
Pitch	Green	0.47
	12%	0.52
Pond	Green	0.51
	12%	0.56
Ponderosa	Green	0.38
	12%	0.40
Red	Green	0.41
	12%	0.46
Sand	Green	0.46
	12%	0.48
Shortleaf	Green	0.47
	12%	0.51
Slash	Green	0.54
	12%	0.59
Spruce	Green	0.41
	12%	0.44
Sugar	Green	0.34
	12%	0.36
Virginia	Green	0.45
	12%	0.48
Western White	Green	0.35
	12%	0.38
Redwood		
Old-Growth	Green	0.38
	12%	0.40
Young-Growth	Green	0.34
	12%	0.35
Spruce		
Black	Green	0.38
	12%	0.42
Engelmann	Green	0.33
	12%	0.35
Red	Green	0.37
	12%	0.40
Sitka	Green	0.37
	12%	0.40
White	Green	0.33
	12%	0.36
Tamarack	Green	0.49
	12%	0.53

Technical Supplement 14F

Pile Foundations

Issued August 2007

Cover photo: Pile foundations may be needed where the bearing strength of the earth materials is low.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14F

Pile Foundations

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Technical Supplement 14F

Pile Foundations

Purpose

Piles are used to transfer foundation forces through relatively weak soil to stronger strata to minimize settlement. The most likely applications for pile foundations in stream restoration and stabilization projects are as support for bank stabilization structures (retaining wall) and anchors for large woody material (LWM). Piles may be used to support ancillary structures such as culverts, structural channels, bridges, and pumping stations. This technical supplement addresses the analyses required to design pile foundations.

Introduction

Foundation structures may be classified into two categories: shallow and deep. There is no specific rule that defines when a particular structure is considered to be shallow or deep. In general, shallow structures are constructed fairly close to ground surface and are usually constructed upwards from the bottom surface of an excavation.

Traditionally, deep foundations refer to piles that are driven into the ground. However, piles are sometimes set into holes that are prebored or drilled into the ground. A hole bored into the ground and filled with concrete is called a drilled shaft. Usually, reinforcing steel is placed into the drilled hole just prior to placement of the concrete. Other terms for drilled shafts are: drilled piers, drilled caissons, cast-in-place piles, cast-in-drilled-hole piles, and augered piles. Another type of drilled shaft foundation is an auger-cast pile.

Piles are normally used to provide foundation capacity to support a structure when the bearing capacity of the soil is insufficient to do so. If a soil's bearing capacity is less than that needed to support a structure, a shallow foundation may become impractical or expensive. By driving piles into the ground, the pile structure can take advantage of the soil's shear strength, as well as its compressive (or bearing) strength. Piles are also used to transfer foundation forces through relatively weak soil strata to stronger strata to minimize settlement.

Piles may be steel H-sections; steel pipe; precast, prestressed concrete; concrete-filled steel shells; or timber. Piles with solid cross sections, or hollow piles with closed ends, typically displace the soil as they are driven and are termed *displacement* piles. Piles with open cross sections, such as H-sections, or pipe piles without closed ends, typically cut through the soil, rather than displacing it as they are driven, so they are termed *nondisplacement* piles. Whether a pile behaves in a displacing or nondisplacing manner depends heavily on the soil properties. Pile driving may cause cohesive soils to remold and cause density changes in cohesionless soils. These changes may result in ground surface elevation changes (heaving or settling) in the general area of the driving operation. Proper pile driving equipment selection and operation can greatly minimize the possible adverse effects of pile installation.

Pile foundations are used in stream restoration and stabilization projects as support for bank stabilization (retaining wall) structures and anchors for LWM. Piles may be used to support ancillary structures such as culverts, structural channels, bridges, and pumping stations.

Piles are typically installed using specialized pile driving equipment. The motive force that drives the pile into the ground is applied by a pile driving hammer, which is attached to the top of a pile. A crane is used to support the pile driving equipment and handle the individual piles.

Bearing capacity

The allowable bearing or axial capacity of a driven pile may be determined from the following equation:

$$Q_{Allowable} = \frac{Q_{total}}{\text{factor of safety}} \qquad \text{(eq. TS14F–1)}$$

$$Q_{total} = Q_{point} + Q_{friction} - Wt_{pile} \qquad \text{(eq. TS14F–2)}$$

where:

Q_{total}	= ultimate capacity of pile
Q_{point}	= end-bearing capacity of pile
$Q_{friction}$	= capacity due to friction along length of pile
Wt_{pile}	= weight of pile
factor of safety	= 3.0

For a cohesionless soil, the following formulas may be used to determine values (table TS14F–1) for the components of the bearing capacity equation:

$$Q_{point} = \gamma' D N'_q A_{point} \qquad \text{(eq. TS14F–3)}$$

where:

γ' = effective unit weight of soil
D = embedded depth of pile
A_{point} = area of pile tip

$$N'_q = e^{\pi \tan\phi'} \tan^2(45^\circ + \frac{\phi'}{2}) \qquad \text{(eq. TS14F–3a)}$$

where:

ϕ' = $\tan^{-1}$ (0.67 tan ϕ)
ϕ = angle of internal friction of soil

$$Q_{friction} = \tau'_{average} \Sigma_o K_{shape} D \qquad \text{(eq. TS14F–4)}$$

where:

$$\tau'_{average} = \gamma' z K_o \tan\phi' \qquad \text{(eq. TS14F–4a)}$$

where:

γ' = effective unit weight of soil
z = depth (along pile) at point of analysis
K = ratio of lateral to vertical soil stress on pile
K_o = 0.7 for piles loaded in compression
K_o = 0.5 for piles loaded in tension
ϕ' = $\tan^{-1}$(0.67 tan ϕ)
ϕ = soil angle of internal friction

Σ_o = perimeter of pile
K_{shape} = pile shape factor
K_{shape} = 1.000 for round perimeter
= 0.785 for square perimeter

I-beam piles are considered to have a square perimeter with side lengths equivalent to their respective depth and width dimensions.

= 0.95 for octagon perimeter
= 0.84 for hexagon perimeter
= 0.60 for triangular perimeter
D = depth of pile

Table TS14F–1 Prescriptive values for cohesionless soils

Soil type	Angle of internal friction (°)	Angle of friction between soil and pile (°)	Bearing capacity coefficient N_q	Maximum allowable capacity, Q	
				Friction (tons/ft²)	Bearing (tons/ft²)
Clean sand	35	30	11	1.00	100
Silty sand	30	25	7	0.85	50
Sandy silt	25	20	5	0.70	30
Silt	20	15	4	0.50	20

For a cohesive soil, the following formulas may be used to determine values (table TS14F–2) for the components of the bearing capacity equation cited above.

$$Q_{point} = A_{point}\left(7.4c + \gamma' D\right) \qquad \text{(eq. TS14F–5)}$$

where:

A_{point} = area of pile tip
c = shear strength of cohesive soil at pile tip depth
γ' = effective unit weight of soil
D = depth of pile

$$Q_{friction} = c'_{average}\,\Sigma_o\,K_{shape}D \qquad \text{(eq. TS14F–6)}$$

where:

$c'_{average}$ = average effective shear stress of soil, along with a given length, or segment of the pile
Σ_o = perimeter of pile
K_{shape} = pile shape factor
K_{shape} = 1.000 for round perimeter
= 0.785 for square perimeter

I-beam piles are considered to have a square perimeter with side lengths equivalent to their respective depth and width dimensions.

= 0.95 for octagon perimeter
= 0.84 for hexagon perimeter
= 0.60 for triangular perimeter
D = depth of pile or length of pile segment

Table TS14F–2 Prescriptive values for cohesive soils

Clay	Shear strength
Normally consolidated	$c_z = 0.5\gamma' z\left(1 - K_o\right)$
Underconsolidated	$c_z = 0.125\gamma' z\left(1 - K_o\right)$
Overconsolidated by erosion	$c_z = 600\ \text{lb/ft}^2 + 0.5\gamma' z\left(1 - K_o\right)$
Overconsolidated by desiccation	$c_z = 2{,}000\ \text{lb/ft}^2$ for z = 0 ft to 20 ft $c_z = 1{,}200\ \text{lb/ft}^2$ for z = 20 ft to 60 ft $c_z = 3{,}000\ \text{lb/ft}^2$ for z = 60 ft to 160 ft For highly fissured clays, use the following: $c' = 0$ $\phi' = 5^\circ$ to 10° and $\tau'_{average} = \gamma' z\, K_o \tan\phi'$

Notes:
For piles loaded in axial compression, $K_o = 0.7$
For piles loaded in axial tension, $K_o = 0.5$
Effective shear strength, $c' = 0.67c$
Z = depth along pile from ground surface (ft) Z = 0 at ground surface

Lateral load capacity

An approximate lateral capacity of a short, rigid pile driven into a cohesive soil can be determined from the following equation (figs. TS14F–1 and TS14F–2):

$$P_{Allowable} = \frac{P_{Ultimate}}{\text{factor of safety}} \qquad \text{(eq. TS14F–7)}$$

where:
$P_{Allowable}$ = allowable lateral load applied to exposed portion of pile
factor of safety = 3

$$P_{Ultimate} = \sigma B\left[\left(\sqrt{4\left(\frac{D}{2}+H\right)^2+D^2}\right)-2\left(\frac{D}{2}+H\right)\right] \qquad \text{(eq. TS14F–8)}$$

where:
σ = allowable soil stress, $\sigma = 9c$
B = width or diameter of pile
D = depth of pile embedment
H = height of pile (above ground) to centroid of applied load

If a design load is known, the required depth of embedment may be determined from the following equation:

$$D = \frac{P_{Ultimate}}{\sigma B} + \sqrt{2\left(\frac{P_{Ultimate}}{\sigma B}\right)^2 + \frac{4HP_{Ultimate}}{\sigma B}} \qquad \text{(eq. TS14F–9)}$$

For most stream restoration and stabilization applications, a driven pile may be considered to be rigid when:

$$\frac{H}{B} \le 12 \qquad \text{(eq. TS14F–10)}$$

An approximate allowable lateral load for a short, rigid pile driven into a ***cohesionless*** soil may be determined using the following equation (figs. TS14F–3 and TS14F–4.

$$P_{Allowable} = \frac{P_{Ultimate}}{\text{factor of safety}} \qquad \text{(eq. TS14F–11)}$$

where:
$P_{Allowable}$ = allowable lateral load applied to exposed portion of pile
factor of safety = 3

$$P_{Ultimate} = \frac{\gamma' B K_p D^2}{2(H+D)} \qquad \text{(eq. TS14F–12)}$$

where:
γ' = effective unit weight of soil
B = width or diameter of pile
K_p = passive pressure coefficient

$$= \tan^2\left(45^\circ + \frac{\phi}{2}\right) \qquad \text{(eq. TS14F–13)}$$

ϕ = soil angle of internal friction
D = embedded depth of pile

If a design load is known, the required depth of embedment may be determined from the following equation.

$$D = \sqrt{\frac{2P_{Ultimate}}{\gamma' B K_p}(H+D)} \qquad \text{(eq. TS14F–14)}$$

This equation is solved iteratively until both values for D are equivalent.

Figure TS14F–1 Schematic showing applied lateral force and assumed soil reactions for single rigid pile driven into a cohesive soil. Concentrated load applied at top of pile

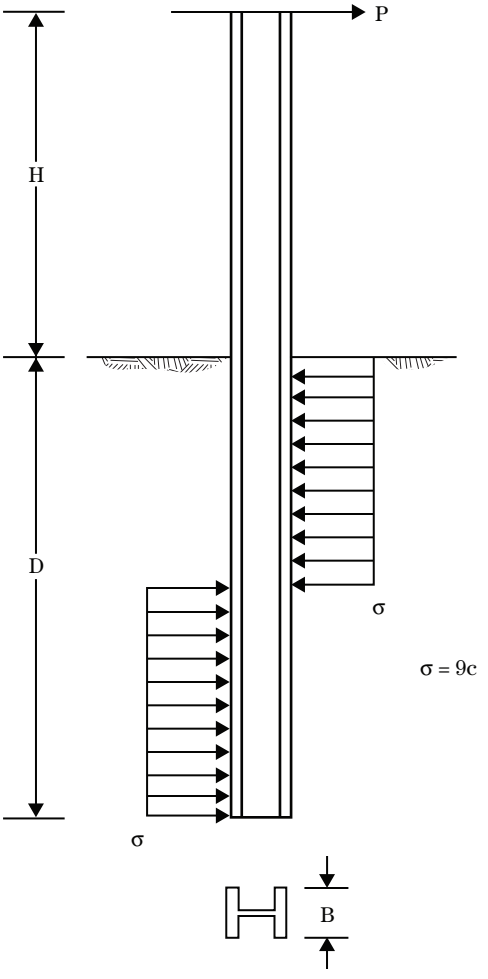


Figure TS14F–2 Schematic showing applied lateral force and assumed soil reactions for a single rigid pile driven into a cohesive soil. Uniformly decreasing load applied

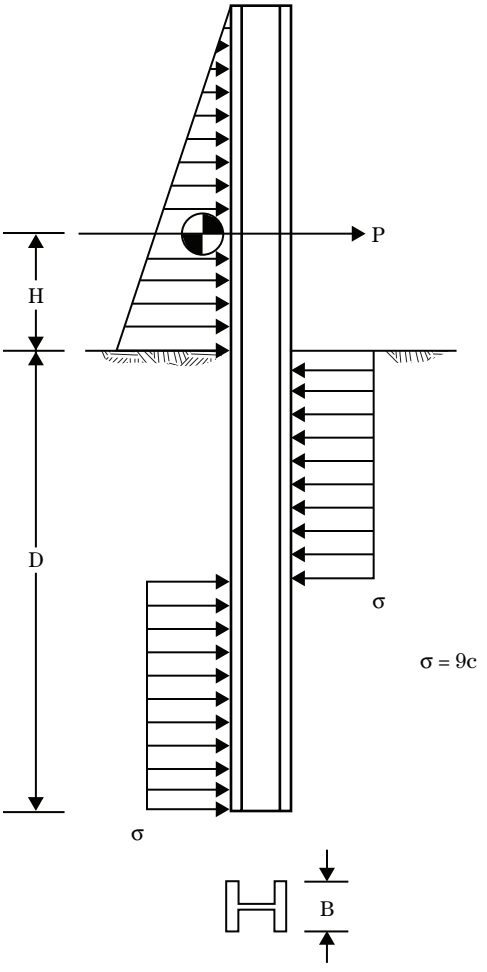


Figure TS14F–3 Schematic showing applied lateral force and assumed soil reactions for single rigid pile driven into a cohesive soil. Concentrated load applied at top of pile

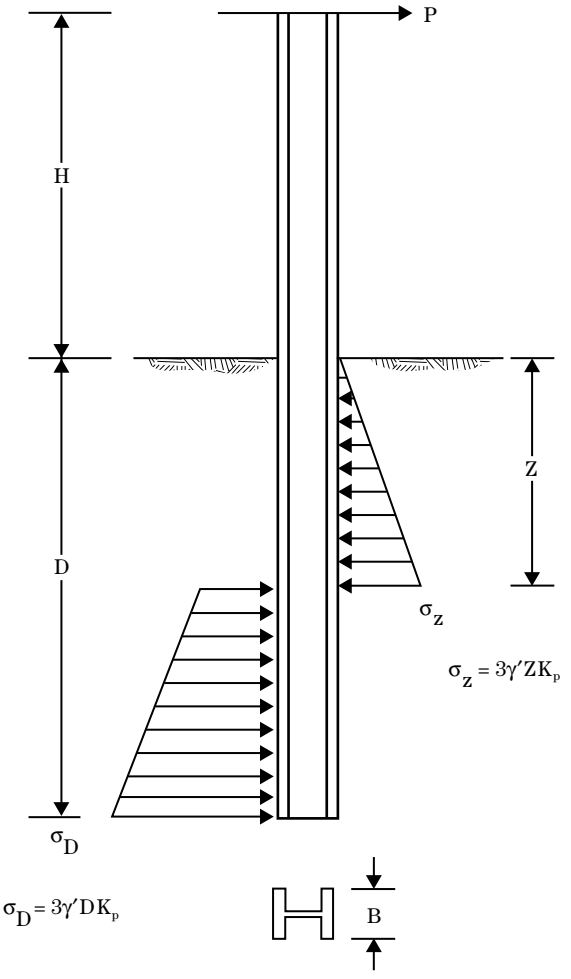
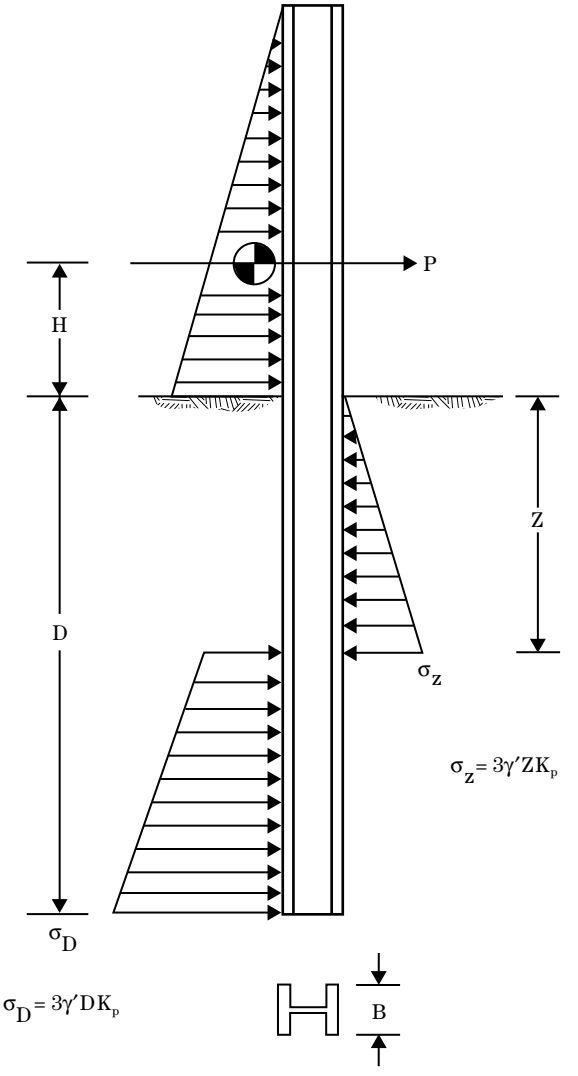
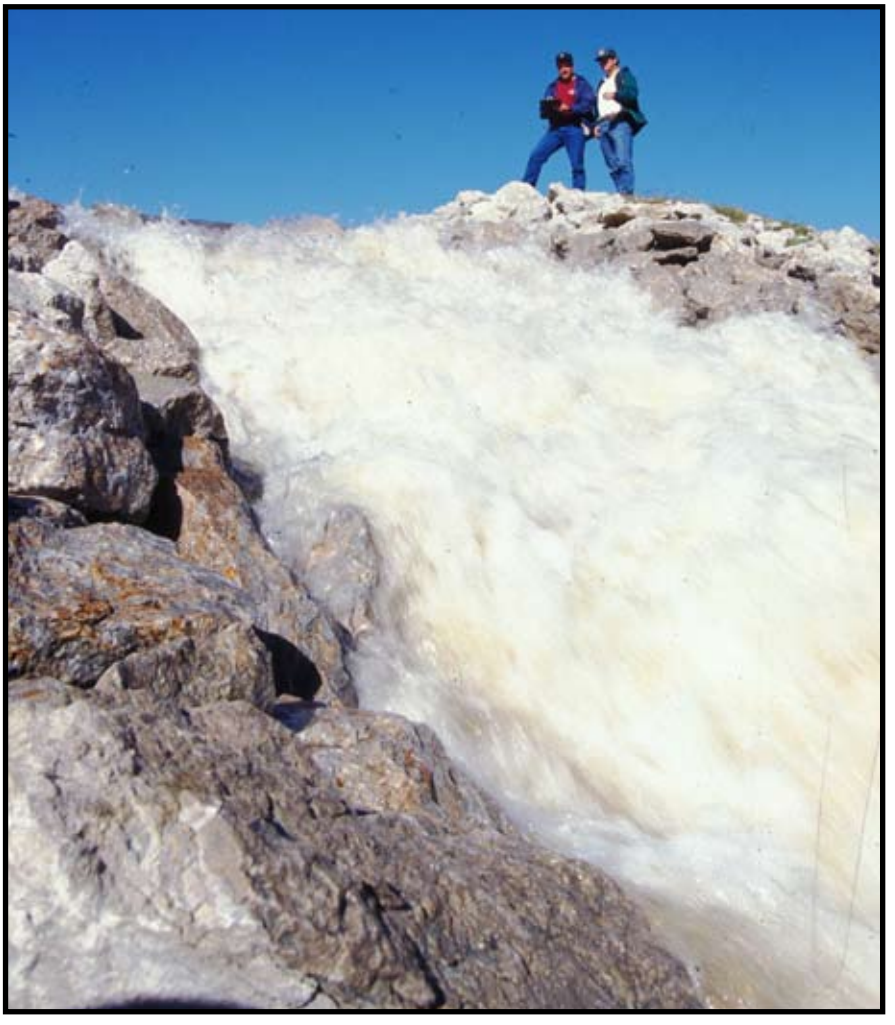


Figure TS14F–4 Schematic showing applied lateral force and assumed soil reactions for single rigid pile driven into a cohesive soil. Uniformly decreasing load applied as shown



Technical Supplement 14G

Grade Stabilization Techniques



(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Streamflow energy may need to be dissipated through the use of inchannel grade control structures.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14G

Grade Stabilization Techniques

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Grade Stabilization Techniques

Purpose

One of the most challenging problems facing river engineers today is the stabilization of degrading channels. Channel degradation leads to damage of both riparian infrastructure, as well as the environment. Bank protection is generally ineffective over the long term and will probably be a waste of resources if the channel continues to degrade. When systemwide channel degradation exists, a comprehensive treatment plan is usually required. A wide variety of structures has been employed to provide grade control in channel systems. The objectives of this technical supplement are to provide a description of some of the more common types of grade control structures that are frequently used throughout the United States and describe the various design factors that should be considered when selecting and siting grade control structures.

Introduction

Grade control is an essential component to stabilize a degrading stream or one that is subject to conditions that may cause degradation. Channel degradation leads to damage of bridges, culverts, petrochemical transmission lines, power lines, sewer and water lines, and other infrastructure. Channel degradation produces an overheightened and oversteepened condition of the channel banks that often leads to severe mass failures of both streambanks. The resulting channel widening and bank erosion cause severe land loss and damage to riparian infrastructure and habitat. As channel degradation continues, the ground water table may also be lowered along the stream, affecting riparian vegetation. Sediment eroded from the degrading channels is transported downstream, adversely impacting flood control channels, reservoir areas, and wetland habitat areas. This sediment also carries significant amounts of nutrients, particularly phosphorus, which may degrade water quality and habitat along the stream system. Consequently, channel degradation is not simply a local problem that only affects a few landowners, but rather, produces systemwide consequences that can impact all taxpayers.

When systemwide channel degradation exists, a comprehensive treatment plan is usually required. This treatment plan usually involves the use of one or more grade control structures to arrest the degradation process. In the widest sense, the term grade control can be applied to any alteration in the watershed that provides stability to the streambed. It can include stream realignments. The most common method of establishing grade control is the construction of inchannel grade control structures. A wide variety of grade control structures has been used in channel systems. These treatments range from simple loose rock structures to reinforced concrete weirs and vary in scale from small streams to large rivers. While some stream rehabilitation practitioners suggest that grade control cannot be constructed in incised channels, the authors have routinely participated in the design and long-term monitoring of successful grade control structures in severely incised channels.

The two primary engineering factors that promote channel stability are continuity of water and sediment through the stream reach and geotechnical bank stability. A series of well-designed grade control structures can adjust sediment transport capacity to sediment supply and can improve bank stability by reducing bank height and reducing shear at the bank toe. As with most water resources activities, there are positive and negative environmental impacts associated with grade control structures. The most serious negative environmental impact commonly associated with grade control structures is obstruction to fish passage. On the positive side, grade control structures can improve the channel stability, improve habitat, and reduce the supply of sediment and nutrients to the channel system. Fish passage issues, as well as other challenges, can be accommodated through appropriate engineering design and by close cooperation with biologists on the planning and design team. Fish passage is described further in NEH654 TS14N.

Grade control hydraulics

There are two basic types of grade control structures. A bed control structure is designed to provide a hard point in the streambed that is capable of resisting the erosive forces of the degradational zone. This is somewhat analogous to locally increasing the size of the bed material. The Lane relation (Lane 1955b) (fig. TS14G–1) illustrates the dynamic relationship, $QS^{+} \propto Q_s D_{50}+$, where the increased slope (S^{+}) of the degrada-

tional reach would be offset by an increase in the bed-material size (D_{50}^{+}) to become stable. Bed armoring controls bed degradation and scour and the increased hydraulic roughness of the bed control structure may dissipate a minor amount of hydraulic energy. A hydraulic control structure is designed to function by reducing the energy slope along the degradational zone to the degree that the stream is no longer competent to scour the bed ($QS^{-} \propto Q_s D_{50}$). The distinction between the operating processes of these two types is important whenever grade control structures are considered.

Energy diagrams (figs. TS14G–2, TS14G–3, and TS14G–4) illustrate the comparison of energy losses that may occur with bed control or hydraulic control grade control structures. Figure TS14G–2 is the preconstruction condition for gradually varied open-channel flow. In figure TS14G–3, a natural stone bed control structure is depicted in the bed between cross sections 2 and 3, reducing the energy gradient due to minor losses occurring with increased roughness. In figure TS14G–4, a hydraulic control structure is depicted in which critical depth for the discharge occurs near the structure crest. A hydraulic drop and a hydraulic jump occur between cross sections 2 and 3. The energy of the downstream reach is reduced by the energy dissipated in the jump, improving downstream stability. Upstream of the drop, the velocity head is reduced, and the pressure head is increased by the raised structure crest.

Because of the complex hydraulic behavior of the flow over grade control structures, it is difficult to designate a single function that applies without exception to each structure. For many situations, the function of a structure as a bed control structure or hydraulic control structure is readily apparent. However, the structure may actually have characteristics of both a bed control and a hydraulic control structure under some conditions. Hydraulic performance or function of the structure can vary with time and discharge. This can occur within a single hydrograph or over a period of years because of upstream or downstream channel changes.

Figure TS14G–1 Lane's balance for water discharge (Q), slope (S), bed-material load (Q_s), and median bed-material size (D_{50})

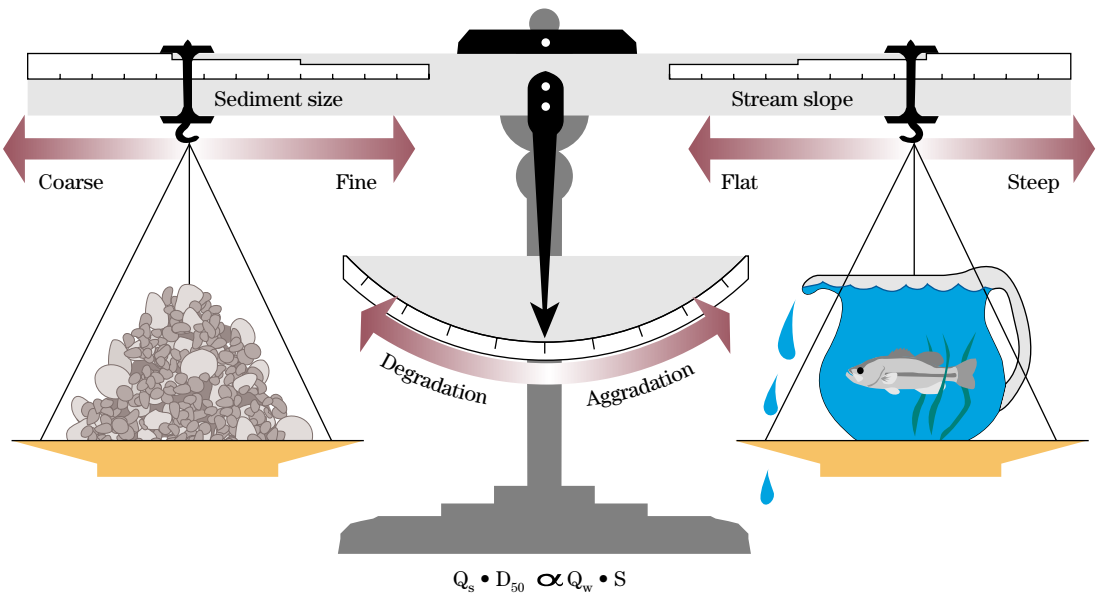


Figure TS14G–2 An energy diagram for the preconstruction condition

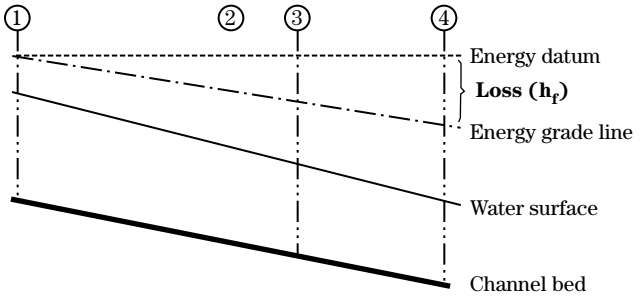


Figure TS14G–3 The modified energy diagram (shown in red) for a bed control structure

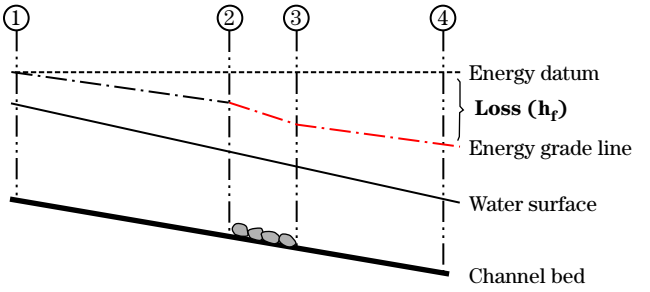
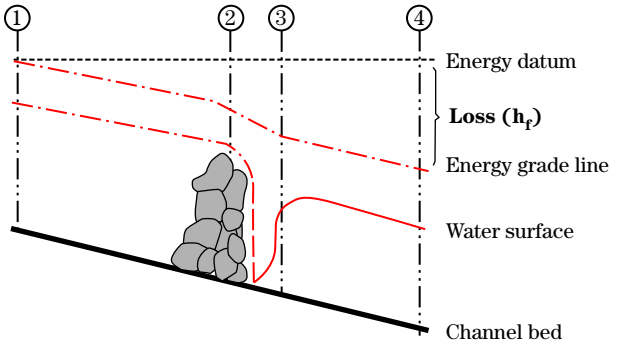


Figure TS14G–4 The modified energy diagram (shown in red) for a hydraulic control structure



Types of grade control structures

Selecting the type of grade control structure is an important general decision, as is the siting and spacing. Certain features are common to most grade control structures including a control section for accomplishing the grade change, an energy dissipation section, and protection of the upstream and downstream approaches. These protected areas often include stone key sections that tie into the banks to protect against flanking. Considerable variations exist in the design of these features. For example, a grade control structure may be constructed of riprap, concrete, sheet piling, treated lumber, logs, soil cement, gabions, compacted earthfill, or other locally available material.

Also, the shape (sloping, stepped, or vertical drop) and dimensions of the structure can vary significantly, as can the various appurtenances (baffle plates, end sills). The applicability of a particular type of structure to any given situation depends on a number of factors such as hydrologic conditions, sediment size and loading, channel morphology, flood plain and valley characteristics, availability of materials, and project objectives, as well as the inevitable time and funding constraints. The successful use of a particular type of structure in one situation does not necessarily ensure that it will be effective in another. Some of the more common types of grade control structures are described in the following sections. Table TS14G–1 provides a brief summary of the advantages and disadvantages of each of these structures. Neilson, Waller, and Kennedy (1991) provide an international literature review on grade control structures, along with an annotated bibliography.

Table TS14G–1 Advantage and disadvantages of selected grade control structures

Structure type	Advantages	Disadvantages
Loose rock structures	Economical to design and build Limited environmental impacts Ease of construction	Generally limited to less than about 3 ft drop heights Potential for displacement of rock due to seepage flows
Channel linings	Provides for energy dissipation through the structure Can be designed to accommodate fish passage	Significant design effort Relatively high cost Larger construction footprint due to length of structure
Loose rock structures with water cutoff	Provides positive water cutoff that eliminates seepage problems and potential for rock displacement Higher drop heights (up to about 6 ft)	More complex design required Higher construction cost than simple loose rock structures More potential for fish obstruction at higher drop heights
Structures with preformed scour holes and water cutoffs	Improved energy dissipation Scour holes provide stable reproductive habitat Higher drop heights (up to about 6 ft)	Larger construction footprint More complex design effort required Increased construction cost More potential for fish obstruction at higher drop height
Rigid drop structures	Can accommodate drop heights greater than 6 ft Provides for energy dissipation Single structure can influence long reach of stream	High construction cost Large construction footprint Significant potential for obstruction to fish Potential for downstream channel degradation due to trapping of sediment
Alternative construction materials	Economically feasible where stone is costly and local labor force is inexpensive and available	Often lack detailed design guidance Increased monitoring and maintenance often required

Loose rock structures

Perhaps the simplest form of a grade control structure consists of placing natural stone or other erosion resistant elements across the channel to form a hard point. Some manufactured concrete products may be used in place of stone. This type of structure includes rock sills, rock sills with impermeable cutoffs, artificial riffles, and sloping rock structures. Various types of loose rock structures are presented herein along with rock sizing procedures and some methods for local scour protection.

Types of loose rock structures

Loose rock structures are generally most effective for drop heights that are less than about 2 to 3 feet. In many applications, a series of loose rock structures are placed relatively close together, effectively providing a greater drop height than a single structure. The series of loose rock structures then provides a degree of conservatism in the design, as one element may reduce stress on the upstream element. Loss of one element may not mean loss of function for the total treatment. The structures must be spaced close enough that channel degradation above one does not undermine the upstream structure. A series of rock sills, each creating a head loss of about 2 feet, was used successfully on the Gering Drain in Nebraska (Stufft 1965). The design concept presented by Whittaker and Jäggi (1986) for stabilizing the streambed with a series of rock sills is shown in figure TS14G–5. These sills are bed control structures that are simply acting as hard points to resist streambed erosion.

Construction of bed sills is sometimes accomplished by placing the rock along the streambed to act as a hard point to resist the erosive forces within the degradational zone. In other situations, a trench may be excavated across the streambed and then filled with rock. A critical component in the design of these structures is ensuring that there is a sufficient volume of erosion resistant material to resist the general bed degradation, as well as any additional local scour at the structure. This is illustrated in figure TS14G–6, which shows a riprap grade control structure designed to resist both the general bed degradation of the approaching nickpoint, as well as any local scour that may be generated at the structure. In this instance, the riprap section must have sufficient mass (layer thickness) to launch into the anticipated scour hole.

A unique type of loose rock structure is used by Newbury and Gaboury (1993). These are often referred to as Newbury riffles. The structures are placed at 5 to 7 channel widths spacing to emulate the spacing of

Figure TS14G–5 Channel stabilization with rock sills

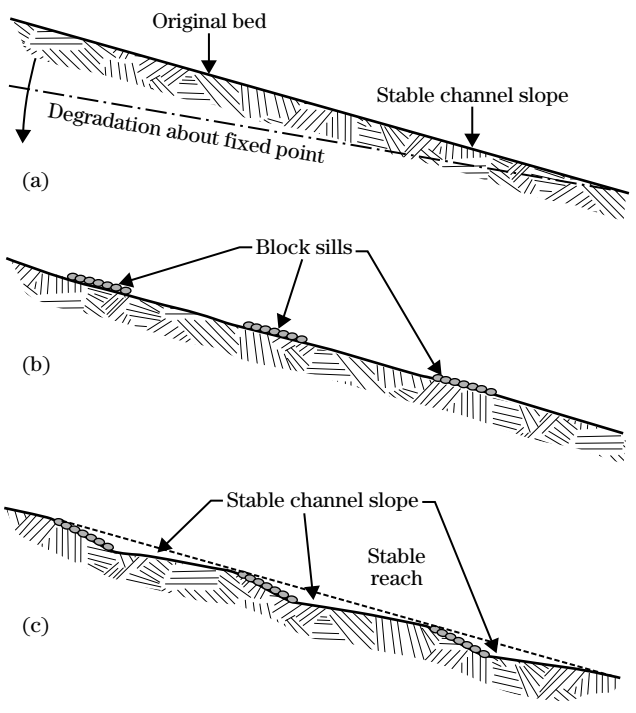
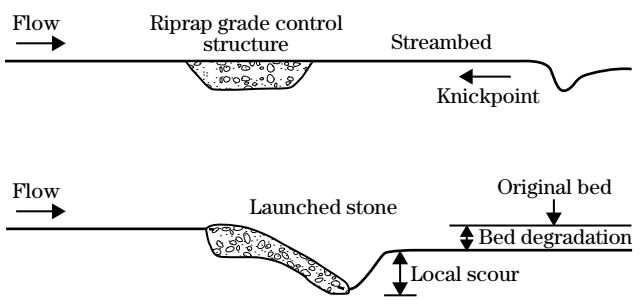


Figure TS14G–6 Top—riprap grade control structure; Bottom—subsequent launching of riprap at the grade control structure in response to advancing bed degradation and local scour

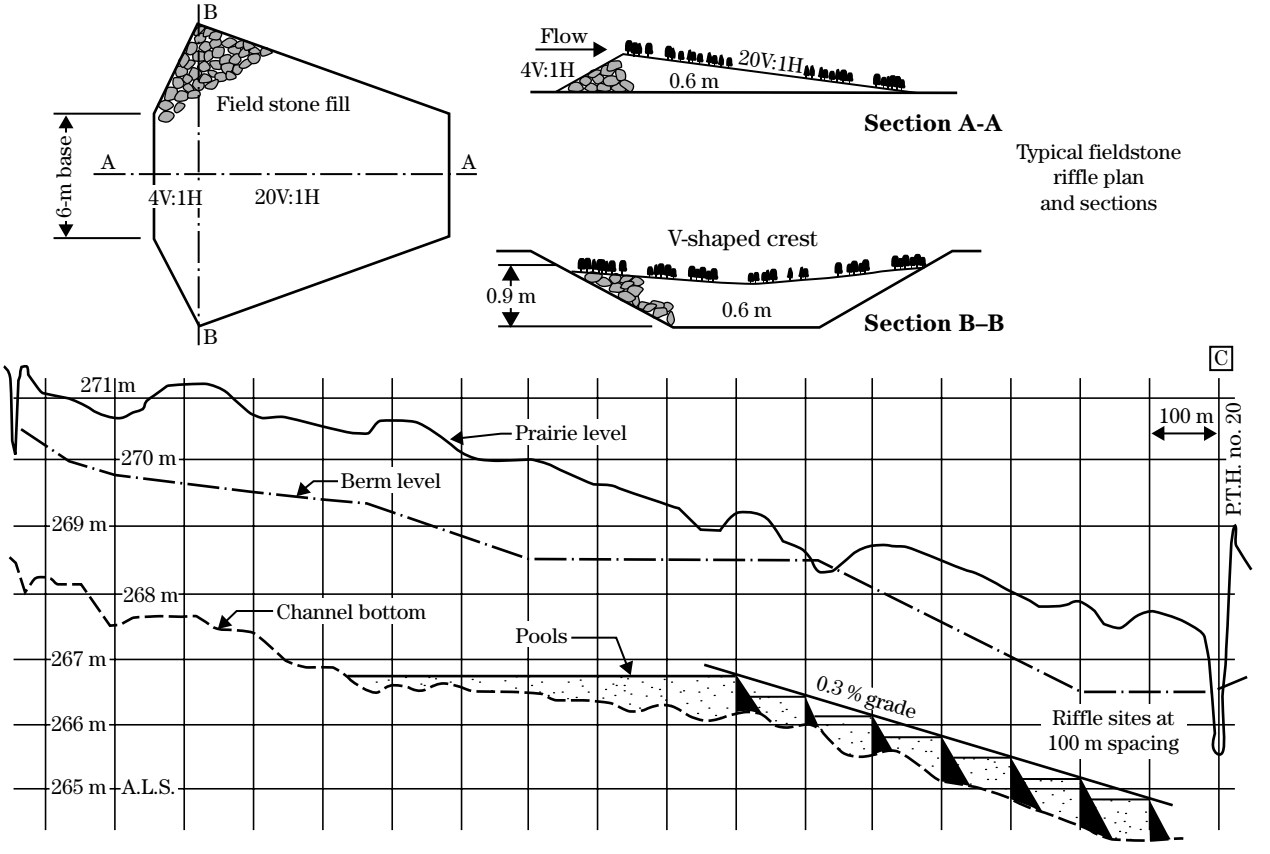


natural riffles. For the Mink Creek example shown in figure TS14G–7 (Newbury 2002), the structures were designed to a height of 0.6 meter that would impound shallow pools for passage of young walleye fry. No cutoff walls or filters were used in this installation, but the structure was sealed by infilling the front slope with shale gravel scraped from the bed.

Rosgen (2001e) describes a cross vane rock structure (fig. TS14G–8) that provides grade control and a pool for fish habitat. Streamflow is shown by the red arrow, and the lowest portion of the structure is located along line A–B, being constructed at the thalweg elevation. As described by Rosgen, no drop in bed elevation exists across the structure, however, a drop in water surface and energy gradient occurs due to lateral constriction. The distance A–B is approximately a third of the stream width, and the structure widens at a 20 degrees to 30-degree angle, expanding to the bankfull width. The vertical angle of the expanding legs is approximately 2 degrees to 7 degrees. The top layer of stones is underlain by footer stones, with the depth of the footer foundation being adjusted to the estimated depth of scour. A pool is excavated within the downstream legs of the structure and may be maintained by the flow turbulence.

A J-hook structure (Rosgen 2001e) is shown in figure TS14G–9. Although primarily developed for bank stabilization, the application shown extends across the low-flow stream and may act as a grade control structure. As shown, the flow is between stones placed near the center of the stream. Notice that both the J-hook and the cross vane rock structures are tied back into the bank to prevent flanking.

Figure TS14G–7 Loose rock structures are shown in plan and profile for Mink Creek, Manitoba, CA



Rock sizing for loose rock structures

A common factor in all loose rock structures is determining the proper stone size. While a more comprehensive description of rock sizing can be found in TS14C, six methods are presented:

Method 1: U.S. Army Corps of Engineers (1994f)

The U.S. Army Corps of Engineers (USACE) developed criteria for sizing steep slope riprap where unit discharges are low and slopes range from 2 to 20 percent. A typical application would be a rock-lined chute. The stone size equation is:

$$D_{30} = \frac{1.95\, S^{0.555} q^{2/3}}{g^{1/3}} \qquad \text{(eq. TS14G–1)}$$

where:
S = bed slope
q = unit discharge

Equation TS14G–1 is applicable to thickness = 1.5 D_{100}, angular rock, unit weight of 167 pounds per cubic foot (lb/ft^3), D_{85}/D_{15} from 1.7 to 2.7, slopes from 2 to 20 percent, and uniform flow on a downslope, with no tailwater. The following steps should be used for this application:

Step 1 Estimate q = Q/b, where b = bottom width of chute.

Step 2 Multiply q by flow concentration factor of 1.25. Use greater factor if approach is skewed.

Step 3 Compute D_{30} using equation TS14G–1.

Step 4 Use uniform gradation having $D_{85}/D_{15} \leq 2$.

Step 5 Restrict application to straight channels with side slopes of 1V:2.5H or flatter.

Method 2: Abt and Johnson (1991)

Abt and Johnson conducted near-prototype flume studies to determine riprap stability when subjected to overtopping flows. Typical uses are for spillway flow or for loose rock grade control structures. Riprap design criteria for overtopping flows were developed for two conditions: stone movement and riprap layer failure. Criteria were developed as a function of stone shape, median stone size, unit discharge, and embankment slope. Stone movement occurred at approximately 74 percent of layer failure. It was determined from testing that rounded stone fails at a unit discharge approximately 40 percent less that angular stone, for

Figure TS14G–8 Cross vane rock grade control structure

Figure TS14G–9 A J-hook grade control structure

the same median size of stone. The resulting equations for angular riprap developed by Abt and Johnson are:

$$q_{design} = \frac{q_f}{0.74} = 1.35 q_f \qquad \text{(eq. TS14G–2)}$$

$$D_{50} = 5.23 S^{0.43} q_{design}^{0.56} \qquad \text{(eq. TS14G–3)}$$

where:
q_f = stone size at failure (in)
q_{design} = design discharge ($ft^3/s/ft$)
S = slope of the riprap layer

Method 3: Whittaker and Jäggi (1986)

$$\frac{q}{\sqrt{g D_{65}^3 (G_S - 1)}} \le \frac{0.257}{J^{\frac{7}{6}}} \qquad \text{(eq. TS14G–4)}$$

where:
q = specific discharge over the ramp ($m^3/s \times m$)
D_{65} = characteristic block diameter of the block mixture (m)
G_S = specific gravity of the blocks compared to that of the water (e.g., 2.65)
J = ramp gradient
g = acceleration due to gravity (m/s^2)

Method 4: Newbury and Gaboury (1993)

tractive force (kg/m^2) = incipient diameter (cm)

(eq. TS14G–5)

Method 5: Robinson, Rice, and Kadavy (1998)
A two-part prediction equation was developed by Robinson, Rice, and Kadavy to determine the highest stable discharge as a function of the median rock size and bed slope. Therefore, knowing any two of the three variables (D_{50} rock size, bed slope, or highest stable discharge) allows calculation of the third. Tests were performed in large flumes and full-size structures with a median rock size up to 11 inches. These large scale rock chutes were tested to failure to develop the following relationships:

$$q = 0.52\, D_{50}^{1.89}\, S_o^{-1.50} \text{ for } S_o < 0.10 \qquad \text{(eq. TS14G–6)}$$

$$q = 4.30\, D_{50}^{1.89}\, S_o^{-1.50} \text{ for } 0.10 < S_o < 0.40$$

(eq. TS14G–7)

where:
q = unit discharge ($ft^3/s/ft$)
S_o = bed slope (ft/ft)
D_{50} = median rock size (ft)

These equations apply to rock chutes constructed with angular riprap with a rock layer thickness of $2D_{50}$. This research was performed on a relatively uniform rock gradation that exhibited a geometric standard deviation ranging from 1.15 to 1.47. These relationships have not been verified for slopes less than 2 percent or greater than 40 percent.

Method 6: Rosgen (2001e)
The Rosgen relationship was developed to determine minimum size of rock for the cross vane and J-hook structures at bankfull flow conditions:

minimum rock size (m) =
0.1724 ln (bankfull shear stress, kg/m^2) + 0.6349

(eq. TS14G–8)

Application of this relationship is limited to river discharges ranging from 0.56 cubic meters per second to 113.3 cubic meters per second, and bankfull depth from 0.26 meter to 1.5 meters.

Rock sizing summary

Figure TS14G–10 compares the six different procedures using a 5 percent sloping (1V:20H) loose rock structure at a unit discharge varying from 1 to 10 cubic meters per meter of width. It should be noted that the D_n varied between the methods, so an absolute comparison was not possible. For instance, Chervet and Weiss (1990) specified D_{65}, Abt and Johnson (1991) and Robinson, Kadvey, and Rice (1998) specified D_{50}, USACE (1994a) specified D_{30}, Newbury and Gaboury (1993) did not specify a rock size within the gradation, and the Rosgen (2002) method calculates the minimum rock size. However, comparison of the curves in figure TS14G–10 indicates that, with the exception of the Rosgen method, there is general consistency among the other five methods. It is important to note that the Rosgen (2002) relationship determines the minimum size of rock required and unlike the other methods, does not calculate a stone gradation. Therefore, it is not surprising that Rosgen's results are not compatible

with the other methods. If the sloping loose rock structures are to be constructed in a location that will encounter completely submerged conditions, traditional riprap-sizing methods (USACE 1994f; U.S. Department of Transportation Federal Highway Administration (FHWA) 2001a) should be used to check structure stability. An example design procedure for a sloping loose rock drop structure, adapted from Watson and Eom (2003), is provided at the end of this technical supplement.

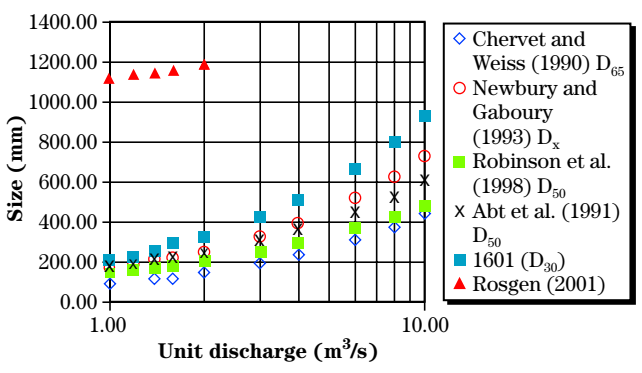
Local scour protection for loose rock structures

Chervet and Weiss (1990) reviewed work by Whittaker and Jäggi (1986) and developed a relationship for predicting local scour at the downstream extent of a loose rock structure, referred to by the authors as a block ramp.

The maximum scour depth (t) can be estimated using the following approach (Tschopp-Bisaz, modified in accordance with Whittaker and Jäggi (1986)):

$$t + h_U \cong 0.85q^{0.5}\left(\frac{q}{h_N}\right)^{0.5} - 7.125D_{90} \qquad \text{(eq. TS14G–9)}$$

Figure TS14G–10 Comparison of rock sizing methods for a 1V:20H sloping face structure



where:

h_U = tailwater depth (m);

h_N = normal supercritical discharge depth over the ramp (m), e.g., calculated according to Strickler's formula, using a coefficient of friction of $k = 21/D_{65}^{1/6}$ ($m^{1/3}/s$)

t = predicted scour depth (m)

Local scour depth is directly related to unit discharge, and an inverse relationship is shown for tailwater depth and the D_{90} of the bed material. Chervet and Weiss (1990) recommend that the downstream extent of the structure should extend below an anticipated local scour depth.

Bitner (2003) reviewed local scour depth, reporting that Castro (1999) defined bed key depth as the local scour depth to which the rock structure should be excavated to prevent undermining. Castro recommended that the scour depth may approach 2.5 times the drop height for gravel or cobble beds, and 3.5 times the drop height for sand beds.

Channel linings

Grade control can also be accomplished by lining the streambed with an erosion resistant material. These structures are designed to ensure that the drop is accomplished over a specified stream reach that has been lined with riprap or some other erosion-resistant material. Rock riprap gradient control structures have been used by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) (formally the Soil Conservation Service (SCS) 1976) for several years. These structures are designed to flow in the subcritical regime with a constant specific energy at the design discharge, which is equal to the specific energy of flow immediately upstream of the structure (Myers 1982). Although these structures have generally been successful, some have had local scour problems. This precipitated a series of model studies to correct these problems and to develop a design methodology for these structures (Tate 1988, 1991). Plan and profile drawings of the improved structure are shown in figure TS14G–11 (adapted from Tate 1991).

Loose rock structures with water cutoff

One problem often encountered with channel lining structures is the displacement of rock (or rubble) due to the seepage flow around and beneath the structure. This is particularly a problem when the bed of the stream is composed primarily of pervious material. This problem can be eliminated by constructing a water barrier at the structure. One type of water barrier consists of simply placing a trench of impervious clay fill upstream of the weir crest (fig. TS14G–12). In general, this type of barrier has limited longevity due to susceptibility to erosion. This erosion can be avoided by using a concrete or sheet pile cutoff wall. The conceptual design of a riprap grade control structure with a sheet pile cutoff wall is shown in figure TS14G–13.

Structures with preformed scour holes and water cutoff

A scour hole is a natural occurrence downstream of any overfall. Sizing of the scour hole is a critical element in the design process, which is usually based on model studies or on experience with similar structures in the area.

The stability of rock structures is often jeopardized at low tailwater conditions. One way to ensure the stability of the rock is to design the structure to operate in a submerged condition. Linder (1963) developed a structure that is designed to operate at submerged conditions where the tailwater elevation (T) does not fall below 0.8 of the critical depth (d_c) at the crest section. Subsequent monitoring of the in-place structures confirmed the successful performance in the field (USACE 1981).

Little and Murphey (1982) developed a loose rock structure incorporating a sheet pile cutoff and weir, and a preformed scour basin lined with riprap that acts as an energy dissipation basin. They observed that an undular hydraulic jump occurs when the incoming Froude number is less than 1.7. Consequently, Little and Murphey developed a grade control design that included an energy dissipating baffle to break up these undular waves (fig. TS14G–14). This structure, referred to as the Agricultural Research Service (ARS)-type low-drop structure, has been used successfully in northern Mississippi for drop heights up to about 2 meters by both the USACE and the U.S. Department of Agriculture (USDA) SCS (USACE 1981). A recent modification to the ARS structure was developed following model studies at Colorado State University (Johns et al. 1993; Abt et al. 1994). The modified ARS structure, presented in figure TS14G–15, retains the baffle plate, but adopts a vertical drop at the sheet pile, rather than a sloping rockfill section as recommended by Little and Murphey.

Smith and Wilson (1992) provide guidance for design and construction of the ARS-type grade control structure. The guidance is replete with information, and several specific points follow:

- For selection of the final structure site, the stream should be straight for a distance of 10 stream widths upstream and for a minimum of 200 feet downstream.
- No gullies or lateral drains should occur in the site.
- The base width of the weir should be constricted to ensure that the water surface elevation of the 2-year discharge moves from critical depth near the weir crest to normal depth of flow in a short distance; for example, a few stream widths.
- The resulting flood-control impacts should not violate flood-control requirements.

Figure TS14G–11 Rock riprap gradient control structure

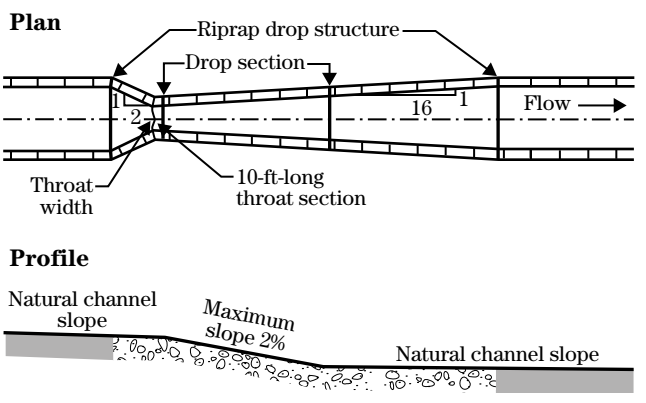


Figure TS14G–12 Top—built riprap grade control structure with an impervious fill cutoff wall; Bottom—launching of riprap at the grade control structure in response to bed degradation and local scour

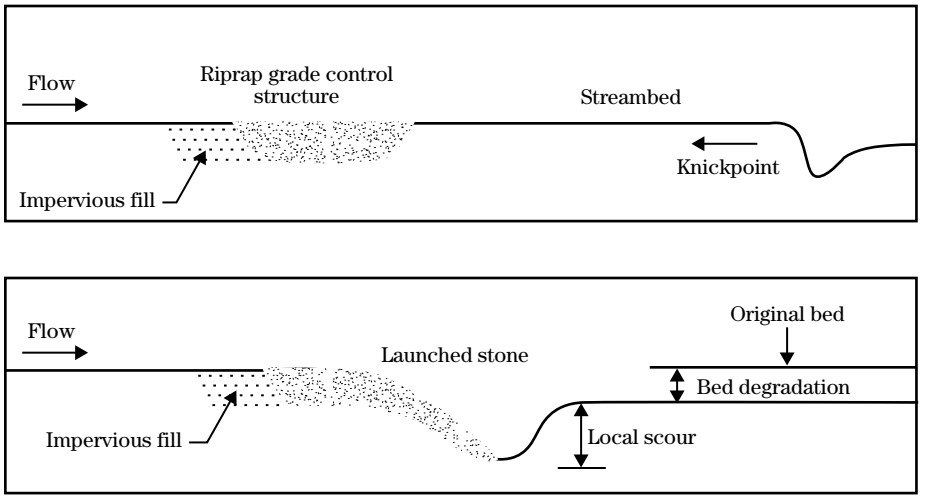


Figure TS14G–13 Top—built riprap grade control structure with a sheet pile cutoff wall (*top*); Bottom—launching of riprap at the grade control structure in response to bed degradation and local scour

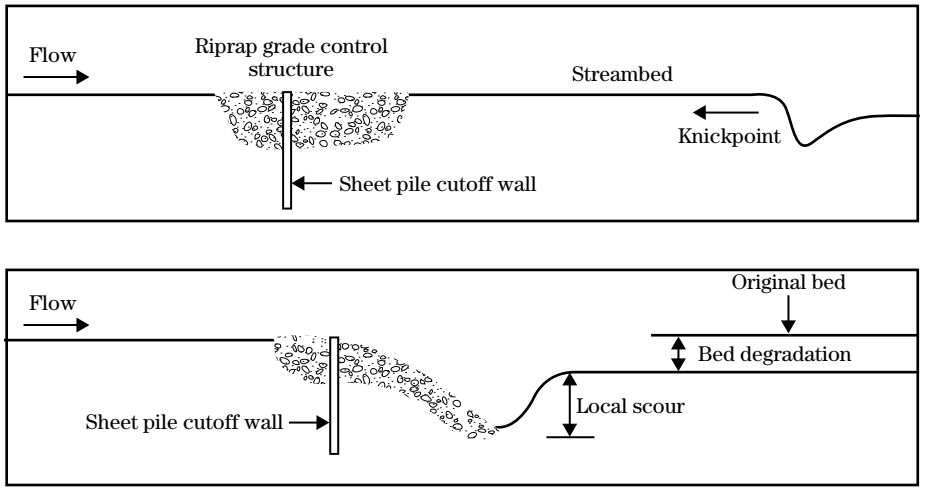
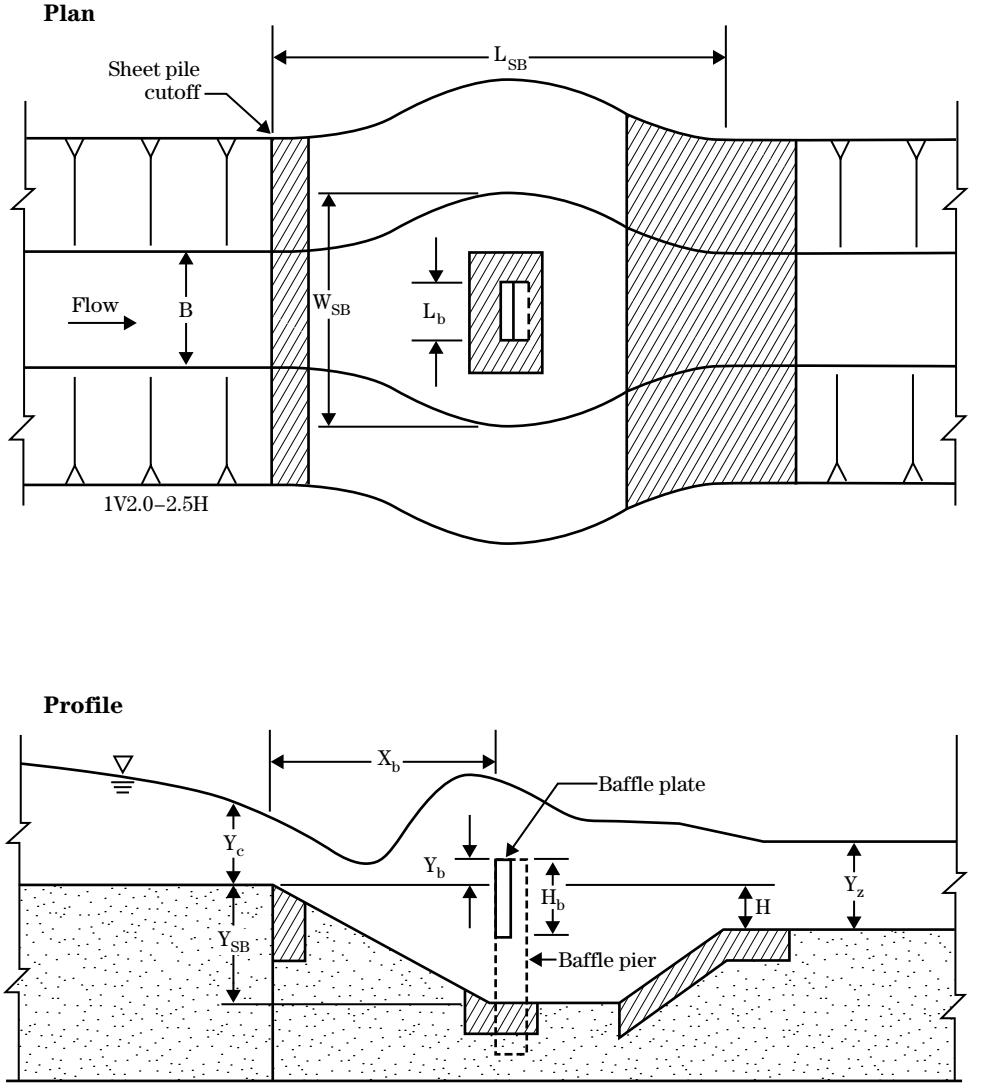


Figure TS14G–14 ARS-type grade control structure with preformed riprap-lined stilling basin and baffle plate



- The stilling basin dimensions should be based on the smaller of the bankfull discharge or the 100-year discharge.
- Downstream tailwater conditions should be based on normal depth calculations of an estimated future, degraded condition.
- Stilling basin riprap size is based on physical model studies referenced in the guidance. Approach stream protection and exit stream protection are specified.

Recent modifications to the ARS-type grade control structure by the USACE Vicksburg District replaced the vertical face downstream of the weir with a 1V:2H sloping face constructed of grouted riprap (fig. TS14G–16). Upstream riprap extends below the water; however, sediment filling of the stone as shown is supporting vegetation Other modifications included elimination of the baffle plate and the construction of an impervious fill section at the weir section in lieu of the sheet pile cutoff wall. Annual monitoring of these structures since the early 1990s has revealed no significant negative structural or channel impacts.

Rigid drop structures

In many situations where the discharge and/or drop heights are large, in excess of 2 meters, grade control structures are frequently constructed of concrete or a combination of sheet pile and concrete. There are many different designs for concrete grade control structures. Two described here are the California Institute of Technology (CIT) and the St. Anthony Falls (SAF) structures. Both of these structures were used on the Gering Drain project in Nebraska, where the decision to use one or the other was based on the flow and stream conditions (Stufft 1965). Where the discharges were large and the stream depth was relatively shallow, the CIT-type drop structure was used. The CIT-type structure is generally applicable to low-drop situations where the ratio of the drop height to critical depth is less than 1; however, for the Gering Drain project this ratio was extended up to 1.2. The original design of this structure was based on criteria developed by Vanoni and Pollack (1959). The structure was then modified by model studies at the USACE Waterways Experiment Station (WES) in Vicksburg, Missis-

Figure TS14G–15 Schematic of modified ARS-type grade control structure

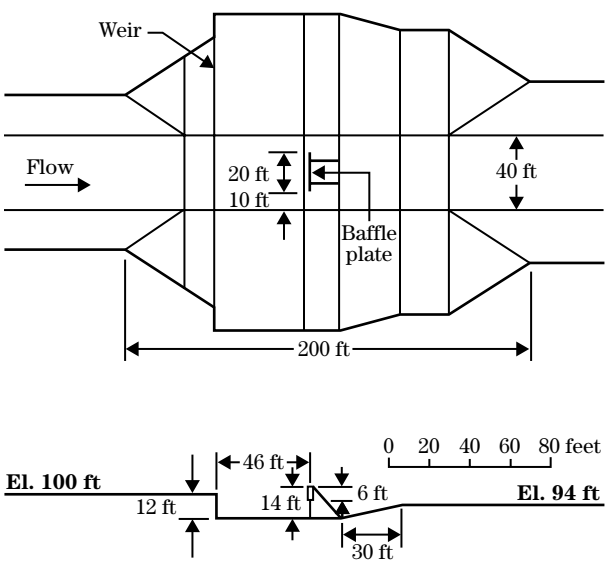


Figure TS14G–16 ARS-type grade control structure with grouted riprap face

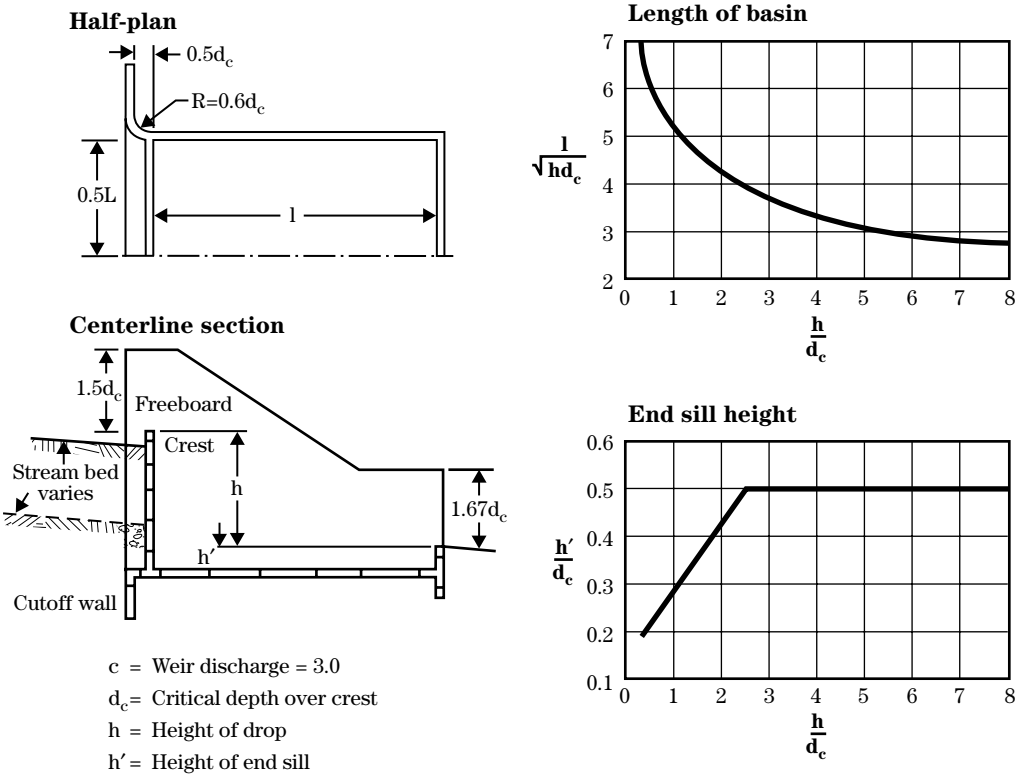
sippi, and is shown in figure TS14G–17 (Murphy 1967). Where the stream was relatively deep and the discharges smaller, the SAF drop structure was used. This design was developed from model studies at the SAF Hydraulic Laboratory for the SCS (Blaisdell 1948). This structure is shown in figure TS14G–18. The SAF structure is capable of functioning in flow conditions where the drop height to critical depth ratio is greater than 1 and can provide effective energy dissipation within a Froude number range of 1.7 to 17. Both the CIT and the SAF drop structures have performed satisfactorily on the Gering Drain for more than 25 years.

The design for a large, rigid structure should include consideration of slope stability including sudden drawdown. Slope stability should also be investigated for the site, approach, and downstream channels. Stability analyses should include sliding stability of the structure, underseepage, and allowance for bearing capacity and settlement. As the hydraulic capacity and drop height of the structure increases, the complexity of design and construction increases.

Alternative construction materials

While riprap, sheet pile, and concrete may be the most commonly used construction materials for grade control structures, cost or availability of materials may prompt the engineer to consider other alternatives.

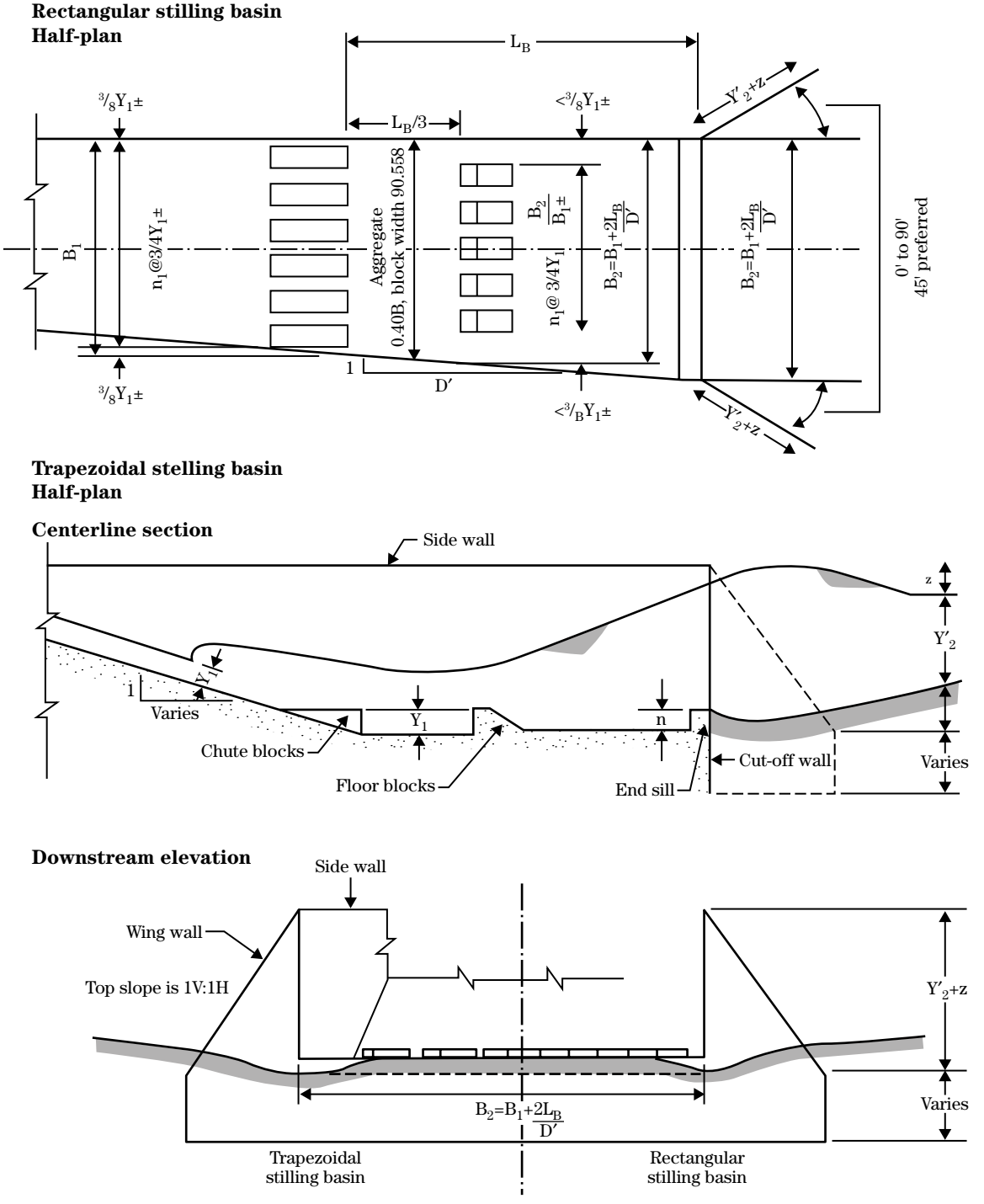
Figure TS14G–17 CIT-type drop structure



c = Weir discharge = 3.0
d_c = Critical depth over crest
h = Height of drop
h′ = Height of end sill
H = Head on weir = 3/2(d_c)
l = Length of basin
L = Length of weir crest
Q = Discharge, $CLH^{3/2}$

Details and design chart for typical drop structure

Figure TS14G–18 SAF drop structure



Gabion grade control structures are often an effective alternative to standard riprap or concrete structures (Hanson, Lohnes, and Klaiber 1986). Guidance for the construction of gabion weirs is also provided by the USACE (1974). Gabion mattresses consist of rectangular-shaped wire-mesh baskets filled with rock (FHWA 1989). Current applications of gabion mattresses include streambed and bank stabilization. Further information on small grade control is provided in NEH654 TS14P; and the use of gabions for bank stabilization is described in NEH654 TS14K. Table TS14G–2 (adapted from FHWA (1989)) presents the advantages and disadvantages of gabion mattresses when used in an erosion control application. Other, more detailed design guidelines for rock gabions can be found in FHWA (1989), USACE (1974), and Maynord (1995).

Bitner (2003) pointed out that an alternative to the conventional riprap or concrete structure that has gained popularity in the Southwestern United States is the use of soil cement grade control structures. These structures are constructed of onsite soil-sand in a mix with Portland Cement to form a high quality, erosion-resistant mixture. Soil cement grade control structures are most applicable when used as a series of small drops, in lieu of a single large-drop structure. Experience indicates that a limiting drop height for these structures is on the order of 1 meter. Design criteria for these structures are presented by Simons and Li (1982).

Thornton et al. (1999) have developed shear resistance criteria for A-Jacks®, an interlocking concrete armor unit manufactured by Armortec Erosion Control Solutions. Current applications of A-Jacks® include coastal shoreline protection, streambed and bank protection, and pier scour mitigation. Depending on their intended application, A-Jacks® vary between 2 to 8 feet in size.

Stone riprap can be bound with cement grout, forming grouted riprap. The apparent advantage in grouted riprap is to increase the shear resistance of individual stone particles. In their review of grouted riprap, Przedwojski, Blazejewski, and Pilarczyk (1995) cited three basic methods of grouting (Rÿkswaterstaat 1995):

- Surface grouting fills approximately 30 percent of the surface voids, with mortar penetrating the surface layer without completely sealing the construction.
- Pattern grouting fills 50 percent to 80 percent of cover-layer voids and penetrates the full thickness of the riprap. Eventually, a mesh of stone-cement aggregates is formed.
- Full grouting fills 100 percent of the cover-layer voids, resulting in an impermeable layer.

They caution that as voids are filled with grout and permeability diminishes, the stability of the layer is adversely affected by excess pore pressures occurring during high discharges or from ground water. Weep holes or other positive drainage should be provided to avoid massive failure. Grouted riprap is addressed further in NEH654 TS14K.

McLaughlin Water Engineers (1986) report that grout has been successfully used to stabilize loose riprap. Many failures have been reported that were associated with seepage and uplift. They recommend that seepage be controlled by constructing a vertical cutoff immediately upstream of the crest, constructing the cutoff by excavating a trench below the riprap subgrade, and placing steel and concrete to form the cutoff wall. Their view of grouted riprap is different from Przedwojski, Blazejewski, and Pilarczyk (1995). McLaughlin Water Engineers recommend that regular

Table TS14G–2 Advantage and disadvantages of gabion mattresses when used in an erosion control application

Advantages	Disadvantages
• Ability to span minor pockets of subsidence without failure • Interlock to allow use of smaller, lower quality rock in the baskets • Economically feasible where riprap-sized rock is not readily available	• Susceptibility of the wire baskets to corrosion, abrasion damage, and vandalism • High labor cost associated with fabrication and filling the baskets • More difficult and expensive repair than standard rock protection

riprap should not be used with grout and that rock with all dimensions greater than the grout thickness be required and placed on a firm subgrade. Grout is then pumped into the voids and vibrated, filling the voids between rocks. The method results in the appearance of a concrete slab with large stones spaced evenly, protruding through the slab. Toe and lateral drains are included for drainage of the grouted area.

General design considerations for siting grade control structures

Design considerations for siting grade control structures include determination of the type, location, and spacing of structures, along with the elevation and dimensions of the structures. Siting grade control structures is often considered a simple optimization of hydraulics and economics. However, these factors alone are usually not sufficient to define the optimum grade control siting conditions. In practice, hydraulic considerations must be integrated with a host of other factors that vary from site to site to determine the final structure plan. Some of the more important factors to be considered when siting grade control structures are described in the following sections. This does not represent an all-inclusive list, since there may be other factors that may be locally important. For example, maintenance requirements, debris passage, ice conditions, or safety considerations may be controlling factors. Consequently, there is no definitive procedure for siting grade control structures. However, consideration of each factor in an analytical and balanced fashion can lead to a more effective design process that will ensure that the plan accomplishes the long-term project goals.

Hydraulic and sediment transport considerations

One of the most important steps in the siting of a grade control structure or a series of structures is the drop height determination. This requires some knowledge of the ultimate stream morphology, both upstream and downstream of the structure, which involves assessment of sediment transport and stream morphologic processes.

The hydraulic siting of grade control structures is a critical element of the design process, particularly when a series of structures is planned. The design of each structure is based on the anticipated tailwater or downstream bed elevation, which in turn, is a function of the next structure downstream. Heede and Mulich (1973) suggested optimum spacing of structures, so that the upstream structure does not interfere with the deposition zone of the next downstream structure. Mussetter (1982) showed that the optimum spacing should be the length of the deposition above the structure, which is a function of the deposition slope (fig. TS14G–19 (adapted from Mussetter). Figure TS14G–19 also illustrates the recommendations of Johnson and Minaker (1944), that the most desirable spacing can be determined by extending a line from the top of the first structure, at a slope equal to the maximum equilibrium slope of sediment upstream, until it intersects the original streambed.

Theoretically, the hydraulic siting of grade control structures is straightforward and can be determined by:

$$H = (S_o - S_f)X \qquad \text{(eq. TS14G–10)}$$

where:
H = amount of drop to be removed from the reach
S_o = original bed slope
S_f = final, or equilibrium slope
X = length of the reach (Goitom and Zeller 1989)

The number of structures (N) required for a given reach can then be determined by:

$$N = \frac{H}{h} \qquad \text{(eq. TS14G–11)}$$

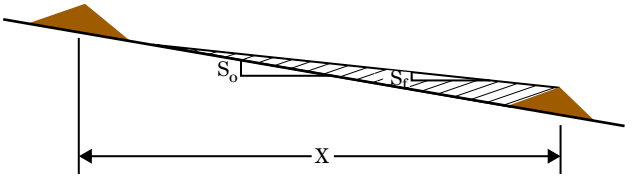
where:
h = selected drop height of the structure

It follows from equation TS14G–10 and figure TS14G–19 that one of the most important factors to consider when siting grade control structures is the determination of the equilibrium slope (S_f). Unfortunately, this is also one of the most difficult parameters to define with any reliability. Equilibrium slope is defined as the channel slope that is required to transport the bed material supplied through the reach, without significant aggradation or degradation of the channel.

With respect to grade control design, this is the slope that is anticipated to develop through time, upstream of the structure. Failure to properly define the equilibrium slope can lead to costly, overly conservative designs, or an inadequate design, resulting in continued maintenance problems and a possible structure failure. The primary factors affecting the final equilibrium slope upstream of a structure include the incoming sediment concentration and load, the channel characteristics (slope, width, depth, roughness), and the hydraulic effect of the structure. Another complicating factor is the amount of time it takes for the equilibrium slope to develop. In some instances, the equilibrium slope may develop over a period of a few hydrographs, while in others, it may take many years.

Many different methods exist for determining the equilibrium slope in a channel (Mussetter 1982; FISRWG 1998; Watson and Biedenharn 1999). These can range from detailed sediment transport modeling (Thomas, Copeland, et al. 1994; USACE 1993c) to less elaborate procedures involving empirical or process-based relationships, such as regime analysis (Lacey 1931; Simons and Albertson 1963), tractive stress (Lane 1953a, b; Simons 1957; Simons and Sentürk 1992; USACE 1994a), or minimum permissible velocity (USDA SCS 1977). In some cases, the equilibrium slope may be based solely on field experience with similar channels in the area. Regardless of the procedure used, the engineer must recognize the uses and limitations of that procedure before applying it to a specific situation. The decision to use one method or another depends on several factors such as the level of study (reconnaissance or detail design), availability and reliability of data, project objectives, and time and cost constraints. Equilibrium is addressed further in NEH654.13.

Figure TS14G–19 Spacing of grade control structure



Geotechnical considerations

The previous description focused only on the hydraulic aspects of design and siting of grade control structures. In some cases, the geotechnical stability of the reach may be an important or even the primary factor to consider when siting grade control structures. This is often the case where stream degradation has caused, or is anticipated to cause, severe bank instability due to exceedance of the critical bank height (Thorne and Osman 1988). When this occurs, bank instability may be widespread throughout the system, rather than restricted to the concave banks in bendways. Traditional bank stabilization measures may not be feasible where systemwide bank instabilities exist. In these instances, grade control, aimed at preventing the onset of incision-triggered mass wasting, may be the more appropriate solution.

Grade control structures can enhance the bank stability of a stream in several ways. Bed control structures indirectly affect the bank stability by stabilizing the bed, thereby reducing the length of bankline that achieves an unstable height. With hydraulic control structures, two additional bank stability advantages are that bank heights can be reduced due to sediment deposition upstream of the structure, increasing bank stability, and by creating backwater conditions, velocities and scouring potential are reduced, which can minimize or eliminate the severity and extent of basal clean out of the failed bank material, thereby promoting self-healing of the banks (Thorne 1990). Therefore, if systemwide bank instability is a significant concern, consideration might be given to raising and/or constricting the weir invert to promote bank stability.

Additional references pertaining to streambank stability include the American Society of Civil Engineers (ASCE 1998); Bishop (1955); Coppin and Richards (1990); Gray and Leiser (1982); Hagerty (1991); Huang (1983); Kouwen, Unny, and Hill (1969); López and Garcia (1997); Morgenstern and Price (1965); Osman and Thorne (1988); Sands and Kapitzke (1998); Simon, Wolfe, and Molinas (1991); Simon et al. (1999); Terzaghi (1943); and Terzaghi and Peck (1967). In addition, geotechnical issues are described in NEH654 TS14A.

The flow of water through a pervious foundation can be a serious problem for a grade control structure. As the drop height of the structure increases, the driv-

ing force increases for subsurface flow and possible erosion beneath the structure. Very silty and sandy soils are the least resistant to seepage or piping failures (McLaughlin Water Engineers 1986). Seepage pressures and velocities must be controlled to prevent internal erosion and particle migration. In extreme cases, seepage may cause failure of the structure foundation and sloughing of the streambank downstream of the crest of the structure. Seepage theory and analysis is addressed in Cedergren (1977), and embankment flownets are addressed in depth by Sherard et al. (1963) and Volpe and Kelly (1985), as referenced in Novak et al. (1997).

Common methods of seepage control include cutoff trenches filled with an impervious material, sheet pile curtains, upstream impervious blankets, and downstream filter blankets. The U.S. Department of Interior, Bureau of Reclamation (1987) provides an intensive description of these methods. Sheet pile is addressed further in NEH654 TS14R, and geosynthetics is addressed in NEH654 TS14D.

Flood control impacts

Stream improvements for flood control and stream stability often appear to be mutually exclusive objectives. For this reason, it is important to ensure that any increased postproject flood potential is identified. This is particularly important when hydraulic control structures are considered; the potential for causing overbank flooding may be the limiting factor with respect to the height and amount of constriction at the structure. Grade control structures are often designed to be hydraulically submerged at flows less than bankfull so that the frequency of overbank flooding is not affected. However, if the structure exerts control through a wider range of flows, including overbank, the frequency and duration of overbank flows may be impacted. When this occurs, the impacts must be quantified and appropriate provisions should be implemented such as acquiring flow easements or modifying structure plans.

Another factor that must be considered when designing grade control structures is the safe return of overbank flows back into the stream. This is particularly a problem when the flows are out of the bank upstream of the structure, but still within the bank downstream. The resulting head differential can cause damage to the structure, as well as severe erosion of the streambanks, depending on where the flow reenters the stream. Some means of controlling the overbank return flows must be incorporated into the structure design. One method is simply to design the structure to be submerged below the top bank elevation, thereby reducing the potential for a head differential to develop across the structure during overbank flows. If the structure will impact overbank flows, a more direct means of controlling the overbank return flows must be provided. One method is to ensure that all flows pass only through the structure. This may be accomplished by building an earthen dike or berm extending from the structure to the valley walls that prevents any overbank flows from passing around the structure (Forsythe 1985). Another means of controlling overbank flows is to provide an auxiliary high-flow structure, which will pass the overbank flows to a specified downstream location, where the flows can reenter the stream without causing significant damage (Hite and Pickering 1982).

Environmental considerations

Projects must work in harmony with the natural system to meet the current needs without compromising the ability of future generations to meet their needs. Engineers and geomorphologists are responding to this challenge by developing new and innovative methods for incorporating environmental features into stream projects. The final siting of a grade control structure is often modified to minimize adverse environmental impacts to the system.

Grade control structures can provide direct environmental benefits to a stream. Cooper and Knight (1987) conducted a study of fisheries resources below natural scour holes and manmade pools below grade control structures in northern Mississippi. They concluded that although greater species diversity occurred in the natural pools, increased growth of game fish and a larger percentage of harvestable size fish were recorded in the manmade pools. They also observed that the manmade pools provided greater stability of reproductive habitat. Shields, Hoover, et al. (1990) reported that the physical aquatic habitat diversity was higher in stabilized reaches of Twentymile Creek,

Mississippi, than in reaches without grade control structures. They attributed the higher diversity values to the scour holes and low-flow channels created by the grade control structures. The use of grade control structures as environmental features is not limited to the low-gradient sand-bed streams of the Southeastern United States. Jackson (1974) documented the use of gabion grade control structures to stabilize a high-gradient trout stream in New York. Jackson observed that following construction of a series of bed sills, trout density increased significantly. The increase in trout density was attributed to the accumulation of gravel between the sills, which improved the spawning habitat for various trout species.

Perhaps the most serious negative environmental impact of grade control structures is the possible obstruction to fish passage. In some cases, particularly when drop heights are small, fish are able to migrate upstream past a structure during high flows (Cooper and Knight 1987). However, as drop heights increase, the structures may restrict or completely block passage of some or all fish and other aquatic organisms, based on their individual species' abilities to jump over or swim through impediments. Therefore, fish passage may be a primary consideration in the selection of structure types and drop heights. For instance, it may be necessary to provide for fish passage to select a series of sloping riprap structures with small drops, in lieu of a single high-drop structure. However, if other factors dictate that a high-drop structure is required, the structure may need to be modified to provide for fish ladders or other passageways (Nunnally and Shields 1985). Various methods of accomplishing fish movement through structures are addressed in NEH654 TS14J. Interested readers are also referred to Nunnally and Shields (1985), Clay (1961), and Smith (1985) for more detailed information.

The environmental aspects of the project must be an integral component of the design process when siting grade control structures. A detailed study of all environmental features in the project area should be conducted early in the design process. This will allow these factors to be incorporated into the initial plan, rather than having to make costly and often less environmentally effective last-minute modifications to the final design. Unfortunately, very little guidance is published concerning the incorporation of environmental features into the design of grade control structures. A source of useful information is found in the following technical reports published by the USACE Environmental Laboratory, WES: Shields and Palermo (1982), Henderson and Shields (1984), and Nunnally and Shields (1985).

Existing structures

Bed degradation can cause significant damage to bridges, culverts, pipelines, utility lines, and other structures along the channel perimeter. Grade control structures can prevent this degradation, thereby providing protection to these structures. For this reason, it is important to locate all potentially impacted structures when siting grade control structures. The final siting should be modified, as needed, within project constraints, to ensure protection of existing structures.

Grade control structures can have adverse, as well as beneficial, effects on existing structures. This may be a concern upstream of hydraulic control structures due to the potential for increased flood stages and sediment deposition. The possibility of submerging upstream structures, such as water intakes or drainage structures, may become a deciding factor in the siting of grade control structures.

Whenever possible, the designer should take advantage of any existing structures that may already be providing some measure of grade control. This usually involves culverts or other structures that provide an erosion-resistant surface across the streambed. Unfortunately, these structures are usually not initially designed to accommodate any significant bed lowering and, therefore, cannot be relied on to provide long-term grade control. However, it may be possible to modify these structures to protect against the anticipated degradation. These modifications may be accomplished by simply adding some additional riprap with launching capability at the downstream end of the structure. In other situations, more elaborate modifications, such as providing a sheet-pile cutoff wall or energy dissipation devices, may be required. Damage to and failure of bridges is the natural consequence of channel degradation. Consequently, it is not uncommon in a channel stabilization project to identify several bridges that are in need of repair or replacement.

Therefore, it is often advantageous to integrate the grade control structure into the planned improvements at the bridge. If the bridge is not in immediate danger of failing and only needs some additional erosion protection, the grade control structure can be built at or immediately downstream of the bridge, with the riprap from the structure tied into the bridge for protection. If the bridge is to be replaced, it may be possible to construct the grade control structure concurrently with the new road crossing.

Local site conditions

When planning grade control structures, the final siting is often adjusted to accommodate local site conditions such as the planform of the stream or local drainage. A stable upstream alignment that provides a straight approach into the structure is critical. Since failure to stabilize the upstream approach may lead to excessive scour and possible flanking of the structure, it is desirable to locate the structure in a straight reach. If this is not possible (as in a very sinuous channel), it may be necessary to realign the channel to provide an adequate approach. Stabilization of the realigned channel may be required to ensure that the approach is maintained. Even if the structure is built in a straight reach, the possibility of upstream meanders migrating into the structure must be considered. In this case, the upstream meanders should be stabilized prior to or concurrent with, the construction of the grade control structure.

Local inflows from tributaries, field drains, roadside ditches, or other sources often affect the siting of grade control structures. Failure to provide protection from local drainage can result in severe damage to a structure (USACE 1981). During the initial siting of the structure, all local drainage should be identified. Ideally, the structure should be located to avoid local drainage problems. However, there may be some situations where this is not possible. The local drainage should either be redirected away from the structure or incorporated into the structure design.

Downstream channel response

Since grade control structures affect the sediment delivery to downstream reaches, it is necessary to consider the potential impacts to the downstream channel when grade control structures are planned. Bed control structures reduce the downstream sediment loading by preventing the erosion of the bed and banks, while hydraulic control structures have the added effect of trapping sediments. The ultimate response of the channel to the reduction in sediment supply varies from site to site. The effects of grade control structures on sediment loading may be so small that downstream degradational problems may not be encountered. However, when a series of hydraulic control structures is planned, the cumulative effects of sediment trapping may become significant. It may be necessary to modify the plan to reduce the amount of trapped sediment or consider placing additional grade control structures in the downstream reach to protect against the induced degradation. If downstream sediment problems are anticipated, a sediment budget analysis should be performed to ensure that the grade control structures will not create channel instability.

Geologic controls

Geologic controls often provide grade control in a similar manner to a bed control structure. A grade control structure can actually be eliminated from the plan if existing geologic control can be used to provide a similar level of bed stability. Caution must always be used when relying on geologic outcrops to provide long-term grade control. Where geologic controls are to be used as permanent grade control structures, a detailed geotechnical investigation of the outcrop is needed to determine its vertical and lateral extent. This is necessary to ensure that the outcrop will neither be eroded, undermined, nor flanked during the project life.

Effects on tributaries

When siting grade control structures, the effects of main stem structures on tributaries should be considered. As degradation on a main stem channel migrates

upstream, it may branch up into the tributaries. If possible, main stem structures should be placed downstream of tributary confluences. This will allow one structure to provide grade control to both the main stem and the tributary. This is generally a more cost-effective procedure than having separate structures on each channel.

Grade control siting summary

The selection of the location, type, and number of grade control structures is the most important aspect of grade control design. As illustrated in this technical supplement, a wide range of grade control designs can be used to satisfy the hydraulic and sediment transport requirements of the stream, and the selection of the appropriate one will generally reflect the consideration of a number of related factors. For instance, one of the most commonly faced questions is whether to provide grade control to a degradation reach with a series of small low-drop type structures or by a single high-drop structure. To select the most appropriate scheme, the engineer must consider a number of factors.

Single high-drop structure

Advantages

- less right-of-way required for a single structure versus several smaller structures
- improved bank stability due to decreased bank heights
- possible reestablishment of hydraulic connection between channel and flood plain
- possible flood attenuation if flows are stored in flood plain behind structure
- ability of single main stem structure to provide grade control to tributaries
- potential habitat benefits associated with large pool area upstream of structure

Disadvantages

- obstructions to fish passage
- potential for downstream degradation due to trapping of sediments
- high cost of large structure
- complex detailed design effort
- potential flood control impacts
- potential for safety problems at high-drop structures

Multiple low-drop structures

Advantages

- less cost for design and construction
- less environmental impacts due to fish passage
- less potential for morphological impacts
- no significant alterations of flows and sediment transport

Disadvantages

- limited impact on bank stability
- difficulty in determining the appropriate siting of a series of structures
- potential environmental destruction associated with construction (access, site preparation) at numerous locations along the channel
- no reconnection of channel and flood plain

In the final analysis, the engineer must weigh all the advantages and disadvantages of the two schemes and determine which approach achieves the project goals at the least cost and with the smallest potential for adverse environmental impact.

Conclusion

Grade control structures have been used effectively as erosion control features in water resources projects for many years. Unfortunately, these structures have often been considered rehabilitative features to be used only after the channel system has been destabilized. A more effective use of these structures is to incorporate them into the initial plans for the channel system in a proactive, rather than a reactive manner. As water resources projects become more and more complex, grade control structures need to be considered in a much broader sense to provide for environmental sustainability, as well as erosion control.

Example: Loose rock structure example design procedure

Many variations are available for the design of sloping loose rock structures. An example design procedure is presented to illustrate a typical design process associated with sloping loose rock drop structures. Inclusion here should not be considered as an endorsement of this particular approach over other approaches or structure types since, as noted earlier, there is no single approach that is applicable to all situations. The following is an example of the design of a series of sloping loose rock grade control structures on Blue Creek in Illinois (Roseboom et al. 2000).

Blue Creek is located approximately 5 miles outside of the town of Pittsfield, Illinois, and has a drainage area of about 3 square miles. Headcutting along Blue Creek was causing severe channel instability and loss of instream habitat. In response to this problem, a series of sloping, loose rock grade control structures were constructed in 1998 for channel stability and habitat restoration. Figure TS14G–20 (Watson and Eom 2003) shows the 1997 preconstruction thalweg profile and structure crests for the 12 grade control structures along the 3,500-foot-long study reach. As shown in figure TS14G–20, the reach average thalweg slope in 1997 was about 0.0029. During a 2002 resurvey, the water surface slope between structures averaged about 0.0012 (fig. TS14G–21 (Watson and Eom 2003)).

The grade control structures generally followed the Newbury and Gaboury (1993) design. The height of structures above the preconstruction bed varied from 2 to 5 feet, and the average elevation difference between structure crests in 1998 was about 1.1 feet. Crest stone diameters averaged 3 feet, but the crest stones were highly variable. The downstream slope of each structure was 1 on 20 (5%), and the upstream face of the weir extended upstream on a 1V:4H slope. Figure TS14G–22 (Watson and Eom 2003) shows photographs of one of the structures 1 month and 18 months following construction. Figure TS14G–23 (Watson and Eom 2003) shows a sketch of a typical structure. Roseboom et al. (2000) stated that no additional stabilization efforts have been required since construction; the eroding streambanks have revegetated, and the pools have deepened.

The following is a design procedure for the sloping rock grade control structures (modified from Watson and Eom 2003):

Step 1 The crest stone is to be constructed of quarry stone (approximately 3 ft by 3 ft by 2 ft) with the approximate center of the structure at the crest elevation specified. The remainder of the crest stone should be constructed to form a shallow V-shape with 0.5 to 1.0 foot of relief. The bed for the crest should be excavated to firm material. If the structure is to be placed on pervious material, consideration should be given to providing an impervious fill section to prevent seepage through the structure.

Step 2 The crest should be keyed into both banks using a riprap-filled trench, which extends to the greater of the top bank elevation or the 2-year flood. A desirable slope for the key trench is 3H:1V. A gravel blanket should be placed in the key trench and over the riprap if sandy material or piping of ground water is observed.

Step 3 Upstream and downstream of the crest is filled using riprap, sized in accordance with EM 1110–2–1601 (USACE 1994a revisions on 1991 version). Recommended slopes are 4H:1V upstream and 20H:1V downstream. The following rock size example is from one of the structures on Blue Creek. The unit discharge (q) was calculated from the bankfull flow of about 13 cubic meters per second and a width of 6 meters to be 2.2 cubic meters per second per meter. From equation TS14G–5, a D_{30} value for the riprap was determined to be 331 millimeters, or 1.09 feet. Figure TS14G–24 (Watson and Eom 2003) shows where the Blue Creek D_{30} value plots with respect to several commonly used riprap gradations. As shown in figure TS14G–24, the Blue Creek D_{30} value plots near the lower limit of both the B-Stone and R–400 stone and is centered within the R–650 stone limits. Therefore, the R–650 stone appears to be the most appropriate for this situation. However, the final choice must be tempered by other factors such as cost, availability, filter requirements (B-Stone might not require addition of filter), and the designer's experience.

Step 4 Spacing of structures along the stream was designed to ensure that the crest elevation of the downstream structure is at or above the toe of the thalweg elevation of the downstream face at

the location of the upstream structure weir crest. Spacing of the structures becomes closer as the existing bed slope steepens and increases where the bed slope is flatter. This is a conservative spacing that assumes that the final stable channel may not create a significant backwater that would cause sediment deposition upstream of the structure.

This is justified because the structures are low in height and do not provide a flow constriction. If the structures were higher or provided a significant flow constriction, a steeper equilibrium slope might develop through sediment deposition, and then the structures could be spaced further apart.

Figure TS14G–20 Blue Creek, IL, 1997 thalweg profile and structure locations and elevations

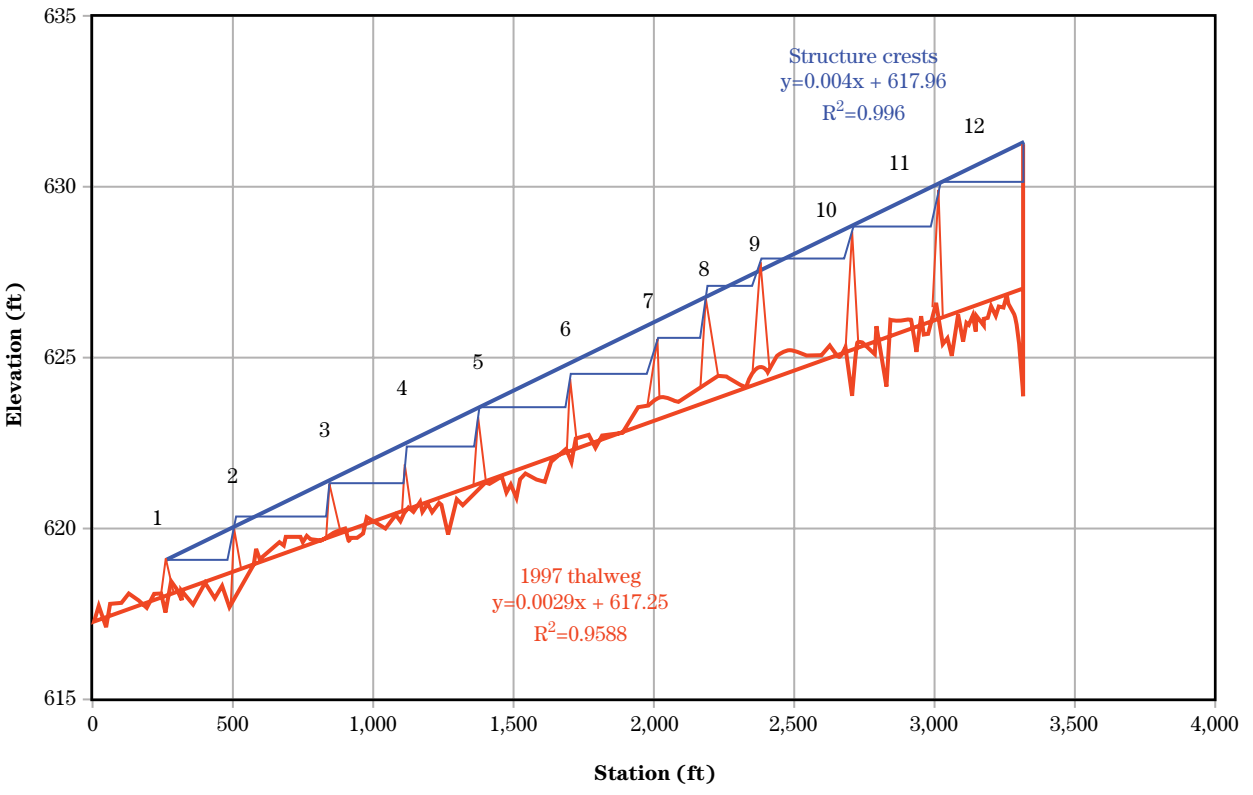


Figure TS14G–21 Blue Creek, IL, thalweg profile surveyed in 2002

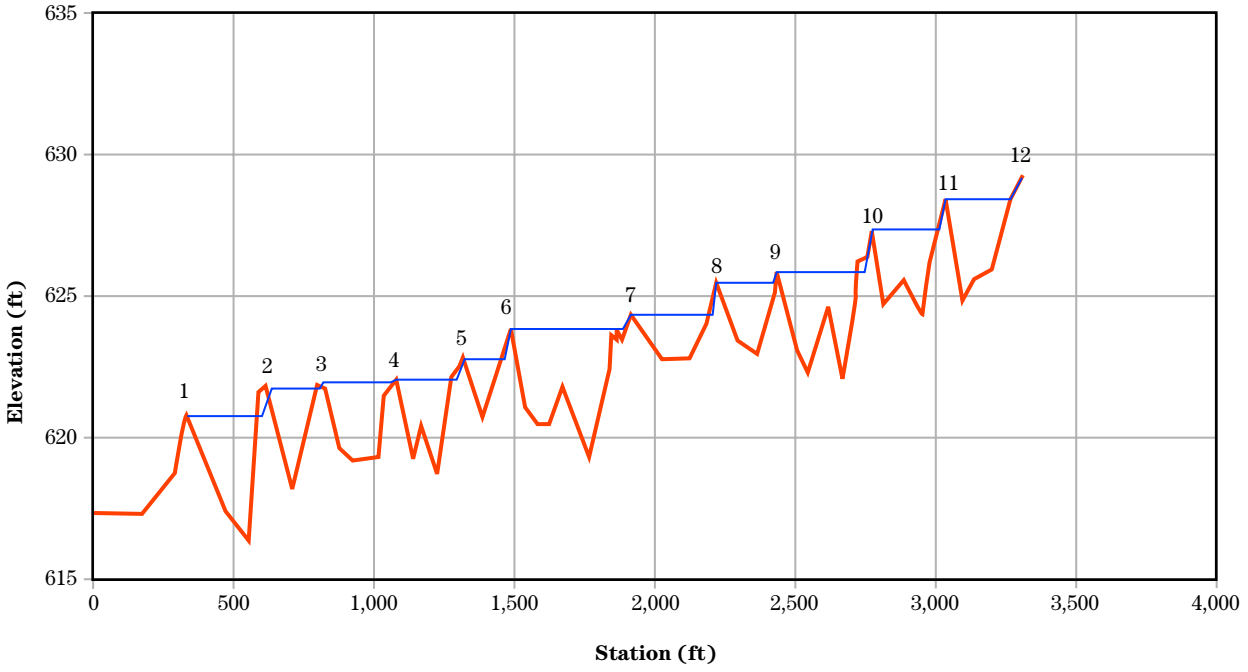


Figure TS14G–22 Grade control structure 1 month and 18 months after construction (Blue Creek, IL)

Figure TS14G–23 Grade control design (Blue Creek, IL)

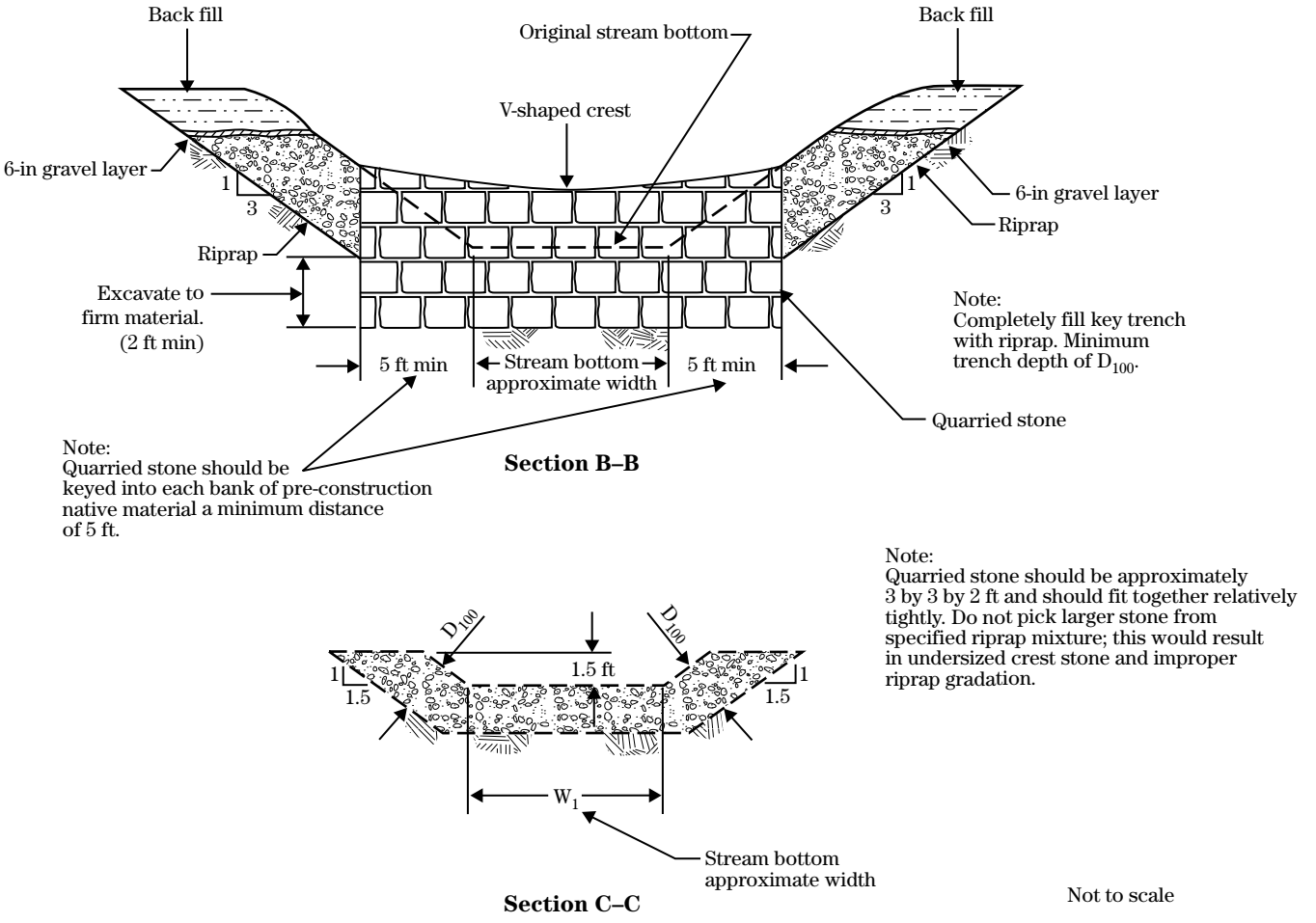


Figure TS14G–24 Grade control design (Blue Creek, IL)

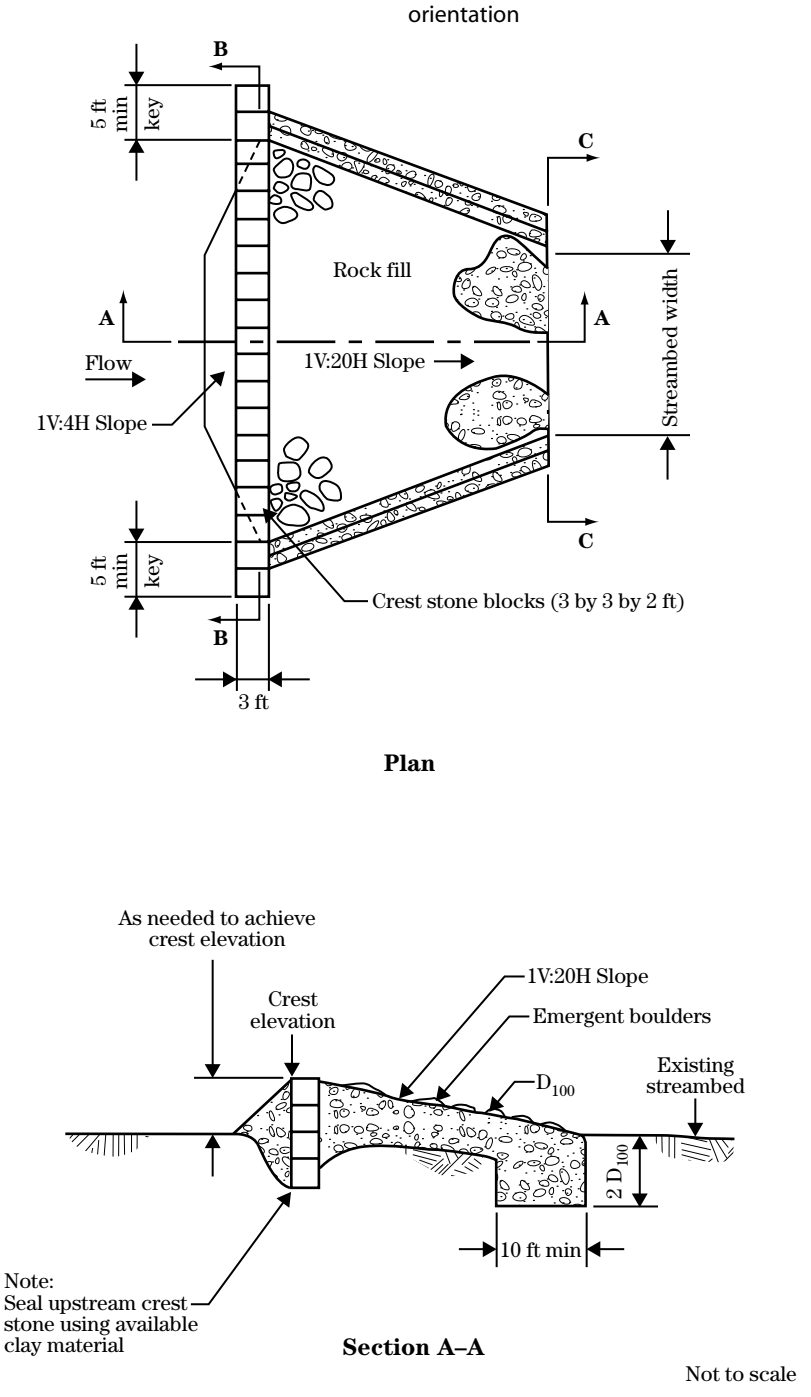
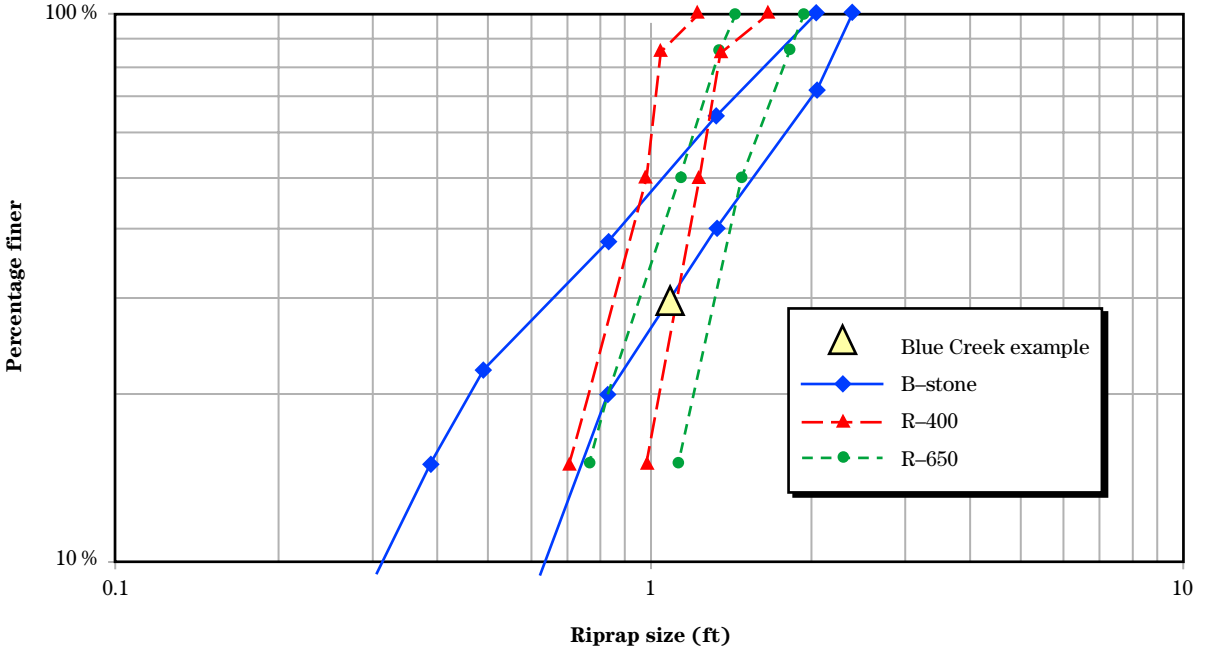


Figure TS14G–25 Riprap gradations for B-Stone, R–400, R–650, and D_{30} from the Blue Creek example



Technical Supplement 14H

Flow Changing Techniques

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Weirs, barbs, spurs, and dikes can be used to focus high stream velocities away from the banks, resulting in bank stability.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14H

Flow Changing Techniques

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Technical Supplement 14H

Flow Changing Techniques

Purpose

Flow changing devices are a broad category of structures that can be used to divert flows away from eroding banks. They are often used to shield banks from eroding flows, build up the toe of the bank, and direct flows to create a stable alignment. While this technical supplement provides descriptions of a variety of techniques, the primary focus is on the analysis, design, and installation of stream barbs. This supplement draws on recent field evaluations that focused both on projects where these structures have performed satisfactory, as well as areas where the performance has been less than satisfactory. A design description includes cautions and warnings related to specific design features. Finally, a step-by-step design procedure for stream barbs is also provided.

Introduction

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) has installed numerous flow-changing techniques in support of both streambank stabilization and stream restoration practices. This supplement primarily addresses stream stabilization techniques that work to decrease flow stresses on an eroding streambank through redirection of flow. While a variety of techniques are described, the primary focus of this supplement is on stream barbs. This supplement also provides current NRCS design recommendations for stream barb design.

Flow-changing techniques

The structures used for stream and bank restoration in NRCS projects can be categorized into one of three general classes. The terms used to identify structure classes are somewhat descriptive of the structure function.

- deflector
- redirective
- retard

A deflector type structure forms a physical barrier that protects the bank and forces the flow to change direction either by direct impact or deflection. These structures tend to be massive and often continuous along the protected reach. When properly designed, deflector structures are stable over a wide range of flow conditions.

Rock riprap, grouted rock, concrete lining, rock jetties, gabions, and spur dikes are examples of deflector structures that have historically been used in streambank protection work. Except for rock jetties and spur dikes, these structures harden the bank and reduce roughness, thereby increasing flow velocity. Common building materials for these structures are graded rock, concrete, earthfill, or combinations of these materials. Some of these techniques are addressed in more detail in NEH654.14.

A redirective type structure is designed to be placed in the stream to minimize direct impact and rely more on the characteristics of fluid mechanics to modify the streamflow direction. These structures tend to be less massive and are submerged at higher stages of flow. Redirective structures are usually discontinuous, independent structures. In many cases, they are more likely to be damaged during major events.

Spurs, rock veins/weirs, stream barbs, and bendway weirs fall into the category of redirective structures. Redirective structures can be contrasted with deflector techniques, such as riprap and gabions, which are more static and harden the bank. Common building materials for these structures typically include large rock, graded rock, and earthfill.

A retard structure increases flow resistance by increasing drag, thereby slowing the velocity in the vicinity of the structure. These structures are more porous with a high percentage of open area. Retard structures are generally used where the channel carries a high sediment load and reducing the velocity will result in sediment deposition. Common building material for these structures can include wood, steel, rock, and live plantings. Fence jetties, Killner jacks, timber piling, live poles, and most bioengineered structures are examples of retard structures. Some of these structures are addressed in more detail in NEH654.14.

It is not uncommon to use all three types on projects initiating and terminating protected reaches with deflector type structures and using redirective and retard structures between the hard points. All of the methods

mentioned can be combined with bioengineering measures to improve stream function and bank stability. A general outline of the different techniques is provided in table TS14H–1. Some of these techniques are addressed in further detail in this technical supplement, as well as in NEH654.14.

Table TS14H–1 Common flow-changing techniques, brief description, structure class, and function

Practice	Description	Structure class	Function
Concrete bank lining	Hard, smooth surface of concrete, gravity, or structural support	Deflector	Flow is physically deflected or trained by physical barrier
Rock masonry bank lining or wall	Hard, semismooth of rock and mortar, gravity support	Deflector	Flow is physically deflected or trained by physical barrier
Geocell slope/bank protection	Fine or granular fill retained in cells, semismooth to rough, vegetated option	Deflector	Flow is physically deflected or trained by physical barrier
Rock riprap	Loose rock on slope, semismooth to rough, full or partial bank	Deflector	Flow is physically deflected or trained by physical barrier
Groins	Rock dike projecting into stream in downstream direction	Deflector	Full range of flows physically deflected away from bank
Dike	Earth or rock full bank height	Deflector	Flow is physically deflected or trained by physical barrier
Stream barbs	Low rock sill projecting into stream	Redirective	Flow direction changed by flow over structure
Bendway weirs	Low rock sill projecting into stream	Redirective	Flow direction changed by flow over structure
Rock vein	Instream rock sill	Redirective	Flow direction changed by flow over structure
Rock “V” weir	Instream rock sill	Redirective	Flow direction changed by flow over structure
Spur dike	Short rock, timber, or earth dike projecting from bank, porous or impermeable	Deflector/retard	Physical barrier, full bank height
Jetties (fence)	Parallel lines of spaced posts, porous	Retard	Velocity of flow through structure is reduced by friction
Live stakes, geogrids, brush layers	Vegetative treatment	Retard/deflector	Velocity of flow through and around vegetation is slowed by friction
Vegetated slope	Vegetative treatment	Retard/deflector	Velocity of flow through and around vegetation is slowed by friction

Spur dikes

Spur dikes are short dikes that extend out perpendicular from the bank into the channel along a reach of eroded bank. Spur dikes can be short or long, but generally with a top elevation above flood stage or equal to the bank elevation. Streamflow impacting spur dikes is retarded and diverted away from the bank. Spacing of the spur dikes is important to prevent formation of strong eddies that can result in erosion between the dikes. Spur dikes are generally constructed using earthfill with rock riprap surface protection. However, soil bioengineering practices can also be used in between spurs.

Groins

Historically, groins have been in widespread use for many years and are the precursors to redirective structures. Much of the guidance for redirective structures is based in part on the experience with groins. However, there are important differences that the designer must keep in mind. Groins typically are higher profile and affect all stages of flow. Their crest is typically above the high-flow water surface elevation, and they are seldom completely submerged. They act to deflect flows away from the bank. They have a significantly higher effect on the shape of the streams cross-sectional shape since they are used to narrow the stream. Since they are rarely overtopped, they can be effective when oriented downstream.

Jetties

Jetties are fence-like structures extending from the bank into the stream. They are often installed in pairs or multiple pairs to train flow towards the center of the channel. They can also be installed on one side of a stream channel to direct flow away from that bank. Jetties can be permeable or impermeable and are usually installed diagonally in a downstream direction along the bank.

Figure TS14H–1 shows an example of permeable fence jetties. Permeable jetties are used for streams with high sediment loads. The flow passing through the jetty is slowed, allowing deposition of material between the jetties. Impermeable jetties are seldom used except where the line of flow must be diverted away from a structure or other feature. Permeable jetties can also be constructed out of woody debris, jacks, or a combination of logs and large boulders. In streams where there is a large amount of woody material and debris, permeable deflectors can collect and retain this material and become less permeable with time. Once they become impermeable, the portions that project from the bank may function more in a redirective capacity.

Figure TS14H–1 (a) Permeable fence jetty, close up; (b) Aerial view (*Photo courtesy of Lamont Robbins, NRCS*)

(a)

(b)

Pin deflectors

A variation of the permeable jetty is the pin or piling deflector. Pin deflectors are generally used in streams where only a small reduction in velocity is needed. Generally, wood pilings are used for their construction. These pilings are driven to a depth where they can resist the forces of the water, as well as any anticipated drift and debris that they may collect. A rule of thumb is a depth that is at least twice that of the projection above the channel bottom, but this is dependent on channel materials. In some applications, it is specified that the piling be driven to refusal. After being driven to the design depth, the pilings can be trimmed with a chain saw to form the design profile. Pilings can be linked with cross pieces or left as individual elements. When connected, they act together. When unconnected, outer wood pilings may fail without putting the rest of the structure in jeopardy.

Bendway weirs

Bendway weirs were developed by the U.S. Army Corps of Engineers (USACE) to reduce erosion along the Mississippi River, and then adapted for smaller streams. As with stream barbs, the premise behind the function of bendway weirs is that flow over the weir is directed perpendicular to the angle of the weir. Bendway weirs are oriented upstream at an angle that is between 50 to 80 degrees to bank tangent. The length of a bendway weir is typically less than a fourth bankfull width. Often, the design is based on baseflow widths. In this case, their length is typically between a fourth to a half of the baseflow width. In all cases, both the length and angle may vary through the bend of the river to better capture, control, and direct the flows. They are typically wide structures with a flat to slight weir slope up toward bank. They should be keyed into the bank at a length equal to the bank height plus anticipated scour depth. More information on the design and application of bendway weirs is provided in the WES Stream Investigation and Streambank Stabilization Handbook (Biedenharn, Elliott, and Watson 1997). While bendway weirs are often used on large streams and rivers (fig. TS14H–2), an example of a bendway weir on a small stream is shown in figure TS14H–3.

Numerous applications have shown that bendway weirs reduce the velocity near the bank. On the little Blue River in Kansas, Balch (2004) observed a 50-percent reduction in stream velocities within the weir field (fig. TS14H–4).

Stream barbs

Stream barbs are low dikes or sill-like structures that extend from the bank towards the stream in an upstream direction. Stream barbs are similar in structure to bendway weirs, perform a similar function, and were developed about the same time by NRCS for

Figure TS14H–2 (a) Bendway weir, under construction; (b) Completed bendway weir (*Photos courtesy of Mark Locke, NRCS*)

(a)

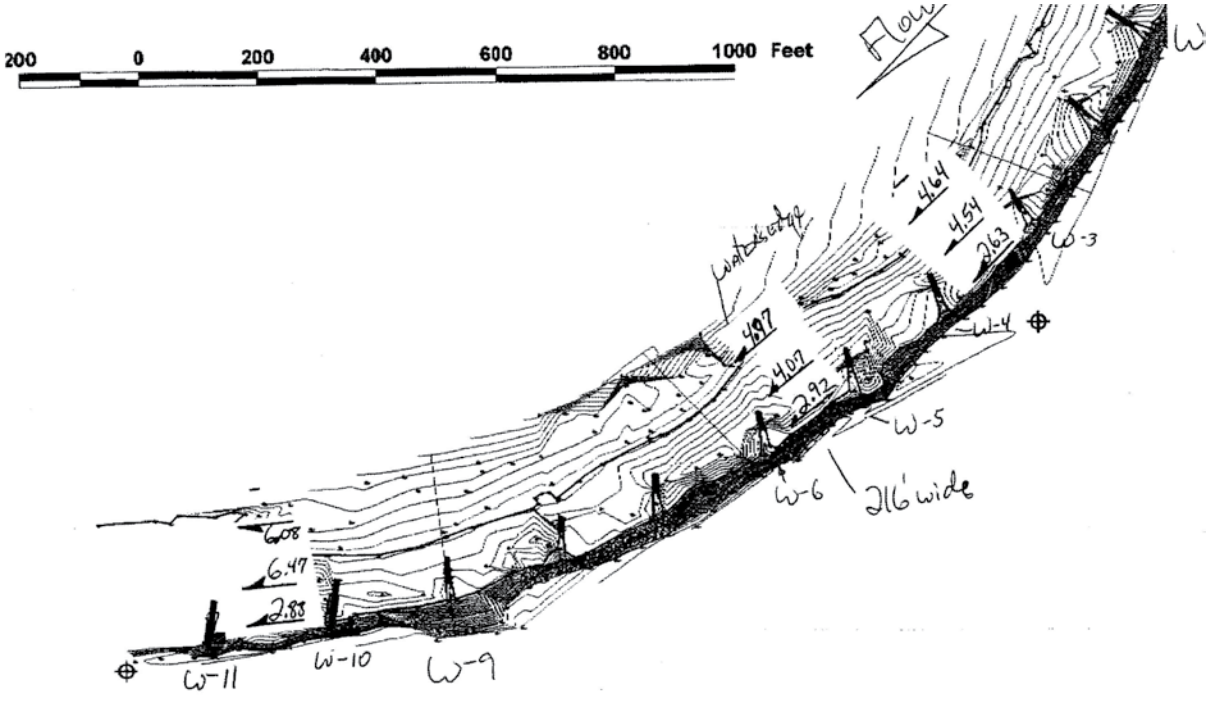
(b)

Figure TS14H–3 Bendway weir (*Photo courtesy of Wayne Kinney, NRCS*)

smaller streams. As flow passes over the sill of the stream barb, it accelerates, similar to flow over the weir of a drop structure, and discharges normal to the face of the weir. Thus, a portion of the streamflow is redirected in a direction perpendicular to the angled downstream edge of the weir. If the weir is too high, flow is deflected instead of being hydraulically redirected, and if too low, the redirected flow is insignificant relative to the mass of the stream.

Performance varies as the streamflow stage varies. At low flows, a stream barb may first deflect flow, and then, as the stage increases, flow passes over the weir and is redirected. At high-flow stage, the weir effect becomes insignificant. The height of the stream barb weir is important, since it will generally function most

Figure TS14H–4 Water velocities on Geffert River Project, Neosho River, Allen County, KS—12 feet of water over weirs. (*Observations and sketch by P. Balch, D. Derrick, and B. Emmert in 2001*)



efficiently during bankfull or channel-forming flow events. Welch and Wright (TN–23(2) (USDA NRCS 2000)) have noted that, for purposes of many stream barb designs in the Pacific Northwest, the bankfull stage generally coincides with the regulatory field interpretation of ordinary high water. Stream barbs are typically constructed with rock; however, brush may be used for some applications. Figure TS14H–5 shows both rock and brush barbs. More information on the design of brush barbs is provided in NEH654 TS14I.

Stream barbs are used for bank protection measures to increase scour of point and lateral bars, direct streamflow towards instream diversions, and change bedload transport and deposition patterns. Other benefits of stream barbs include encouraging deposition at the toe of a bank, reducing the width to depth ratio of a stream channel, and providing pool habitat for fish. Trees with rootwads can be added to these structures to improve fish habitat value. The design of stream barbs is addressed in more detail later in this technical supplement.

Figure TS14H–5 (a) Rock barbs; (b) Brush barbs

(a)

(b)

Vanes

Vanes are structures constructed in the stream designed to redirect flow by changing the rotational eddies normally associated with streamflow. They are used extensively as part of natural stream restoration efforts to improve instream habitat. There are quite a few variants on rock vane design. The Rosgen style cross vane and J-hook structures are addressed in NEH654 TS14G and NEH654.11.

Vanes are typically oriented upstream 20 to 30 degrees to the bank tangent. However, the angle may vary as they work around the curve. Design of vanes is based on bankfull depth. The length is typically a third of the bankfull width, and the height at the bank is a third of the bankfull depth. The weir slope is 2 to 7 degrees up towards bank. The required stone size for vanes is often very large. A typical rock vane is shown in figure TS14H–6.

Figure TS14H–6 Rock vane

Stream barbs

The NRCS has installed numerous stream barbs to protect streambanks throughout the country in support of stream restoration practices. The term stream barb refers to a low-sill (typically rock) structure that projects from the streambank into the flow, angled in an upstream direction. These structures typically have geometry developed from site-specific hydrologic and hydraulic characteristics. Their purpose is to decrease flow stresses on an eroding streambank primarily through redirection of flow.

In the early 1990s, NRCS field staff in eastern Oregon began using low rock sills in stream restoration work. These structures were designed to redirect flow away from eroding banks and required much less rock than traditional rock riprapped banks. The structures were referred to as stream barbs. These structures offered an alternative to rock riprap (which had lost favor with state fisheries personnel), and NRCS field staff were enthusiastic because they seemed to work well with other bioengineering bank treatments. However, there were no set design procedures or guidelines for installing them, other than to use the largest rock available. A field evaluation in 1993 by NRCS West National Technical Center personnel resulted in the development of preliminary design guidelines for layout and installation of stream barbs. Since those first guidelines were issued, these structures have been installed at many sites across the country. Field and empirical observations have resulted in changes to the original guidelines and improvements continue. In 2001, the National Design, Construction, and Soil Mechanics Center (NDCSMC), in cooperation with state NRCS personnel, began to conduct a systematic review of stream barb projects at various sites across the country to compile the lessons learned in their successful design and implementation (Saele et al. 2004). This effort included site visits, review of plans, and interviews with designers. This section incorporates current design practices with a step-by-step worksheet to facilitate design and layout of these structures.

Hydraulic function

As noted earlier, a stream barb is a low sill-like structure that projects into the streamflow, oriented in an upstream direction. Stream barbs redirect streamflow with a very low weir and disrupt the velocity gradient in the near-bank region. Stream barbs can provide two hydraulic functions which serve to provide stability to a streambank.

- divert erosive streamflows away from the bank
- encourage deposition at the toe of the bank

The low-weir section is pointed upstream and forces the water flowing over it into a hydraulic jump. Flowing water turns to an angle perpendicular to the downstream weir face causing the flow to be directed away from the streambank. Figure TS14H–7 shows observations of near bank velocity reductions through a series of stream barbs during moderate flows.

The weir effect continues to influence the bottom currents even when the barb is submerged by flows greater than the channel-forming flow. When functioning to divert flows in this manner, the height of the structure in relation to the design storm is more important.

Stream barbs can encourage the creation of a low bench at the toe of an eroding bank. In this case, the height of the structure is not as critical. The disruption of the velocity gradient as the water flows over the weir section reduces channel bed shear stress and slows near bank flows, resulting in sediment deposition adjacent to the barb. The flow separation caused by the hydraulic jump and flow redirection creates an eddy downstream of the barb. This eddy can promote sediment deposition. However, it is important to note that a significant sediment load must exist in the stream at low to moderate events for this deposition to occur. The best sediment deposition performance has been observed where plants were included in the design and when additional plantings were provided after deposition began. Treatments such as tree revetments (see NEH654 TS14I) between the barbs also act to encourage sediment deposition.

Design criteria

The following is a generalized discussion of design criteria specific to stream barb design. Since all designs in a riverine environment are site specific, the user is cautioned that there are certainly variants in many of the recommendations that are provided herein. Refer to figures TS14H–8 and TS14H–12 for clarification and identification of terms.

Figure TS14H–7 Approximate surface velocity measurements at Snake River at Moose, WY. The average of the annual mean annual streamflows from 1996 to 2004 at this site was approximately 3,200 ft^3/s.

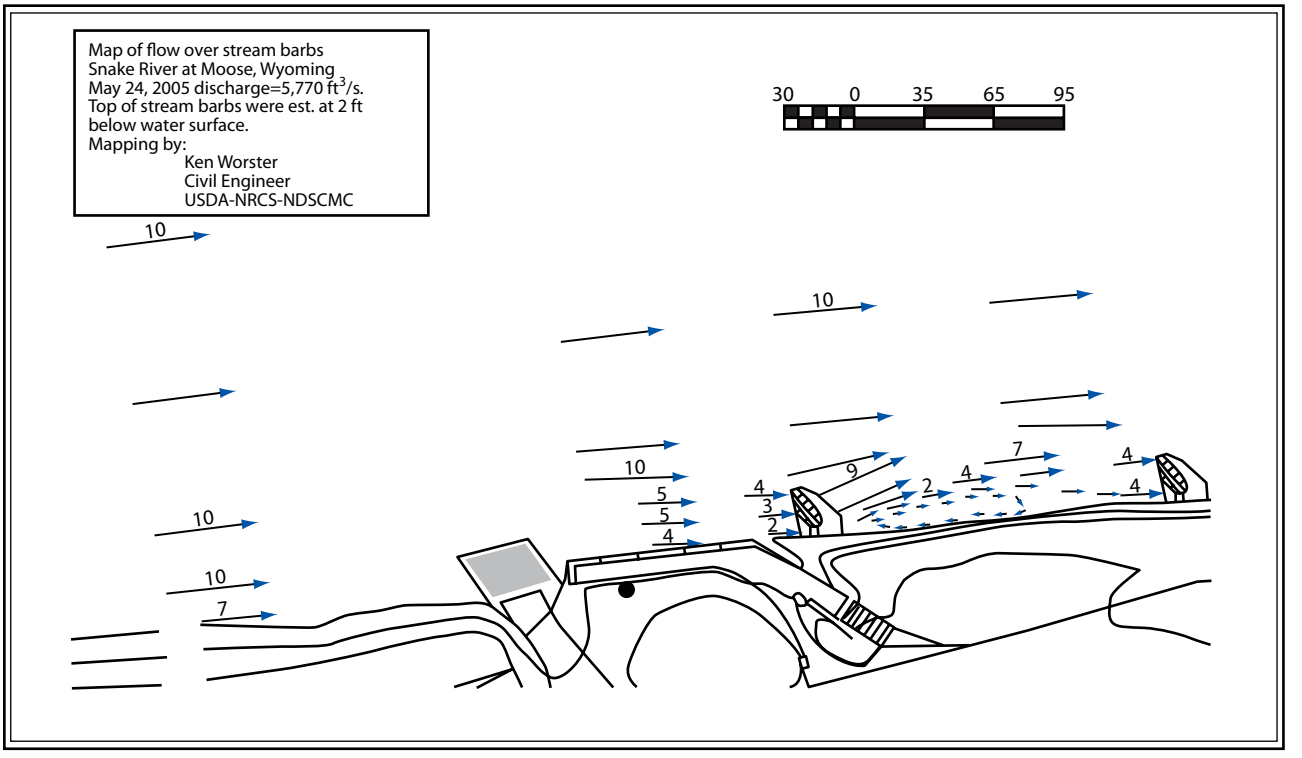
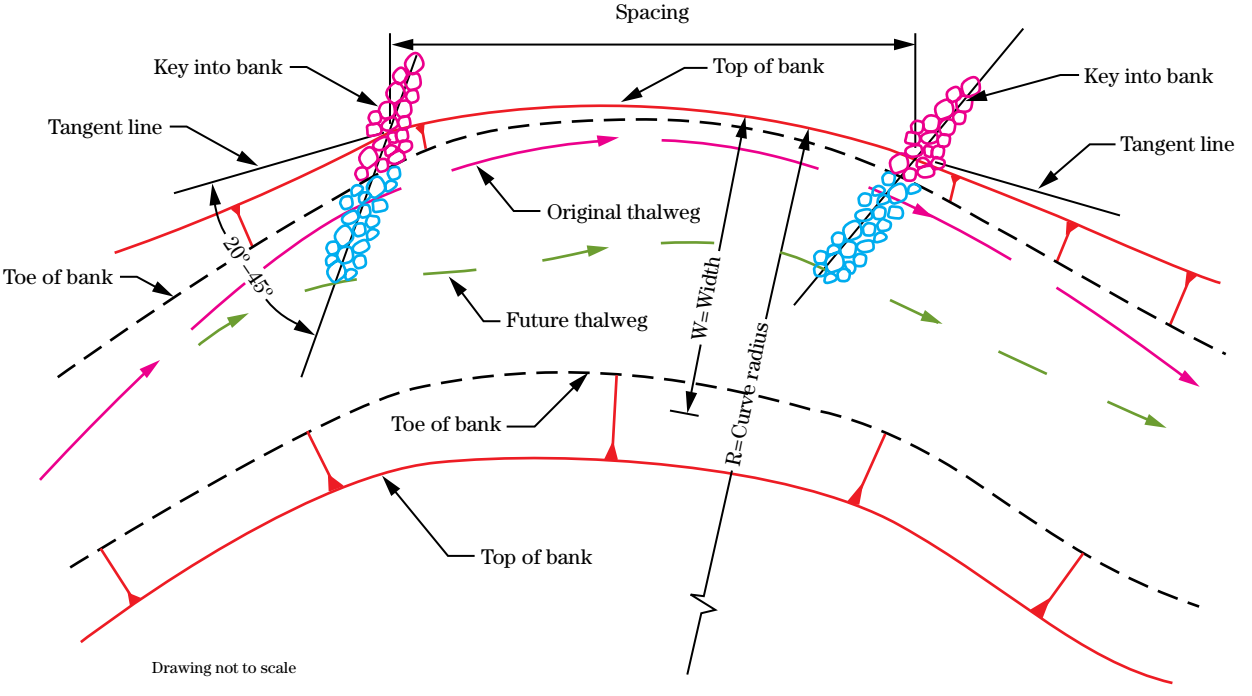


Figure TS14H–8 Typical stream barb design layout



Bank erosion—The cause of bank instability must be carefully assessed by the designer. Stream barbs are appropriate for sites where the mechanism of failure is toe and lower bank erosion. They decrease near-bank velocities and create low-flow eddying adjacent to the toe of the bank which promotes sediment deposition. They are often used in combination with soil bioengineering methods since the sediment deposition and accumulation between the barbs promotes riparian establishment and development. Soil bioengineering techniques may also enhance further deposition between the barbs.

Stream barbs will not protect banks that are eroding due to rapid drawdown or mass slope failure. Problems have been observed where stream barbs have been applied to repair problems that are geotechnical, rather than fluvial in nature.

Channel stability—Stream barbs are not appropriate where the grade of the channel is unstable. In degrading streams, the foundation of the stream barb may be undermined, while in aggrading streams, the stream barb may be buried. In addition, problems have been observed where these techniques have been applied in braided streams or stream systems that are prone to avulsions.

Channel approach—The placement, length, and alignment of barbs are dependent on the approach that the channel makes into the project area. Using stream barbs to make abrupt channel alignment changes should be avoided. The designer should consider the full range of flow behavior at the site as the alignment may change at high flows. For all significant design flow levels, the stream barb should serve to redirect, rather than deflect or split the flow.

Location—Stream barbs are typically placed along the outside of a bend where the thalweg is near the streambank. Generally, these structures are not used when the thalweg is away from the bank, except in situations where the channel is excessively wide or where they are used to induce sediment deposition at the toe of an eroding bank. The stream barb should then be located to capture the flow with a longer weir section, control it through the curve, and direct it downstream towards the center of the channel.

The furthest upstream stream barb should be located in the area that is first impacted by active bank erosion. Research by Matsuura and Townsend (2004) indicates that stream barbs upstream of the active erosion were less effective than those placed at the point that bank erosion starts. Designers should note that since most of the stress is in the lower two-thirds of a bend, protection should extend to the point where the bank is stable and vegetated.

Field assessments documented by Sean Welch and Scott Wright in NRCS TN–23(2) (USDA NRCS 2000) indicate that the placement should be restricted to the outer portions of the current meander belts. This will reduce the possibility of flanking. Figure TS14H–9 illustrates a typical meander belt in a Rosgen C4 class river.

Bend radius—While stream barbs are primarily used to control erosion in bends, their performance may not be satisfactory in sharp bends. When the meander bend radius divided by stream width is much less than three (R/W<3), there are often problems with erosion below the stream barb as a result of flow separation. This restriction may be relaxed by protecting the banks between the barbs, increasing the number of barbs and decreasing the angle between the barb and the bank. However, in appearance, this may result in nearly a fully riprapped bank.

Determining a radius is not necessarily a simple exercise. Many bends are, in fact, more of a spiral. In addition, the bend radius and approach angle may change at high flow. The designer must assess affects at low, moderate, and high flows. As with all aspects of stream barb design, experience and judgment play an important role.

Studies are underway to develop design measures that will improve stream barb performance for R/W<3 (Matsuura 2004). Also, it should be noted that some sites have been observed with R/W ratios approaching two that seem to be functioning well. However, this may be due to approach and alignment at the erosive flows being such that the radius is in effect increased.

Angle—The structure weir section must be oriented in an upstream direction. The angle (θ) generally varies, from 20 to 45 degrees off a tangent to the bank, depending upon the curvature of the bend and the intended realignment of the thalweg. The tighter the stream bend, the smaller the angle, and for situations where R/W <3, it probably should be less than 20

degrees. If the purpose is to maintain a deep thalweg near the streambank, then a tight angle (20°) is desirable. A vector analysis, assuming a perpendicular flow direction from the weir alignment, can be used to estimate the angle required to turn the flow.

Length—There are two important length terms associated with stream barbs: weir length (L_w) and effective length (L_e). Weir length defines the length of the weir section of the stream barb and is relative to how much flow can be redirected and energy dissipated. The longer the weir, the more streamflow affected and energy dissipated. Effective length is a function of the stream width (W) and defines the perpendicular projection of the stream barb from the bank into the stream. Experience has shown that an L_e greater than a third the stream bankfull flow width has been observed to result in unsatisfactory results by causing erosion on the opposite bank.

Maximum effective length: $L_e = \frac{W}{4}$

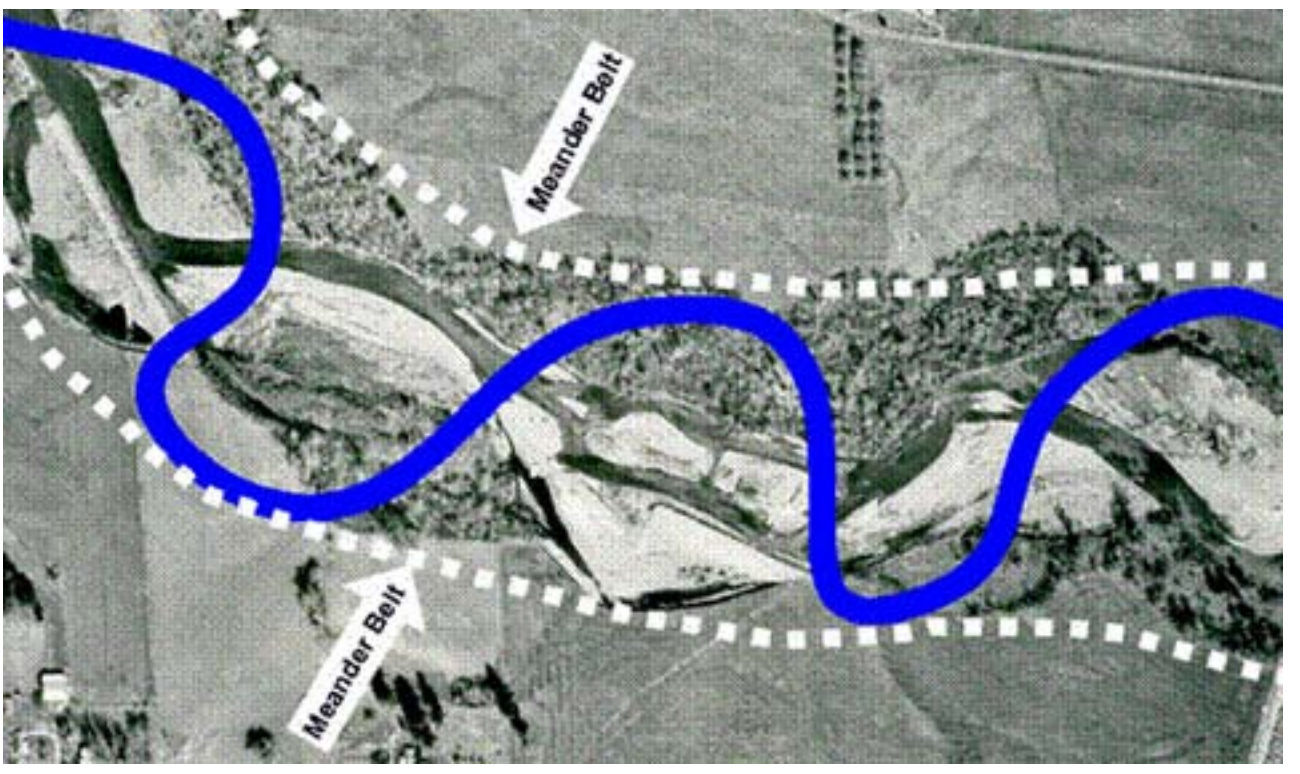
$$L_W = \frac{L_e}{\sin\theta}$$

Suitable range of L_e for effective bank protection:

$$\frac{W}{10} < L_e < \frac{W}{4}$$

For stream barbs to affect the dominant flow pattern, they must cross the thalweg. Shorter stream barbs will affect only secondary, near-bank currents. If the calculated effective length results in barbs that do not influence the dominant flow path, adjustments should be made to the barb length. If this is not feasible, other techniques should be considered. Stream barbs that extend much beyond the effective length tend to alter the meander pattern of the stream and could adversely impact the opposite bank. Stream barbs should not

Figure TS14H–9 Historical meander migration limits



be used to change the meander pattern of an entire stream system or to channelize the streamflow.

Number and spacing—The number of stream barbs required at any given site will be determined by the following:

- spacing
- the length of the eroding meander bend
- channel geometry
- desired effect for treatment of reach

Proper spacing of stream barbs is necessary to prevent the streamflow from cutting between two barbs and eroding the bank. A vector analysis consists of plotting the proposed layout with vectors projecting at right angles to the downstream side of the stream barb. This can provide the designer with an indication of flow lines and flow interception by subsequent stream barbs. Given that the flow will leave the stream barb in a direction perpendicular to the downstream weir face, the subsequent structure should be placed so that the flow will be captured in the center portion of the weir section before the streamflow intersects the bank. Since the flow direction is controlled by the alignment of the stream barb, the downstream side of the stream barb is typically straight, so that this direction can be better estimated. Another method that can be used is shown on the design worksheet.

Although there is much local variation, typically, stream barbs influence the flow patterns for a distance downstream from five to ten times L_e. A limited stream barb spacing of four to five times L_e provides more consistent results.

Height—The height of the stream barb weir section (H_w) is related to the channel-forming or bankfull flow depth. The main portion of the weir should be below the bankfull flow depth, such that significant flow is over the weir. In some situations, a stream barb may be used to protect banks from flows that are considerably larger than bankfull. In these situations, the height may be larger, but generally, should not exceed the bankfull flow level, as this results in a jetty, rather than a barb.

The height of the stream barb weir is generally limited as follows:

$$H_W = \frac{1}{3}D_a \text{ to } \frac{1}{2}D_a \qquad \text{(eq. TS14H–1)}$$

D_a = average bankfull flow depth (as defined on design worksheet)

Once flows are more than five times the height of the stream barb, the relative effectiveness of the barb in redirecting flow is significantly reduced. If the height of the design storm is significantly higher than the height of the barb, it may be advisable to increase the height, augment the stream barbs with more bank protection between the barbs, or select another treatment technique.

The relative height between successive stream barbs is important. The difference in height between stream barbs should approximate the energy grade line of the stream regardless of local variations in bed topography.

Profile—A stream barb is intended to function as a weir; therefore, the profile is nearly flat with a positive slope towards the bank (slope of 1V:5H is common). Stream barbs constructed with a negative slope or where rocks have been displaced resulting in a negative slope may force water closer to the bank, and thereby increase, rather than decrease erosion. The profile should transition from the weir section to a steeper slope at the bank (1V:1.5H to 1V:2H is common). A typical configuration would be a profile starting at one-third H at the outer end and increasing to one-half to two-thirds H at the bank end of weir section. The top of the key must be high enough to prevent water from flowing around and eroding behind the structure. Banks that are frequently overtopped will require a more extensive key that extends further back into the bank. Bank material will also need to be considered when designing the dimensions of the key.

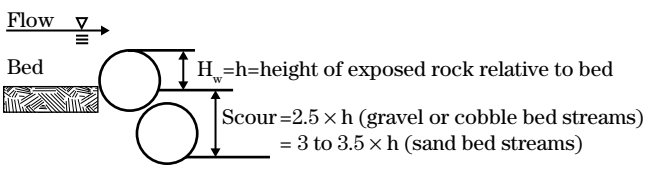
Width—The width of a stream barb generally ranges from one to three times the design D_{100} rock size. The width does not need to be more than two rock diameters and can even be the width of a single large rock at the tip of the barb. However, stream barbs with a top width of a single stone have been shown to be more susceptible to damage than structures which are multiple stones in width. The stream barb width may also need to be increased (10 to 15 feet total width) to accommodate construction equipment in large rivers or where necessary. Wider structures will result in a

more uniform, stronger hydraulic jump. Wider structures should be used if a deep scour hole downstream of the barb is expected.

Length of bank key—The purpose of the bank key is to protect the structure from flanking due to erosion in the near bank region. The bank key length should be at least 8 feet and not be less than one and a half times the bank height. Buried logs with rock ballast can be used in conjunction with the bank key. An inadequate key into the bank has been frequently observed to cause the structure being flanked. Rilling from overbank return flows down the backfilled bank key has also been observed to be a problem. It is also suggested that the key be planted with live poles and/or live clumps. The design can take advantage of the required excavation into the bank to assure adequate moisture is provided to these soil-bioengineering practices. This planting will not only enhance stability but also provide important habitat benefits. More information on soil bioengineering practices is provided in NEH654 TS14I.

Depth of the bed key—The depth of the bed key is determined by calculating the expected scour depth around the tip of the structure. This scour depth will likely exceed the depth of the thalweg. If a bed key is not incorporated, or if the bed key is too shallow, scour may erode the bed material downstream, causing the rock to fall into the scour hole. Higher barbs cause greater flow convergence, and thus greater scour depths. To reduce scour depths, decrease the barb height. The bed key is typically placed at a minimum depth of D_{100}. Scour analysis is addressed in NEH654 TS14B can be used to make these estimates. In lieu of a scour analysis, scour depth can be estimated using the information provided in figure TS14H–10.

Figure TS14H–10 Depth of bed key



If it is not feasible to excavate below the anticipated scour depth, the designer can increase the width of the weir section so that sufficient stone is available to launch into and armor the scour hole.

Scour hole development—Developing a scour hole at the nose or tip of a stream barb may be a project goal as it can provide important benefits to instream habitat. Numerous practitioners have documented the formation of these scour holes. Figure TS14H–11 (TN–23(2) USDA NRCS 2000) illustrates a typical scour hole at the tip of a stream barb in a Rosgen C4 class river.

One of the most frequently observed causes of failure is due to scour undermining the structure. Many practitioners have noted that the ends of stream barbs are often shortened with time as the rock at the nose falls into this hole. Efforts have been made to use larger rock to resist this, but it has been found that the best performance in gravel-bed streams is provided from barbs that are designed with sufficient key in to the invert of the channel.

Scour at the nose of stream barbs in sand-bed streams has been especially difficult to estimate. One approach, used on fine to medium sand rivers, is to construct the weir section of the stream barb and allow the induced scour hole to form overnight. The designer then returns the next day to rebuild the end of the structure using the launched material as a foundation (Balch 2004).

Rock size—Rock for stream barbs shall be durable and of suitable quality to assure permanence in the climate in which it is to be used. Because stream barbs are positioned to redirect fluvial forces at locations where these forces are greatest within stream channels, the rock used to construct them must be larger than the rock that would be required in a riprap revetment along the streambank at the same location. Numerous failures have been attributed to using undersized rock.

Material sizing should follow standard riprap sizing criteria for turbulent flow. One guide is the NRCS Far West States-Lane method, NEH650.16. The rock should be sized for the design flow and then modified in accordance with the following:

D_{50}, stream barb = 2 × D_{50}, as determined for streambank riprap

D_{100}, stream barb = 2 × D_{50}, stream barb

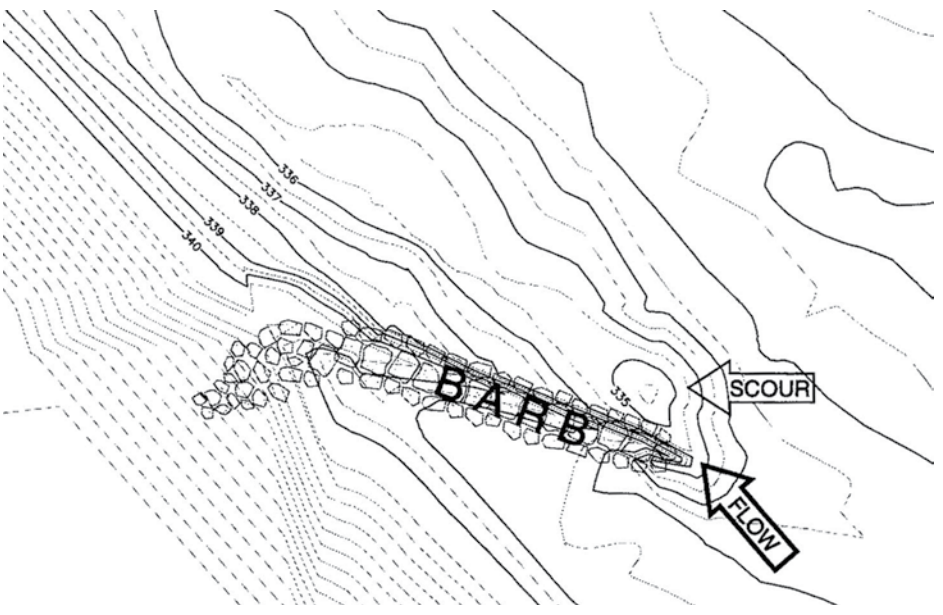
$D_{minimum}$ = 0.75 × D_{50}, as determined for streambank riprap

Note that the Far West States-Lane method gives the riprap D_{75}, and not the D_{50}. A designed gradation is required to obtain the riprap D_{50}. When the ratio of curve radius to channel width is less than six, rock sizes become extremely large and may result in a conservative design.

Rock in the barb should be well graded in the D_{50} to D_{100} range for the weir section; the smaller material may be incorporated into the bank key. The largest rocks should be used in the exposed weir section at the tip and for the bed key (footer rocks) of the barb. The Isbash curve (NEH650.16) is not appropriate for sizing rock for stream barbs, as it results in sizes too small for this application.

In general, structures that are constructed with graded material perform better than ones built out of a few large boulders. This may be due to the fact that a structure built with a larger number of smaller stones can be more easily constructed to a specified grade and can adjust better than one made out of a few larger boulders. However, it should be noted that, depending on availability, large rock (generally greater than 3 feet in diameter) can be less expensive by weight and can take less time to install. More information on stone size is provided in NEH654 TS14C and NEH654 TS14G.

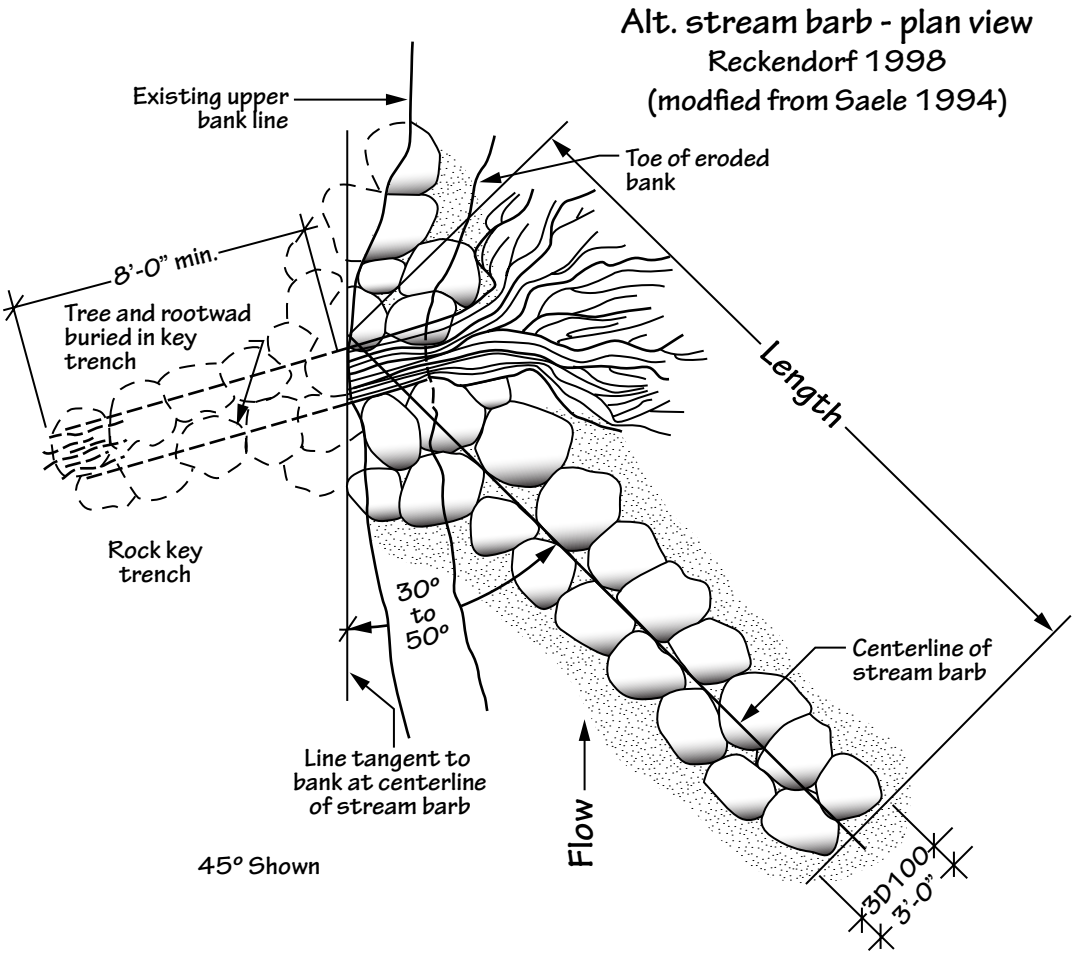
Figure TS14H–11 Scour effects at the barb tip



Woody debris—Rootwads and other woody debris have been incorporated into stream barbs to enhance aesthetics and the habitat benefits of the structure. Details of such structures are provided in figure TS14H–12. Large wood elements have also been incorporated into the weir, as well. Rootwad sections have been incorporated both perpendicular to the weir, as well as longitudinally. In either case, the anchoring requirements of the wood elements must be considered.

If the wood element is not anchored sufficiently, it may break loose, damage the structure, and possibly result in adverse downstream impacts. Anchoring could be accomplished by cabling to rock bolsters, soil anchors, or with the weight of the rocks that make up the barb. Forces of the flows during design conditions, as well as buoyancy should be considered. In addition, the consequences of the woody material catching floating debris should be considered in the design and evaluation of its anchoring requirements. More information related to designing soil anchors is provided in NEH654 TS14E.

Figure TS14H–12 Rootwad used in the key of a stream barb



Finally, the designer should also consider how the placement of woody debris within the structure might also affect its hydraulics. Woody material should not be placed and aligned where it might direct flows into the bank.

Design worksheet

This section provides a generalized worksheet for designing a stream barb. The user is cautioned that, as with all stream projects, the design and placement of stream barbs are site specific. These listed steps will likely need to be modified and adjusted for specific projects. Figures TS14H–8, TS14H–10, and TS14H–12 will facilitate these steps.

Step 1 Investigate site and obtain physical- and geomorphic-based parameters. The designer should determine if site is suitable for stream barbs.

Can *yes* be answered to the following questions::

Is erosion occurring on the outside of a bend?

Is the channel bed stable or quasi stable?

Is the stream thalweg close to the eroding bank toe?

Is this a natural channel (uncontrolled)?

If the answer is *yes* to all of the above questions, proceed.

Step 2 Determine bankfull elevation, radius of outer bank, typical section, and hydraulic gradient. Develop a plan drawing of site from aerial photo or from survey information showing outer bank, bankfull line on opposite bank, on the eroding bank if it is significantly different than top of bank, and the thalweg. Locate beginning and ending points of the eroding bank. Using CAD or other methods, approximate the outer bank radius and bankfull width. If the radius varies significantly through eroded section of bend, determine the radius, width, and area at the beginning of erosion and at one or two other points that typify the stream curve.

From field survey and cross-sectional data, determine widths, radius, and area of bankfull discharge.

Radius of bend (R)R_1 = _____
R_2 = _____

Bankfull width (W) W1 = _____
W_2 = _____
A_1 = _____

Bankfull area (A)A_2 = _____

Determine the average depth

$$D_a = \frac{\frac{A_1}{W_1} + \frac{A_2}{W_2} + \cdots \frac{A_i}{W_i}}{i} \quad D_a = _____$$

Note: The value of $\frac{A}{W}$ for each section should be somewhat similar. Use extreme outliers with caution.

Calculate the ratio of radius of bend to width (R/W) for each section of the bend, and determine the most favorable angle θ for stream barb alignment. See the description, and use the guide below.

$\frac{R_1}{W_1} \geq 3$

If <3, consider other treatment
If <6, consider reduced angle, $\theta \leq 30^\circ$
If >6, $\theta = 30^\circ$ to 45° generally satisfactory
If >9, consider larger angle, $\theta > 45^\circ$

Step 3 Mark the beginning point of bank erosion on the outer bank curve. This determines the location of the first stream barb and marks the point where the downstream face of the weir will intercept the bank line.

Step 4 Draw a tangent to bank curve passing through the point where the weir line intercepts the bank. Refer to design layout (fig. TS14H–13). Note that the circled numbers refer to the step numbers listed herein.

Step 5 Beginning at the tangent point above, draw a line angled upstream, θ degrees (determined in step 2), from the tangent line and extending streamward. This line forms the downstream face of the stream barb. Extend this line out a sufficient distance to cross the thalweg, and measure the length from the bank. This length determines the stream barb weir length.

Step 6 Determine the effective length (L_e) of stream barb:

$$L_e = L \times \sin\theta =$$

$$\text{Check length: } \frac{W}{4} =$$

$$\text{Is } L_e \leq \frac{W}{4} \text{ ?}$$

If the answer is *yes*, proceed. If no, consider a reduced weir length or reevaluate the use of stream barbs at this site. Toe erosion may be caused by processes other than direct streamflow.

Step 7 Locate subsequent stream barbs:

From a point on the outer end of the first stream barb, draw a line extending downstream to the point where it intercepts the bank. This projected line (7), should be parallel to the tangent line (4). Determine L_s, the distance from this point back to the point where previous stream barb intercepts the bank. If, L_s is $\leq 5 \times L_e$, then this point is a suitable location for the next stream barb. If this point is $>5 \times L_e$, consider limiting the distance to $5 \times L_e$. It is important to note that anecdotal evidence indicates that close spacing may be required in fast, high-energy streams.

Step 8 Repeat steps 4 through 6 for subsequent stream barbs. Typically the last stream barb ends near the end of the eroding section of bank or end of bend.

Step 9 Determine stream barb section properties.

$$H = \frac{1}{3} D_a = \text{ height of weir section, outer end}$$

$$H = \frac{1}{2} D_a = \text{ height of weir section, bank end}$$

$$S = \left(\frac{1}{3} \text{ to } \frac{1}{2}\right) \times 2.5 \times D_a = \text{ depth of bed key}$$

Step 10 Determine rock size per the description on rock size (TS14H–16).

Step 11 Prepare construction drawings. See figure TS14H–14, Typical construction drawing. Figure TS14H–15 shows a detail that illustrates one possibility of incorporating a rootwad into a rock stream barb.

Cost

The cost of rock stream barbs can vary considerably given availability of material, construction access, and permitting requirements. Stream barbs are often used in combination with other treatments. In general, their cost is between $2,000 and $5,000 per individual barb. Maintenance may involve replacement of materials. Monitoring should focus particularly on the area immediately below a series of stream barbs and the bank key.

Construction considerations

Instream devices like stream barbs are best constructed during low flow. Achieving a design key in depth may require dewatering, which may be accomplished with a cofferdam. If the designs include soil bioengineering or planting, either as part of the project or to stabilize the root or bank key, then appropriate planting designs also need to be considered. All stream or river design techniques should consider critical spawning and migration periods, as well as other regulatory concerns.

Conclusion

A variety of flow-changing techniques are applicable for use in stream design projects. They can provide valuable stability and habitat benefits. Stream barbs have been well received, and it is apparent these structures will continue to be a valuable tool for streambank restoration projects in NRCS. However, they do not work in all circumstances and must be designed to fit site-specific conditions.

Figure TS14H–13 Drawing and layout details

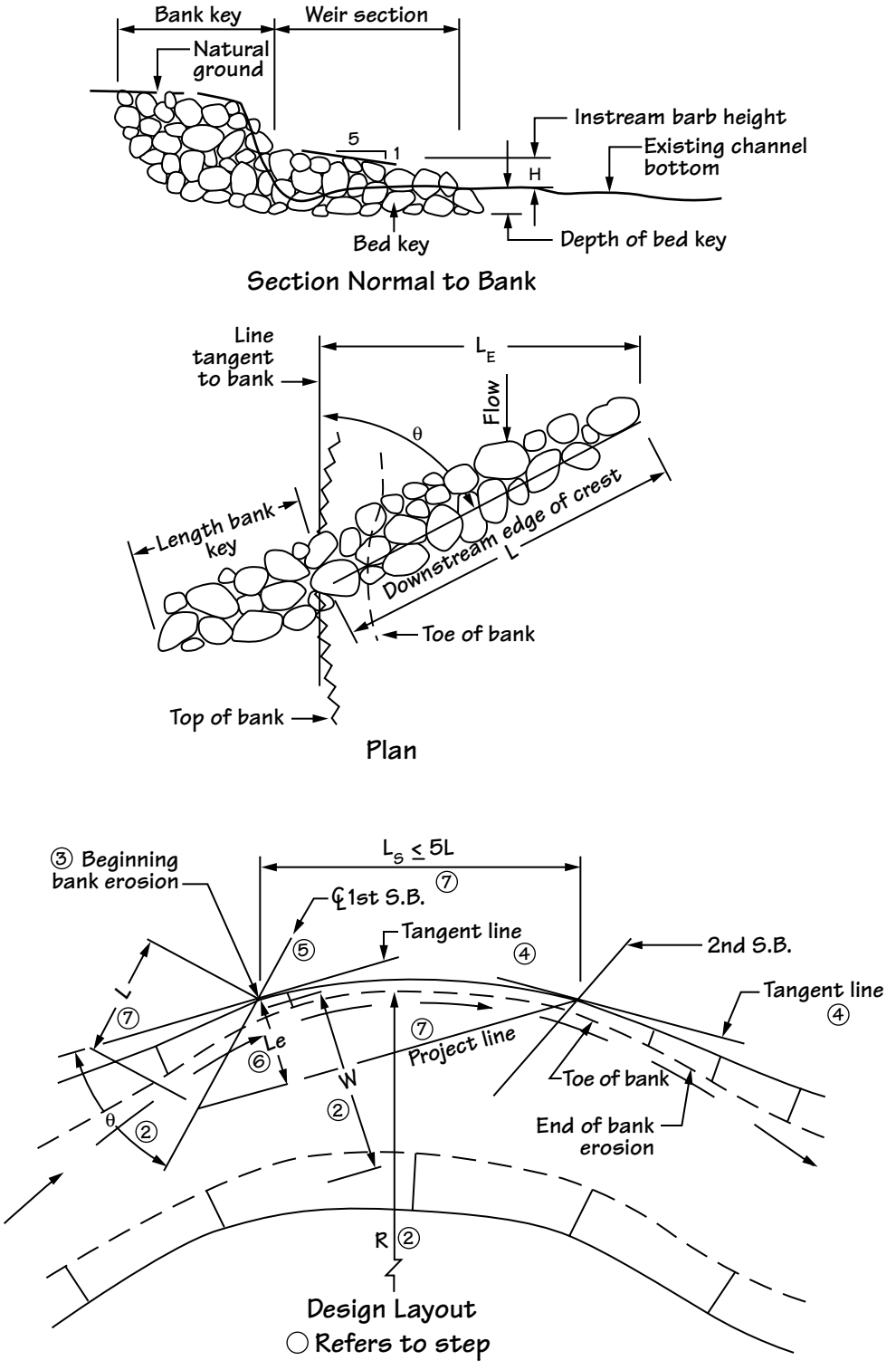


Figure TS14H–14 Typical stream barb construction drawing

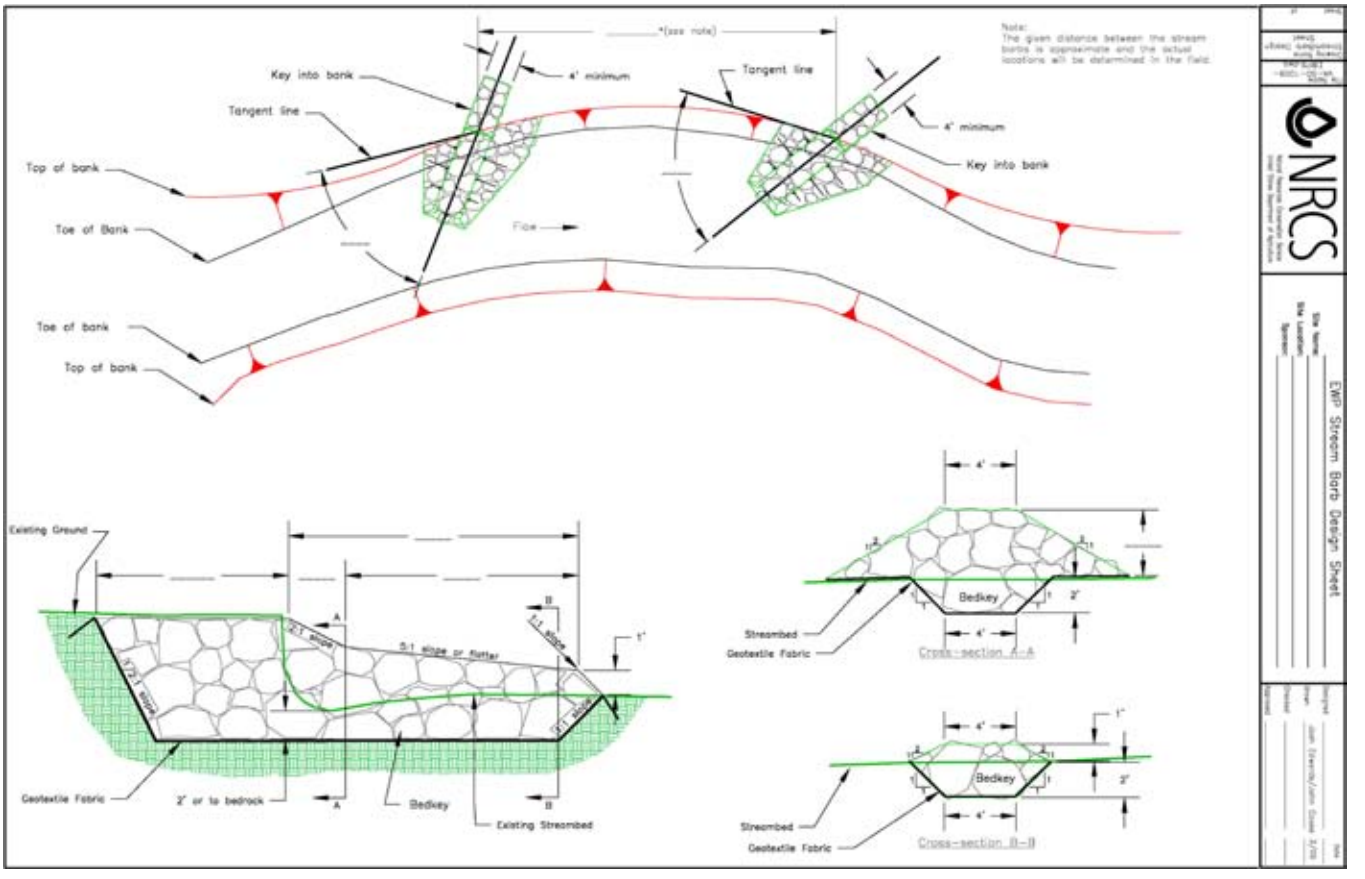
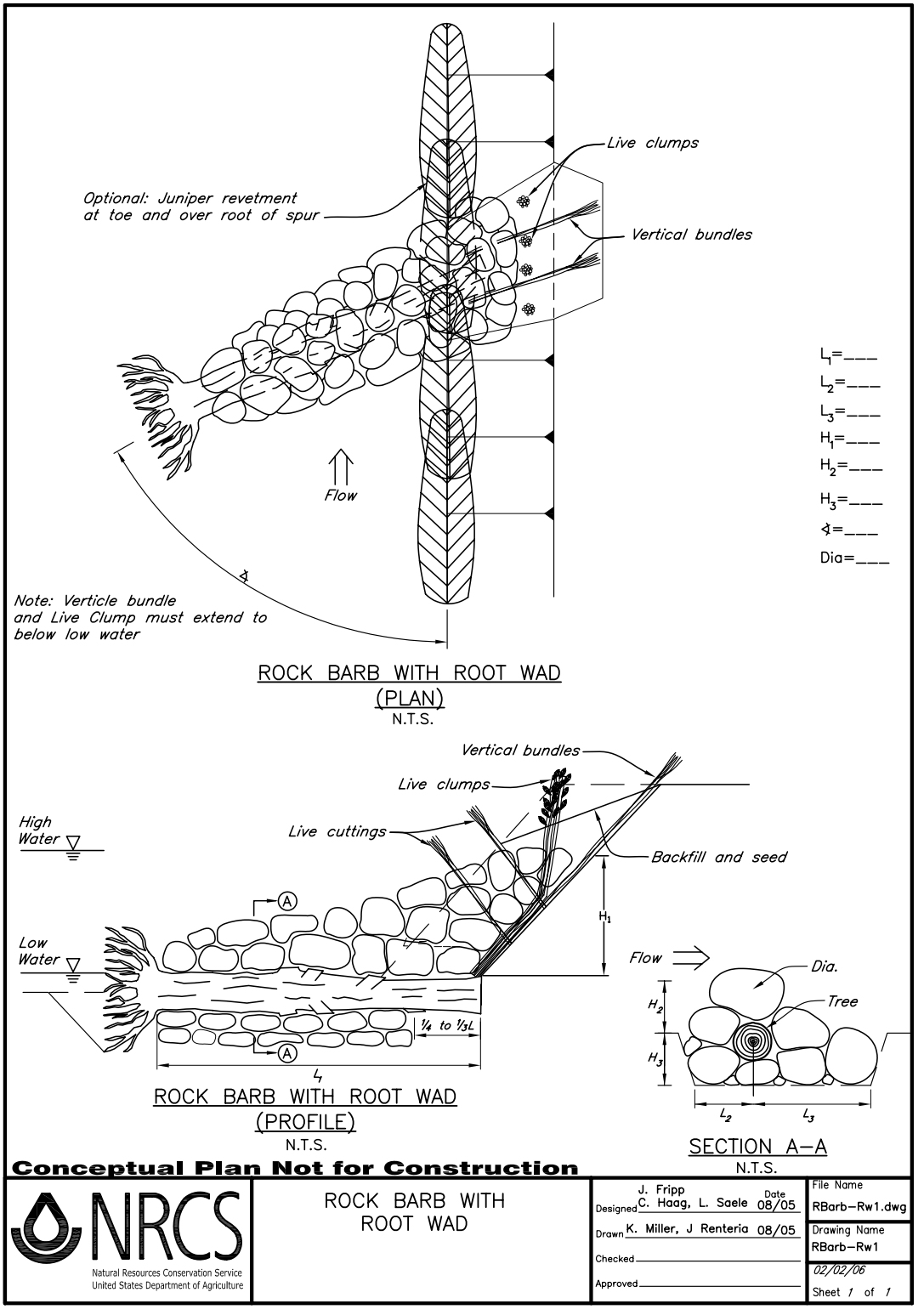


Figure TS14H–15 Detail showing the use of a rootwad incorporated into a stream barb



Technical Supplement 14I

Streambank Soil Bioengineering

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Earth materials, live and inert plant materials, and manmade materials can be used to form soil bioengineering solutions to streambank erosion problems.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14I

Streambank Soil Bioengineering

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Technical Supplement 14I

Streambank Soil Bioengineering

Purpose

Streambank soil bioengineering is defined as the use of living and nonliving plant materials in combination with natural and synthetic support materials for slope stabilization, erosion reduction, and vegetative establishment. As a result of increased public understanding and greater appreciation of the environment, many Federal, state, and local governments, as well as grassroots organizations, are actively engaged in implementing soil bioengineering treatments to stabilize streambanks. Stabilizing streambanks through the integration of natural vegetation has many advantages over using hard armor linings alone. When compared to streams with little or no vegetation on their banks, streams with well-established perennial vegetation on their banks typically have higher economic value, better water quality, and better fish and wildlife habitats. A variety of vegetative techniques are in widespread use. Many of these include soil bioengineering practices. The value of vegetation in civil engineering and the role of woody vegetation in stabilizing streambanks have gained considerable recognition in recent years (Greenway 1987; Coppin and Richards 1990; Gray and Sotir 1996). However, streambank soil bioengineering is not universally applicable. There are important considerations to take into account for their successful application and long-term sustainability. This technical supplement provides guidance for the analysis, design, installation, and maintenance of some of the most effective and commonly used soil bioengineering techniques.

Introduction

Soil bioengineering is an integrated watershed-based technology that uses sound engineering practices in conjunction with integrated ecological principles to assess, design, construct, and maintain living vegetative systems. This technology can be applied to repair damage caused by erosion and failures in the land and protect or enhance already healthy, functioning systems (Gray and Sotir 1996). Streambank soil bioengineering uses plants as structural components to stabilize and reduce erosion on streambanks. When selecting the best-suited soil bioengineering techniques, it is important to have a clear understanding of the ecological systems of the adjacent areas. Plant selection and the techniques used will play an initial role in site stabilization and, ultimately, serve as the foundation for the ecological restoration of the site. The successful establishment and long-term sustainability of herbaceous and woody plants are extremely important to the physical and biological functions of the streams and the connected watershed system.

Streambank soil bioengineering has a long history with many milestones.

- Tapestries have been found in Chinese emperor's tombs that depict Chinese peasants using willow bundles for streambank stabilization along the Yellow River in the year 28 B.C.
- In Europe, soil bioengineering techniques were used by Celtic villagers to create walls and fences.
- Romans used wattles and poles for hydro construction.
- The first written record of soil bioengineering was documented by Leonardo Da Vinci (1452–1519), where he recommended using rootable, living willow branches to stabilize agricultural irrigation channels, thus creating living streambanks. In the 16th century, streambank soil bioengineering treatments were used throughout Europe.
- In 1791, Woltmann published a soil bioengineering manual illustrating live stake techniques (Stiles 1991). In about 1800, soil bioengineers in Austria were using brush trenches to trap silt and reshape channels.
- In the 1900s, European soil bioengineers were using many of the treatments in use today (Stiles 1988).
- In 1934, Charles J. Kraebel, U.S. Forest Service, installed willow wattles above a road near Berkeley, California (Kraebel and Pillsbury 1934).
- In the late 1930s, the U.S. Department of Agriculture (USDA) Soil Conservation Service (SCS), now the Natural Resource Conservation Service (NRCS), began working on the Winooski River Watershed in Vermont after a succession of extremely damaging storm events. They used a series of soil bioengineering techniques

such as fascines, brush dams, brush mattresses, and live stakes along the Winooski River streambanks. In 1995, a detailed study of the project was completed. More than 50 out of 92 demonstration sites are still functioning today. The study found that the most successful measures generally included a mix of vegetation and mechanical treatments at each site (USDA NRCS 1999a).

After World War II, the availability of cheap energy; surplus bulldozers and dump trucks; the high cost of labor; and the advent of cheap, well-designed steel and concrete structures caused hard, inflexible structures to take over from the soil-bioengineered structures as the preferred methods for treating streambank erosion. Over the past few decades, it has become apparent that these hard structures have inherent problems that have caused a breakdown of the riparian ecosystem because of their overuse and, often, inappropriate use. A movement back to flexible and more natural streambank soil bioengineering treatments offering broader functions has come from this realization, and so has begun the modern age of soil bioengineering.

Benefits of streambank soil bioengineering

Streambank soil bioengineering has aesthetic benefits. Streambank soil bioengineering provides improved landscape and habitat values (Lewis 2000). However, most designers are interested in the specific structural benefits provided by the vegetation. Gray (1977), Bailey and Copeland (1961), and Allen (1978) describe five mechanisms through which vegetation can aid erosion control:

- reinforce the soil through the plant roots
- dampen waves or dissipate wave energy
- intercept high-water velocities
- enhance water infiltration
- deplete water in the soil profile by uptake and transpiration

Klingeman and Bradley (1976) point out four specific ways vegetation can protect streambanks.

- The root system helps hold the soil together and increases the overall bank stability by its binding network structure, that is, the ability of roots to hold soil particles together.
- The exposed vegetation (stalks, stems, branches, and foliage) increases roughness, which can increase the resistance to flow and reduce the local flow velocities, causing the flow to dissipate energy against the deforming plant, rather than the soil.
- The vegetation acts as a buffer against the abrasive effect of transported materials.
- Close-growing vegetation can induce sediment deposition by causing zones of slow velocity and low shear stress near the bank, allowing coarse sediments to deposit. Vegetation is also often less expensive than most structural methods; it improves the conditions for fisheries and wildlife, improves water quality, and can protect cultural/archeological resources.

Streambank soil bioengineering can be cost effective on local problems if applied early. Erosion areas often begin small and eventually expand to a size requiring costly traditional engineering solutions. Installation of streambank soil-bioengineered systems while the site problem is small will provide greater economic savings, minimize potential construction impacts to adjoining resources, and provide a better project. Landowners and volunteers can install many of the smaller, less complex soil bioengineering projects. The use of native, locally available plant materials and seed may provide additional savings. Costs for the vegetative materials are generally limited to labor for locating the harvesting sites, harvesting, handling and transporting to the project site, as well as the purchase of supplies (erosion control fabrics, twine, wood, and rock). Indigenous plant species are usually readily available as cuttings or rooted plants and well adapted to local climate and soil conditions. In addition, the use of indigenous materials can often have major aquatic and terrestrial habitat value. For example, plant materials can be selected to boost the habitat value by providing food and cover for birds and mammals or by providing overhanging shade to improve instream conditions for fish, waterfowl, and other aquatic life.

Streambank soil bioengineering work is often useful on sensitive or steep sites, in areas with limited ac-

cess, or where working space for heavy machinery is not feasible. Years of monitoring have demonstrated that streambank soil bioengineering systems are strong initially and grow stronger with time as vegetation becomes established.

Streambank soil bioengineering is especially useful as a transition between conventional, inert bank stabilization and the upland zone. Abrupt transitions from conventional projects, such as riprap, to the upland zone are often prone to scour attack. Established soil bioengineering treatments can act to protect and reinforce the transition and reduce the possibility of washouts and flanking.

The structural benefits of soil bioengineering are varied. Initially, the systems offer mechanical support by controlling soil movement. Over time, the root systems from the establishing woody and herbaceous species increase the strength and structure of the soil. They create a strong and dense matrix of large anchor and small feeder roots that resist streambank erosion forces. They are capable of growing when they are broken off or partially uprooted by high water velocities. They capture nutrients, remove nitrogen and phosphorous from the soil, and trap and retain pollutants, thus improving water quality. In addition, if the plant species and measures are appropriately chosen, the entire project becomes self-supporting through the native invasion of the surrounding plant community. Vegetation improves the hydrology and mechanical stability of slopes through root reinforcement and surface protection. The reinforced soil mantle acts as a solid mass, reducing the possibility of slips and displacements (USDA NRCS 1996b). Even if plants die, roots and surface organic litter continue to play an important role during reestablishment of other plants. Once plants are established, root systems reinforce the soil mantle and remove excess moisture from the soil profile. Often, this is the key to long-term soil stability.

Aboveground biomass is also important because it provides roughness along the stream channel that reduces stream velocities and allows sediment to drop out. This aboveground biomass is a buffer along the stream channel that provides numerous benefits. This buffer increases water infiltration by slowing the flow, provides protection to the streambank by lying down as the high water flows past, provides fish and wildlife habitat, and traps sediment (Eubanks and Meadows 2002). The aboveground biomass is flexible and functions to absorb and reduce the energy along the streambank during high flows. By comparison, hard, rigid structures tend to be inflexible and deflect energy.

Riparian areas

Riparian areas are the zones along streams and rivers that serve as interfaces between terrestrial and aquatic ecosystems. The highly saturated soils in these zones are home to many species of water-loving flora and fauna. Riparian areas are important because they:

- provide erosion control by regulating sediment transport and distribution
- enhance water quality
- produce organic matter for aquatic habitats
- provide fish and wildlife habitat
- act as indicators of environmental change
- are among the most diverse, dynamic, complex biological systems on Earth

Riparian areas are shaped by the dynamic forces of water flowing across the landscape. Flooding, for instance, is a natural and necessary component of riparian areas. Many riparian plant species, such as cottonwood, require floods to regenerate by seed. Geomorphological characteristics of the stream valley, such as flood plain level (connectivity), drainage area, stream capacity, channel slope, and soils, are some of the factors that influence the frequency, duration, and intensity of flooding (Leopold, Wolman, and Miller 1964). In turn, flooding and related sediment transport processes influence the size and structure of the stream channel and composition of the riparian vegetation (Hupp and Osterkamp 1996).

Riparian health and streambank stability are simply a reflection of the conditions in the surrounding landscape. Healthy streams and riparian areas are naturally resilient, allowing recovery from natural disturbances such as flooding (Florsheim and Coats 1997). Streambank stability is a function of a healthy riparian and upland watershed area. When stream and riparian systems are degraded, this resiliency to natural disturbances is diminished. Excessive flooding, erosion in the form of

downcutting and widening, and associated sedimentation often will increase, creating a loss of physical and biological equilibrium in the stream corridor.

Riparian planting zones

Success of streambank soil bioengineering treatments depends on the initial establishment and long-term development of riparian plant species. The success of the plants, in turn, depends on numerous factors including:

- species selected
- procurement methods
- installation and handling techniques
- time of year
- soil compaction
- soil type
- nutrients
- salinity
- ice
- sediment
- debris load
- flooding
- accessibility to water
- drought
- hydrology
- climate
- location relative to the stream

It is important to note the location and types of existing vegetation in and adjacent to the project area. The elevation and lateral relationships to the stream can be described in terms of riparian planting zones. Proposed streambank soil bioengineering techniques should also be assessed and designed in terms of the location of the plants relative to the stream and water table. These riparian planting zones can be used to determine where riparian species should be planted in relation to the waterline during different periods of flow. Figure TS14I–1 illustrates an idealized depiction of riparian planting zones (Riparian/Wetland Project Information Series No. 16).

Some of these zones identified in figure TS14I–1 may be absent in some stream systems (Hoag and Landis 1999). Sections that have missing zones will be especially prevalent in streams in the American Southwest, as well as areas that have been impacted by development. Before working on a streambank stabilization project, local experts should be consulted to determine which zones are present. Following is a brief description of each zone.

Toe zone—This zone is located below the average water elevation or baseflow. The cross-sectional area at this discharge often defines the limiting biologic condition for aquatic organisms. Typically, this is the zone of highest stress. It is vitally important to the success of any stabilization project that the toe is stabilized. Due to long inundation periods, this zone will rarely have any woody vegetation. Some areas of the Southwest, however, will have woody vegetation. Often riprap or another type of inert protection is required to stabilize this zone.

Bank zone—The bank zone is located between the average water elevation and the bankfull discharge elevation. While it is generally in a less erosive environment than the toe zone, it is potentially exposed to wet and dry cycles, ice scour, debris deposition, and freeze-thaw cycles. The bank zone is generally vegetated with early colonizing herbaceous species and flexible stemmed woody plants such as willow, dogwood, elderberry, and low shrubs. Sediment transport typically becomes an issue for flows in this zone, especially for alluvial channels.

Bankfull channel elevation—Bankfull stage is typically defined at a point where the width-to-depth ratio is at a minimum. Practitioners use other consistent morphological indices to aid in its identification. Often, the flow at the bankfull stage has a recurrence interval of 1.5 years. Due to the high velocities and frequent inundation, some high risk streambank soil bioengineering projects frequently incorporate hard structural elements, such as rock, below this elevation. Where there is a low tolerance for movement, many projects rely on inert or hard elements in this zone.

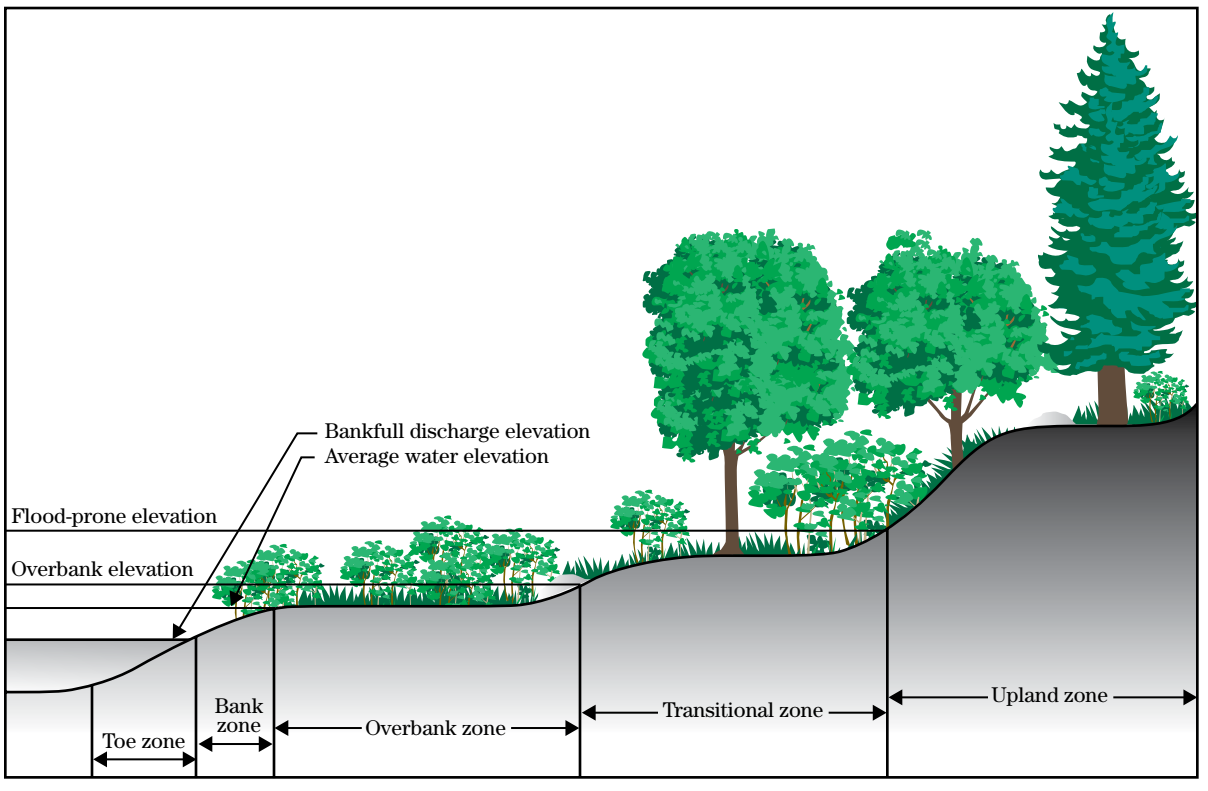
Bankfull flow is often considered to be synonymous with channel-forming discharge in stable channels and is used in some channel classification systems, as well as for an initial determination of main channel dimensions, plan, and profile. In many situations, the channel velocity begins to approach a maximum at bankfull stage. In some cases, on wide, flat flood plains, channel velocity can drop as the stream overtops its bank and the flow spills onto the flood plain. In this situation, it may be appropriate to use the bankfull hydraulic conditions to assess stability and select and design streambank protection. However, when the flood plain is narrower or obstructed, channel velocities may continue to increase with rising stage. As a result, it may also be appropriate to use a discharge greater than bankfull discharge to select and design streambank protection treatments. A further description of bankfull discharge is provided in NEH654.05.

Overbank zone—This zone is located above the bankfull discharge elevation. This typically flat zone may be formed from sediment deposition. It is sporadically flooded, usually about every 2 to 5 years. Vegetation found in this zone is generally flood tolerant and may have a high percentage of hydrophytic plants. Shrubby willow with flexible stems, dogwoods, alder, birch, and others may be found in this zone. Larger willows, cottonwoods, and other trees may be found in the upper end of this zone.

Transitional zone—The transitional zone is located between the overbank elevation and the flood-prone elevation. This zone may only be inundated every 50 years. Therefore, it is not exposed to high velocities except during high-water events. Larger upland species predominate in this zone. Since it is infrequently flooded, the plants in this zone need not be especially flood tolerant.

Upland zone—This zone is found above the flood-prone elevation. Erosion in this zone is typically due to overland water flow, wind erosion, improper farming

Figure TS14I–1 Riparian plant zones indicate where different riparian plant species should be planted



practices, logging, development, overgrazing, and urbanization. Under natural conditions the upland zone is typically vegetated with upland species.

Defining and managing risks

Streambank soil bioengineering offers a broad-based approach to solving many stream problems. However, it is not appropriate for all sites and situations and may offer a higher level of risk than conventional structures such as sheet pile or riprap. While NEH654.02 addresses risk in detail, some particular issues related to streambank soil bioengineering are described in this section.

The use of plants in a project may present problems. Those problems include failure to survive and grow, vulnerability to drought and flooding, timing of the installation, impact of soil nutrient and sunlight deficiencies on establishment success and growth, uprooting by freezing and thawing, damage by ice and debris, impact of undermining currents, damage by wildlife and livestock feeding or trampling, and need for special management measures to ensure long-term project success (modified from Allen and Leech 1997). Many of these problems can be resolved through careful planning and integration of other technologies. If care is taken in planning and design, vegetation often survives well under adverse conditions due to its flexibility and self-repairing capabilities. Some example modifications to features of a streambank soil bioengineering project are shown in table TS14I–1.

Table TS14I–1 Design modifications to account for site conditions

Issue	Concern	Possible action or design modification
Duration of inundation	Some plant species and soils cannot withstand long flooding duration	• Choose plant materials that can withstand long inundation such as willow, dogwood, or elderberry, which can withstand 1 to 6 months of inundation • Use or combine inert or soil reinforcement material in areas of prolonged inundation
Susceptibility of plant materials to disease or insects	Loss of plants could endanger the project	• Use a diversity of species in the plant mix so that the loss of one or two species will not endanger the entire treatment area • Monitor the installation regularly for the first year or two during the establishment period • Apply a fungicide or insecticide as needed to promote healthier growth
Excessive velocity	High velocities could damage or destroy the project	• Compare estimated velocity and/or shear thresholds at site to recommendations for limiting velocity and shear when selecting project type and method of repair
Increased resistance to flood levels	Increased roughness resulting from project may result in more frequent out-of-bank flows	• Choose plant material that remains supple. Avoid plant material that will be tree-like and form an obstruction to the flow • Install the vegetative treatment further up on the bank • Coordinate possible affects with flood plain regulatory authorities • Excavate floodway to account for lost conveyance
Predation by herbivores	Loss of plants could endanger the project	• Fence the project area • Fence planting areas within the project • Surround the area with vegetation that the expected herbivores do not eat • Choose plant material that they typically do not eat—thorny or otherwise unappetizing to the expected herbivores

Projects which are referred to as streambank soil bioengineering can range from those that rely almost solely on plant material to those that primarily rely on inert material to provide bank strength. A project that relies primarily on inert or hard material will be less flexible than a project that relies more on plant material for its strength. Thus, the acceptable level of risk, as well as the tolerance for additional movement at the project areas, will generally steer the project selection. Table TS14I–2 provides a general discussion of streambank stabilization project and tolerance for movement.

Determining appropriateness of treatments

Streambank soil bioengineering offers an excellent approach to solving many stream problems. However, as with any technology, it is not appropriate for all sites and situations. NEH654.03 describes site investigations that can be used to assess site characteristics. The successful application of streambank soil bioengineering presents some additional questions that should be considered before starting to work in the stream (table TS14I–3 (modified from Wells 2002)).

Limiting velocity and shear criterion

The effects of the water current on the stability of any streambank protection treatment must be considered. This evaluation includes the full range of flow conditions that can be expected during the design life of the project. Two approaches that are commonly used to express the tolerances are allowable velocity and allowable shear stress. While these two hydraulic parameters are briefly described in this technical supplement, the reader should also review NEH654.06 for more information.

Flow in a natural channel is governed in part by boundary roughness, gradient, channel shape, obstructions, and downstream water level. If the project represents a sizable investment, it may be appropriate to use a computer model such as the U.S. Army Corps of Engineers (USACE) HEC–RAS computer program (USACE 1995a) to assess the hydraulic conditions. However, if a normal depth approximation is applicable, velocity can be estimated with Manning's equation. It is important to note that this estimate will be an average channel velocity. In some situations, the velocity along the outer bank curves may be considerably larger.

Table TS14I–2 Relationships between type of streambank stabilization project and type of site

Site description	Tolerance for movement	Type of project
Eroding streambank threatening a home or municipal sewage treatment plant	None—streambank must be made static	Relies primarily on hard or inert structures, but may include a vegetative component for adjunctive support, environmental, and aesthetic benefits
Eroding streambank adjacent to a secondary road	Slight—road must be protected for moderate storms, but some movement is allowed	Rely on streambank soil bioengineering measures that incorporate hard or inert components
Eroding streambank threatening hiking trails in a park	Moderate—a natural system is desired, but movement should be slowed	May rely entirely on vegetative protection, but more likely on streambank soil bioengineering measures that incorporate some hard or inert components
Eroding streambank in rangeland	Relatively high—but erosion should be reduced	Rely on fencing, plantings, or streambank soil bioengineering measures—perhaps ones that incorporate some hard or inert components in areas that have suffered significant damage
Erosion on a wild and scenic stream system	High—but erosion should be reduced	Do nothing or rely on plantings and vegetative streambank soil bioengineering measures

Table TS14I–3 Questions to ask before starting a streambank soil bioengineering project

Question	Issue
What is the land use conversion trend for the drainage area?	Past and future land use conversion significantly alters hydrology. Streambank protection measures of any kind may not be successful because of high stresses created by changing hydrologic conditions. The watershed, as well as the site, should be investigated. Designs should consider the effects of potentially new or altered flow, as well as sediment conditions in the watershed
Is a management plan in place and being maintained?	Locally, determine the land use in the immediate area of the site and whether the landowner has a working management plan in place. In some cases, changing the management plan (livestock grazing plan, proper farming techniques, buffer width, conservation logging techniques) may be all that is needed to allow the stream to recover on its own. This is the least expensive alternative and may have less overall impact on the stream. However, if the impact is from upstream development, this approach may have a negative impact because the erosion will continue
Is the purpose of the streambank soil bioengineering project to protect critical structures such as a home, business, or manufacturing site?	In an emergency situation, select soil bioengineering treatments that incorporate sound engineering design components into the overall design. In this case, hard or inert structures (rock, geogrid) are necessary. The use of a soil bioengineering solution can significantly improve surface protection, internal reinforcement strength, aesthetics, habitat, and water quality benefits (table TS14I–2)
Are both sides of channel unstable?	This condition may indicate that the channel is incised or that a large-scale adjustment is occurring in the stream channel, possibly from a systemwide source. These conditions can generate flash flooding, excessive velocity, and shear stress, making it difficult to establish any solution until the correct cross-sectional area and planform has been established. For more information, refer to the channel evolution model (Simon 1989) and NEH654.03
Is the channel grade stable?	If the channel bed is downcutting, any bank treatment may be ineffective without some measure taken to stabilize the grade. Headcuts, overfalls, and nickpoints are indicators of unstable channel grades (NEH654 TS14G)
Is local scour on the bends an issue?	Any bank treatment may be ineffective unless toe and bed protection can be provided below the anticipated scour depth. Depending on the event (1-, 2-, 5-, 10-year event), a general rule of thumb is to add 2 to 5 feet to the deepest depth of water at the eroding outside meander bends. This will be a rough indication of potential scour depth. Specialists need to be involved in the assessment, design, and installation (NEH654 TS14B)
What is the bank height?	When the bank is high, slope stability factors typically add complexity to the design and need to be analyzed, designed, and installed by specialists such as geotechnical engineers. The bank height generally becomes an issue above 6 feet
What is the velocity of the stream at design flows?	The ability of soil bioengineering measures to protect a streambank in part depends on the force that the water exerts on the boundary during the design event. When velocity (or shear) forces exceed a threshold for the type of treatment being considered, other measures or materials may be required in conjunction with the treatment to ensure stability. More details on this important issue are presented later in this document
What is the depth of the water?	Most woody plant species do not grow in standing water. The level and durations of frequent flooding (every 1 to 2 years) will help determine the elevation needed for toe protection and vegetative components

Table TS14I–3 Questions to ask before starting a streambank soil bioengineering project—Continued

Question	Issue
Is a noncohesive soil layer present in the slope?	Noncohesive soil layers may require special design measures. The lower in the slope the weak layer occurs, the more comprehensive the design will need to be to stabilize the bank (NEH654 TS14A)
Is bank instability due to piping or ground water sapping?	Soil bioengineering measures can assist in controlling piping and sapping. An intensive investigation into the reason for the streambank erosion is important to ensure that the actual cause is treated, instead of a symptom (NEH654 TS14A)
Will mature vegetation adversely affect the stream hydraulics?	Changes in flood elevations due to flow resistance on vegetative banks may not be allowable in some settings. This is especially true in urban areas where the stream channel is narrow and flood plains are limited
Is there a stable bank to tie into at each end of the treatment area?	Any streambank protection measure is susceptible to flanking if it is not properly tied into stable points. It is important that both the upstream and downstream ends of the treatment are well keyed-in and protected
What site conditions may inhibit plant growth?	Soil tests are recommended to determine the presence of plant establishment opportunities. Soil texture, restrictive layers, and limiting factors (pH, salts, calcic soils, alkalinity) should be evaluated. The amount and seasonal availability of water, regional extremes in temperature, wind (affects growth and survival, desiccation), and microclimate (cold pockets, solar radiation pockets, wind turbulence, and aspect) are also significant factors to be considered. In many situations, these issues may be overcome by installing native plant materials that grow in or near the area
Is there anything in the stream water or surface runoff that will inhibit plant growth?	Adverse water quality can inhibit plant growth. Check any stream monitoring records for possible problems, and investigate the watershed for sources of potential contaminants. In some cases, the use of plant materials will improve water quality
Will the site be shaded during the growing season?	If the site will be shaded, choose plant species that tolerate and thrive in shade conditions
Is there significant surface runoff from above the streambank?	Identify sources of surface runoff during the site inventory. In some cases, a diversion or waterway may need to be installed to control runoff and erosion. In other cases, vegetated soil bioengineering filter strips or constructed wetland systems can be designed to intercept and treat the water before it enters the stream
Are beaver, muskrats, moose, elk, or deer present in the area?	Browsing animals can damage the vegetation used in soil bioengineering treatments. If these animals are in the area, special precautions may be required to ensure the installations are able to establish. This is especially important during the first growing season. If established during the first year, they will continue to grow and survive. Typically, temporary plant protection measures are all that is needed
Are adequate plant materials available from the natural surrounding area or from local nurseries?	Soil bioengineering techniques require large quantities of plant materials. Locating an adequate nursery or harvesting source of plant material is essential to the success of the project. This source should be as close as possible to the site to ensure that adapted plants are used. Plants can be harvested at higher elevations and brought down to lower elevations, but do not take materials from low elevations and move them to higher elevations (Hoag 1997)
Are invasive species present in the area?	Aggressive invasive species may out-compete the soil bioengineering species and make it difficult for them to get established. It is necessary to eradicate the invasive species prior to the soil bioengineering installation

The average shear stress exerted on a channel boundary can be estimated with the equation provided below, assuming the flow is steady, uniform, and two-dimensional.

$$\tau_0 = \gamma R S_f \qquad \text{(eq. TS14I–1)}$$

where:

τ_0 = average boundary shear (lb/ft^2)
γ = specific weight of water (62.4 lb/ft^3)
R = hydraulic radius (A/P, but can be approximated as depth in wide channels)
S_f = friction slope (can be approximated as bed slope)

The local maximum shear can be up to 50 percent greater than the average shear in straight channels and larger along the outer banks of sinuous channels. Temporal maximums may also be 10 to 20 percent larger, as well. More information on the calculation of this hydraulic parameter is presented in NEH654.08.

Recommendations for limiting velocity and shear vary widely (table TS14I–4). Not all techniques presented in this technical supplement are noted in this table. However, the designer can compare techniques with similar attributes to those listed in the table to estimate the limiting shear.

The designer should proceed cautiously and not rely too heavily on these values. Judgment and experience should be weighed with the use of this information. The recommendations in table TS14I–4 were empirically determined and, therefore, are most applicable to the conditions in which they were derived. The recom-

Table TS14I–4 Compiled permissible shear stress levels for streambank soil bioengineering practices

Practice	Permissible shear stress (lb/ft²)*	Permissible velocity (ft/s)*
Live poles (Depends on the length of the poles and nature of the soil)	Initial: 0.5 to 2 Established: 2 to 5+	Initial: 1 to 2.5 Established: 3 to 10
Live poles in woven coir TRM (Depends on installation and anchoring of coir)	Initial: 2 to 2.5 Established: 3 to 5+	Initial: 3 to 5 Established: 3 to 10
Live poles in riprap (joint planting) (Depends on riprap stability)	Initial: 3+ Established: 6 to 8+	Initial: 5 to 10+ Established: 12+
Live brush sills with rock (Depends on riprap stability)	Initial: 3+ Established: 6+	Initial: 5 to 10+ Established: 12+
Brush mattress (Depends on soil conditions and anchoring)	Initial: 0.4 to 4.2 Established: 2.8 to 8+	Initial: 3 to 4 Established: 10+
Live fascine (Very dependent on anchoring)	Initial: 1.2 to 3.1 Established: 1.4 to 3+	Initial: 5 to 8 Established: 8 to 10+
Brush layer/branch packing (Depends on soil conditions)	Initial: 0.2 to 1 Established: 2.9 to 6+	Initial: 2 to 4 Established: 10+
Live cribwall (Depends on nature of the fill (rock or earth), compaction and anchoring)	Initial: 2 to 4+ Established: 5 to 6+	Initial: 3 to 6 Established: 10 to 12
Vegetated reinforced soil slopes (VRSS) (Depends on soil conditions and anchoring)	Initial: 3 to 5 Established: 7+	Initial: 4 to 9 Established: 10+
Grass turf—bermudagrass, excellent stand (Depends on vegetation type and condition)	Established: 3.2	Established: 3 to 8
Live brush wattle fence (Depends on soil conditions and depth of stakes)	Initial: 0.2 to 2 Established: 1.0 to 5+	Initial: 1 to 2.5 Established: 3 to 10
Vertical bundles (Depends on bank conditions, anchoring, and vegetation)	Initial: 1.2 to 3 Established: 1.4 to 3+	Initial: 5 to 8 Established: 6 to 10+

* (USDA NRCS 1996b; Hoag and Fripp 2002; Fischenich 2001; Gerstgrasser 1999; Nunnally and Sotir 1997; Gray and Sotir 1996; Schiechtl and Stern 1994; USACE 1997; Florineth 1982; Schoklitsch 1937)

mendations must be scrutinized and modified according to site-specific conditions such as duration of flow, soils, temperature, debris and ice load in the stream, plant species, as well as channel shape, slope and planform. Specific cautions are also noted in the table. However, there are anecdotal reports that mature and established practices can withstand larger forces than those indicated in table TS14I–4.

Plants for soil bioengineering

Consult local expertise and guidelines when selecting the appropriate plant material. Where possible, it is best to procure harvested cuttings from areas that are similar in their location, relative to the stream. Installation will be most successful where the soil, site, and species match a nearby stable site. Harvest three or more species from three to five different locations.

Woody plants

Adventitiously rooting woody riparian plant species are used in streambank soil bioengineering treatments because they have root primordia or root buds along the entire stem. When the stems are placed in contact with soil, they sprout roots. When the stem is in contact with the air, they sprout stems and leaves. This ability to root, independent of the orientation of a stem, is a reproductive strategy of riparian plants that has developed over time in response to flooding, high stream velocities, and streambank erosion.

Many woody riparian plant species root easily from dormant live cuttings. They establish quickly and are fast-growing plants with extensive fibrous root systems. These plants are typically hardy pioneer species that can tolerate both inundation and drought conditions. The keystone species that meet these criteria are willows, cottonwoods, and shrub dogwoods. These traits allow their use in treatments such as fascines, brush mattress, brush layer, and pole cuttings. Typically, the most consistently successful rooting plants are the willow (*Salix* spp.). Data from projects nationwide indicate that shrub willows root successfully on average 40 to 100 percent of the time. Shrub dogwoods (*Cornus* spp.), on the other hand, are more variable in their rooting success, ranging from 10 to 90 percent, but more typically averaging in the 30 to 60 percent range. Rooting success of both willows and dogwoods can be affected by the timing of planting, age of the material used, handling and storage, installation procedures, and placement in the proper hydrologic regime on the streambank.

Cottonwoods and poplars (*Populus* spp.) have also been used successfully in streambank soil bioengineering. However, typical riparian species such as birches (*Betula* spp.) and alders (*Alnus* spp.) do not root well from unrooted hardwood cuttings; therefore, they are not suitable for certain soil bioengineering techniques such as poles or live stakes. They are, however, useful as rooted plant stock for many soil bioengineering measures including hedgelayers, branch packing, cribwalls, vegetated reinforced soil slopes, and live siltation construction. Additionally, these and other species can be included in a riparian seed mix or installed as rooted plants as part of the stream and riparian restoration. In some cases, a pilot study will allow wise selection of some nonstandard plant materials by testing how effectively locally available genotypes are adapted to soil and hydrologic conditions on site. Table TS14I–5 lists a number of woody species which are applicable to many of the techniques described.

Willow (*Salix* spp.)—Willows used in soil bioengineering systems are analogous to annual or short-lived perennial grasses in a seed mixture (nurse or companion crop) (figs. TS14I–2 and 14I–3). They provide a quick pioneer plant cover for soil protection. Their longevity depends on the region of the country and specific site conditions. In sunnier, more open sites or in more arid climates, willows may persist for decades. In the Northeast, willows are generally an early successional pioneer species and will decline and yield to the natural invasion of other species as shade (5 hours or less per day) develops on the site. In all cases, they prefer damp soils.

Some species develop roots from many locations along the stem, known as suckering, but some do not sucker at all. Plants are either male or female and are easily propagated asexually, thus allowing for the use of male, nonsuckering plants to avoid spreading if desired.

Dogwood (*Cornus* spp.)—Species include gray, redosier, roughleaf, alternate leafed, and silky (fig. TS14I–4). All are multistemmed shrubs that are valu-

Table TS14I–5 Woody plants with very good to excellent ability to root from dormant, unrooted cuttings and their soil bioengineering applications

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Species with very good to excellent rooting ability from live hardwood material					
Populus balsamifera	Balsam poplar	Local collections	1,2,3,4,5,8,9,0,A	Very good	Live cuttings, poles
Populus deltoids	Eastern cottonwood	Local collections	1,2,3,4,5,6,7	Very good	Poles, live cuttings
Populus balsamifera ssp *trichocarpa*	Black cottonwood	Local collections	4,8,9,0,A	Very good	Poles, live cuttings
Salix alaxensis	Feltleaf willow	Local collections	A	Very good	Poles, live cuttings
Salix amygdaloides	Peachleaf willow	Local collections	1,2,3,4,5,6,7,8,9	Very good	Poles, posts, live cuttings
Salix barclayi	Barclay's willow	Local collections	A	Very good	Poles, posts, live cuttings
Salix brachycarpa	Barren Ground willow	Local collections	A	Very good	Poles, posts, live cuttings
Salix boothii	Booth's willow	Local collections	7,8,9,0	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix cottetii	Bankers' Dwarf willow (cultivar)	Nursery	Introduced 1,2,3	Very good	Fascines, brush mattress, brush layering, live cuttings
Salix discolor	Pussy willow	Local collections	1,2,3,4,9	Very good	Fascines, poles, brush layering, live cuttings
Salix drummondiana	Drummond's willow	Local collections	7,8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix interior	'Greenbank' Sandbar willow (cultivar)	Nursery	1,3,4,5	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix interior	Sandbar willow	Local collections	1,3,4,5	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix melanopsis	Coyote willow (green stem)	Local collections	8,9,0	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix eriocephala	Missouri River willow	Local collections	7,8,9,0	Very good	Fascines, poles, brush layering, live cuttings
Salix fluviatilis	'Multnomah' River willow (cultivar)	Nursery	9 (Coast only)	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix fluviatilis	River willow	Local collections	9	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix geyeriana	Geyer willow	Local collections	7,8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix gooddingii	'Goodding's willow	Local collections	6,7,8,0	Very good	Poles, posts, live cuttings
Salix hookeriana	Clatsop' Hooker willow (cultivar)	Nursery	9, 0 (Coast only)	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings

Table TS14I–5 Woody plants with very good to excellent ability to root from dormant, unrooted cuttings and their soil bioengineering applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Salix hookeriana	Hooker willow	Local collections	9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix laevigata	Red willow	Local collections	7,8,9,0	Very good	Poles, live cuttings
Salix lasiolepis	'Rogue' Arroyo willow (cultivar)	Nursery	9,0	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lasiolepis	Arroyo willow	Local collections	6,7,8,9,0	Very good	Poles, live cuttings
Salix lemmonii	'Palouse' Lemmon's willow (cultivar)	Nursery	8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lemmonii	Lemmon's willow	Local collections	8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix eriocephala spp. *ligulifolia*	'Placer' Erect willow (cultivar)	Nursery	9,0 (Coast only)	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix ligulifolia	Strapleaf willow	Local collections	8,9,0	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lucida ssp. *lasiandra*	'Nehalem' Pacific willow (cultivar)	Nursery	9 (Coast only)	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lucida ssp. *lasiandra*	Pacific willow	Local collections	7,8,9,0,A	Excellent	Poles, live cuttings
Salix pentandra	'Aberdeen Selection' Laurel willow (cultivar)	Nursery	Introduced 8,9,0	Excellent	Poles, live cuttings
Salix purpurea	'Streamco' Purpleosier willow (cultivar)	Nursery	Introduced 1,2,3	Excellent	Fascines, brush mattress, brush layering, live cuttings
Salix sericea	'Riverbend Germplasm' Silky willow (cultivar)	Nursery	1,2,3	Excellent	Fascines, poles, brush mattress, brush layering, live cuttings
Salix sericea	Silky willow	Local collections	1,2,3	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix sitchensis	'Plumas' Sitka willow (cultivar)	Nursery	9,0 (Coast only)	Very good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix sitchensis	Sitka willow	Local collections	9,0,A	Very good	Fascines, brush mattress, brush layer
Sambucus nigra ssp. *anadensis*	Common elderberry	Local collections	1,2,3,4,5,6,8,0,A	Very good	Fascines, brush mattress, brush layering, live cuttings

Table TS14I–5 Woody plants with fair to good ability to root from dormant, unrooted cuttings and their soil bioengineering applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Species with fair to good rooting ability from live hardwood material					
Baccharis pilularis	'Coyote' brush	Local collections	7,9,0	Fair	Fascines, brush mattress, brush layering, live cuttings
Baccharis salicifolia	Mule's Fat	Local collections	6,7,8,0	Fair	Fascines, brush mattress, brush layering, live cuttings
Cephalanthus occidentalis	'Keystone' Common Button-bush (cultivar)	Nursery	1,2,3,5,6,7,0	Good	brush mattress, brush layering, Fascines
Cephalanthus occidentalis	Common buttonbush	Local collections	1,2,3,5,6,7,0	Fair	brush mattress, brush layering, Fascines
Cornus amomum	'Indigo' Silky dogwood (cultivar)	Nursery	1,2,3,4,5,6	Good	Fascines, brush mattress, brush layering, live cuttings
Cornus amomum	Silky dogwood	Local collections	1,2,3,4,5,6	Fair	Fascines, brush mattress, brush layering, live cuttings
Cornus sericea	'Ruby' Redosier dogwood (cultivar)	Nursery	1,3,4,5,7,8,9,0,A	Good	Fascines, brush mattress, brush layering, live cuttings
Cornus sericea	Redosier dogwood	Local collections	1,3,4,5,7,8,9,0,A	Fair	Fascines, brush mattress, brush layering, live cuttings
Cornus sericea ssp. *occidentalis*	'Mason' Western Redosier dogwood (cultivar)	Nursery	9,0 (Coast only)	Good	Fascines, brush mattress, brush layering, live cuttings
Cornus sericea ssp. *occidentalis*	Western Redosier dogwood	Local collections	9,0,A	Good	Fascines, brush mattress, brush layering, live cuttings
Lonicera involucrate	Black Twinberry	Local collections	3,7,8,9,0,A	Fair	Fascines, brush layering, live cuttings
Philadelphus lewisii	'Lewis' Mock-orange	Local collections	9,0	Fair	Fascines, live cuttings
Physocarpus capitatus	Pacific ninebark	Local collections	9 (Coast only)	Fair	Fascines, brush layering, live cuttings
Physocarpus opulifolius	Common ninebark	Local collections	1,2,3,4,5	Fair	Fascines, brush mattress, brush layering, live cuttings
Populus angustifolia	Narrowleaf cottonwood	Local collections	4,5,6,7,8,9,0	Fair	Poles, live cuttings
Populus fremontii	Fremont cottonwood	Local collections	6,7,8,0	Fair	Poles, live cuttings
Rubus spectabilis	Salmonberry	Local collections	8,9,0	Fair	Fascines, live cuttings
Salix alba	White willow	Local collections	introduced 1,2,3,4	Fair	Poles, posts, live cuttings
Salix bebbiana	Bebb willow	Local collections	1,3,4,5,7,8,9,0,A	Fair	Poles, live cuttings

Table TS14I–5 Woody plants with fair to good ability to root from dormant, unrooted cuttings and their soil bioengineering applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Salix humilis	Prairie willow	Local collections	1,2,3,4,5,6	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix drummondiana	'Curlew' Drummond's willow (cultivar)	Nursery	7,8,9,0	Good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix drummondiana	Drummond's willow	Local collections	7,8,9,0	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix exigua	'Silvar' Coyote willow (cultivar)	Nursery	6,7,8,9,0,A	Good	Fascines, poles, brush mattress, brush layering, live cuttings
Salix exigua sp interior	Sandbar willow (grey stem)	Local collections	6,7,8,9	Fair	Fascines, live cuttings, poles, brush mattress, brush layering
Salix lucida	Shining willow	Local collections	1,3,4,5,7,8,9,0,A	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix lutea	Yellow willow	Local collections	4,5,7,8,9,0	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix nigra	Black willow	Local collections	1,2,3,5,6	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix planifolia	Plainleaf willow	Local collections	1,3,4,5,7,8,9,0,A	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Salix prolixa	'Rivar' Mackenzie's willow (cultivar)	Nursery	8,9,0,A	Good	Poles, live cuttings
Salix prolixa	Mackenzie's willow	Local collections	8,9,0,A	Fair	Poles, live cuttings
Salix scouleriana	Scouler's willow	Local collections	9,0 (Coast only)	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Sambucus racemosa	Red elderberry	Local collections	9	Fair	Brush layering, live cuttings
Spiraea douglasii	'Bashaw' Douglas Spirea (cultivar)	Nursery	9 (Coast only)	Fair	Fascines, brush mattress, brush layering, live cuttings
Spiraea douglasii	Douglas Spirea	Local collections	0,9	Fair	Fascines, brush mattress, brush layering, live cuttings
Symphoricarpos albus	Common snowberry	Local collections	9 (Coast only)	Fair	Fascines, brush mattress, brush layering, live cuttings

Table TS14I–5 Woody plants with fair to good ability to root from dormant, unrooted cuttings and their soil bioengineering applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Caribbean area					
Batis maritima	Barilla, Saltwort	Local collections	C,H	Good	Brush mattress, brush layering, live cuttings
Bucida buceras	úcar, gregre	Local collections	C	Fair	Poles, live cuttings
Bursera simaruba	almácigo, turpentine tree	Local collections	C	Good	Poles, live cuttings
Clusia rosea	Cupey	Local collections	C,H	Good	Pole, live cuttings
Commelina ssp.	Cihítre	Local collections	C	Good	Brush mattress, brush layering, live cuttings
Cordia sebestenea	Vomitel, geiger tree	Local collections	C,H	Fair	Poles, live cuttings
Erythrina poeppigiana	Bucayo, bucare, mountain immortale	Local collections	C,H	Good	Poles, live cuttings
Glyricidia sepium	Mata ratón, Glyricidia	Local collections	C,H	Good	Poles, live cuttings
Hibiscus spp.	Hibiscos	Local collections	C,H	Good	Fascines, poles, brush mattress, brush layering, live cuttings
Hymenocallis caribaea	Lirio blanco, Spyder lilly	Local collections	C	Fair	Brush mattress, brush layering, live cuttings
Lagerstroemia indica	Astromelia	Local collections	C	Good	Fascines, poles, brush mattress, brush layering, live cuttings
Mangrove species	(Rhizophora, Avicenia, Conocarpus)	Local collections	C,H	Good	Pole, live cuttings
Nicolaia elatior	Flor de cera, Torch ginger	Local collections	C	Fair	Brush mattress, brush layering, live cuttings
Pictetia aculeata	Fustic	Local collections	C	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Rhoeo spathacea	Sanguinaria	Local collections	C,H	Good	Brush mattress, brush layering, live cuttings
Sansevieria hyacinthoides	Lengua de chucho, sweet Sansevieria	Local collections	C	Good	Brush mattress, brush layering, live cuttings
Sphagneticola trilobata	Margarita, Bay Biscayne creeping oxeye	Local collections	C	Fair	Brush mattress, brush layering, live cuttings
Zingiber spp.	Jengibre, Ginger	Local collections	C, H	Fair	Fascines, poles, brush mattress, brush layering, live cuttings
Cordyline terminalis	Ti	Local collections	H	Good	Live cuttings, poles
Polyscias guifoylei	Panax	Local collections	H	Good	Live cuttings, poles

Table TS14I–5 Woody plants with fair to good ability to root from dormant, unrooted cuttings and their soil bioengineering applications—Continued

Scientific name	Common name and cultivars*	Procure from	Region of adaptation**	Rooting ability	Soil bioengineering technique
Erythrina variegate	Tropic Coral' Tall Erythrina (cultivar)	Nursery	H	Good	Live cuttings, poles

**Region code number or letter

1–Northeast (ME, NH, VT, MA, CT, RI, WV, KY, NY, PA, NJ, MD, DE, VA, OH)

2–Southeast (NC, SC, GA, FL, TN, AL, MS, LA, AR)

3–North Central (MO, IA, MN, MI, WI, IL, IN)

4–North Plains (ND, SD, MT eastern, WY eastern)

5–Central Plains (NE, KS, CO eastern)

6–South Plains (TX, OK)

7–Southwest (AZ, NM)

8–Intermountain (NV, UT, CO western)

9–Northwest (WA, OR, ID, MT western, WY western)

0–California

A–Alaska
C–Caribbean
H–Hawaii

*Cultivar
The NRCS Plant Materials Program is responsible to locating native species to address conservation problems

Once a species is identified, the Plant Material Centers make multiple collections of this species, plant them out, compare them against each other, select the best ones, and release them to the public market.

The release notice describes where the cultivar was collected and how and where it was tested. This release notice, or pedigree, also explains how the cultivar performed in various soil series, precipitation zones, and provides other information regarding its growing requirements

Figure TS14I–2 *Salix exigua* ssp. *exigua* (Coyote willow)

Figure TS14I–3 *Salix amygdaloides* (Peachtree willow)

Figure TS14I–4 *Cornus sericea* (Redosier dogwood)

able to wildlife as fruit producers and are adapted to most soil conditions including wetter sites. Gray dogwoods prefer drier sites. The shrubs spread through bird activity, branch layering, wounding, and root suckering. Dogwoods generally persist longer than willows because of their shade tolerance. Some practitioners apply rooting hormone to stimulate development from cuttings of dogwood and some other species, while others find that this is not cost-effective, and may be counterproductive to long-term, successful survival.

Redosier and silky dogwoods both perform excellent when used in soil bioengineering techniques. Gray dogwood, while excellent for wildlife, is a multistemmed clone and does not always perform well when used in soil bioengineering techniques. Roughleaf and alternated leafed dogwood also do not perform well in soil bioengineering techniques.

Cottonwoods and poplars (*Populus* spp.)—Numerous native cottonwoods exist and are suitable to a range of settings. Species include black, narrowleaf, Fremont and Eastern cottonwoods, and balsam poplar. All are trees that typically inhabit coarse-textured soils that are periodically flooded such as flood plains and streambanks. Unlike willows, cottonwoods may require periodic phases of dry soils. Black cottonwood (fig. TS14I–5) occurs with whiplash and yellow willow on coarse, well-drained soils that flood periodically. Narrowleaf cottonwood (fig. TS14I–6) is found at slightly higher elevations with redosier dogwood and alder and prefers coarse-textured, wet sites that drain quickly. Hardwood cuttings should be taken from sections with smooth bark, rather than older, deeper furrowed branches, as these stem tissues generate more roots and shoots from active nodes. The live cuttings should be generally tapered from the bottom to the top. If a tree form is needed, do not cut the top apical bud off; strip off all but the top five to six buds from the pole. Cutting the top off will cause the resulting cottonwood to be more shrub-like than tree-like.

Size and form

Hardwood propagation is defined as a cutting taken from a mature woody stem for the purpose of propagation. Hardwood cuttings are made from branches, stems, or trunks. They are collected when the plants are dormant. Dormant hardwood cuttings can be divided into four general categories.

- whips
- bundles
- poles or live stakes
- post cuttings

Whips are typically the current year's growth or 1-year-old materials. Because of their small size, they should generally not be used in drier areas or areas without consistent deep watering. Pole cuttings or live stakes can be fabricated from shrub and tree species and usually range in diameter from 3/4 to 2 inches. All leaves are removed from pole cuttings and live stakes. Essentially, pole cuttings and live stakes are the same materials. Post cuttings are much larger and are taken from large shrub and tree species and range in diameter from 3 to 6 inches. Bundles are packages of smaller diameter cuttings from various species with the branches left intact.

Collection and preparation

Field identification of plant species for collection can be difficult during the dormant season, and willows, in particular, are notoriously challenging. Most field guides for trees and shrubs rely mainly on characteristics such as leaves and flowers that are observable during the growing season. A fruit and twig key can supplement a field guide in attempting to make a determination. Alternatively, source material can be located and identified in advance, preferably when leaves and other readily distinguishing features are visible. However, for most projects, precise species identification is unnecessary. Usually determining what the general plant group is (willow vs. alder), where it is growing, what the soils are, and what the water regime is will be sufficient to allow for collection of suitable materials.

Most species should be harvested when the plants are dormant or entering dormancy. This is typically in the late fall to early spring, after leaves fall and before the buds swell. Choose and harvest healthy material that is free of splits, rot, disease, and insect infestation. While it is often appropriate to include material that ranges in age up to 7 years, material should be harvested from plants that are at least 2 years old. In drier areas, current year's growth to 1-year-old stock should not be used. This younger material is often too small and does not have enough stored energy for good root establishment, and its small diameter makes it prone to drying. Harvesting of live materials should leave at

Figure TS14I–5 *Populus balsamifera* ssp. *trichrocarpa* (Black cottonwood)

Figure TS14I–6 *Populus angustifolia* (Narrowleaf cottonwood)

least a third of the parent plant intact. The equipment should be sharp to make clean cuts.

The amount of time required for cutting, bundling, transporting, and handling woody branch materials is highly dependent on a number of variables. With reasonably good access, one person can collect over 200 stems per hour. Frequently, finding the targeted plants, locating suitably sized materials, and carrying them in tied bundles back to the vehicle is a complicated and slow process, in which case, the production rate can be a mere 10 stems per hour. Finding and collecting large-sized materials typically requires more time per stem than does smaller diameter brush cuttings. Many willows grow in large stands, while most other species will be spread out and be mixed in with other species, reducing production rates. If the people performing the cutting work cannot correctly identify plant materials, a skilled botanist or forester must be supplied for that process.

Soaking the material is desirable. Soaking hydrates the stem and starts swelling the root primordia. The roots will start to emerge from the bark in 15 to 30 days depending on the species and temperature. The optimum time for soaking is 14 days. Alternatively, live cuttings can be installed the same day they are harvested. If it is necessary to harvest material significantly before installation, the live cuttings should be stored dry, but in 50 to 90 percent humidity at approximately 33 to 40 degrees Fahrenheit. Hardwood cuttings can last up to 4 months if refrigerated under continuous, cold conditions. Material that has been stored cold and dry should be rehydrated by soaking before planting. Limited mold growth may occur and is usually tolerated without compromising the viability of the cuttings. If the harvested material is stored under wet conditions for longer than 10 days, the rooting process may start. These initial roots are typically tender, making it difficult to use the material without breaking or damaging them.

Table TS14I–5 provides information about plant species associated with soil bioengineering techniques. The information is based on the expertise of soil bioengineering practitioners. Table TS14I–5 lists plant species and their performance as dormant, unrooted, hardwood cuttings. The performance of a species in any given technique may vary from that listed in the chart based on many factors. When selecting the soil bioengineering techniques, it is important to have a clear understanding of ecological influences of the area. Plant selection and the soil bioengineering techniques used will play a role in site stabilization and will create the foundation for the ecological restoration of the site. When planning a project, the practitioner should consider using plant species that could be planted as seeds or seedlings, in addition to dormant, unrooted hardwood cuttings to enhance the restoration process. There are many soil bioengineering techniques not listed in table TS14I–5 due to their consideration as an adaptation to the basic techniques.

Herbaceous plants

Soil bioengineering also uses grasses, legumes, and forbs for streambank stabilization. These plants are typically applied in a seed mix under erosion control fabric. With adequate moisture they sprout quickly and put out root systems that hold soil in place. Table TS14I–6 lists a number of grass, legumes, and forbs species that are useful in soil bioengineering projects.

USDA NRCS Plant Materials Program: Plant development for streambank stabilization

The USDA NRCS Plant Materials Program is a nationwide network of 26 Plant Materials Centers (PMC). The PMCs service area boundaries are ecologically distinct. The PMCs evaluate plants for specific conservation traits, select top performers, and make these materials available to the public as conservation plant releases. The PMCs also develop innovative ways for land managers to use and manage a variety of conservation plants. Specialists relay information about new plant releases and offer on-the-ground assistance with conservation plantings. The Plant Materials Program evaluates streambank stabilization species based primarily on the following criteria:

- rooting/layering ability
- growth rate
- branching density
- disease resistance

Table TS14I–6 Grasses, legumes, and forbs for soil bioengineering systems

Scientific name	Common name	Region/ wetland status	Soil	Shade tolerance	Drought tolerance	Flood tolerance
Ammophila breviligulata	American beachgrass	1, facu 2, upl 3, upl*	Sands	Poor	Fair	Poor
Andropogon gerardii	Big bluestem	1, fac 2, fac 3, fac- 4, facu 5, fac- 6, facu 7, fac- 8, facu 9, facu	Loams	Poor	Good	Fair
Beckmannia Syzigachne	Sloughgrass	1, obl 3, obl 4, obl 5, obl 7, obl	Silt	Poor	Poor	Good
Calamagrostis canadensis	Blue-joint reedgrass	1, facw+ 2, obl 3, obl 4, facw+ 5, obl 7, obl 8, obl 9, facw+ 0, facw+	Silt	Poor	Poor	Good
Carex aquatilis	Water sedge	1, obl 3, obl 4, obl 5, obl 7, obl 8, obl 9, obl 0, obl	Silt	Poor	Poor	Good
Carex nebrascensis	Nebraska sedge	3, obl 4, obl 5, obl 7, obl 8, obl 9, obl 0, obl	Silt	Poor	Poor	Good
Carex utriculata	Beaked sedge	A, obl	Silt	Poor	Poor	Good

Table TS14I–6 Grasses, legumes, and forbs for soil bioengineering systems—Continued

Scientific name	Common name	Region/ wetland status	Soil	Shade tolerance	Drought tolerance	Flood tolerance
Deschampsia caespitosa	Tufted hairgrass	1, facw 2, facw 3, facw+ 4, facw 7, facw- 8, facw 9, facw 0, facw A, fac	Loam	Poor	Poor	Good
Distichlis spicata var.*stricta*	Inland saltgrass	8, fac+* 9, fac+ 0, facw*	Loam	Good	Good	Good
Eleocharis palustris	Creeping spikerush	1, obl 2, obl 3, obl 4, obl 5, obl 6, obl 7, obl 8, obl 9, obl 0, obl A, obl	Silt	Poor	Poor	Good
Elymus lanceolatus	Streambank wheatgrass	3, facu- 4, fac 5, fac 7, fac 8, upl 9, facu- A, upl	Loam	Fair	Fair	Fair
Elymus virginicus	Wildrye	1, facw- 2, fac 3, facw- 4, fac 5, fac 6, fac 7, facw 8, facw 9, facw	Loams	Good	Fair	Good
Elytrigia elongate	Tall wheatgrass	3, fac 4, fac 5, fac 8, fac	Loam	Fair	Fair	Good

Table TS14I–6 Grasses, legumes, and forbs for soil bioengineering systems—Continued

Scientific name	Common name	Region/ wetland status	Soil	Shade tolerance	Drought tolerance	Flood tolerance
Elytrigia intermedia	Intermediate wheatgrass	2, facu 3, facu	Loam	Fair	Fair	Fair
Eragrostis trichodes	Sand lovegrass		Sands	Poor	Good	Poor
Festuca rubra	Red fescue	1, facu 2, facu+ 3, fac- 5, fac 6, fac* 7, facw- 8, fac 9, fac 0, fac A, fac	Loams	Good	Good	Fair
Hemarthria altissima	Limpograss	6, facw	Sandy	Poor	Poor	Good
Glyceria striata	Mannagrass	1-9, obl 0,A, obl	Silt	Fair	Fair	Good
Juncus balticus	Baltic rush	1, facw+ 3, obl 4, obl 5, obl 6, obl 7, obl 8, facw 9, obl 0, obl A, obl	Silt	Good	Good	Good
Juncus mertensianus	Merten's rush	7, obl 8, obl* 9, obl 0, obl A, obl	Silt	Fair	Fair	Good

Table TS14I–6 Grasses, legumes, and forbs for soil bioengineering systems—Continued

Scientific name	Common name	Region/ wetland status	Soil	Shade tolerance	Drought tolerance	Flood tolerance
Juncus tenuis	Poverty rush	1, fac- 2, fac 3, fac 4, fac 5, fac 6, fac 7, facw- 8, fac 9, fac 0, fac A, facw C, obl H, fac+	Silt	Fair	Fair	Fair
Panicum	Coastal	1, facu- 2, fac	Sands to	Poor	Good	Good
Amarulum	Panicgrass	6, facu-	loams			
Panicum virgatum	Switchgrass	1, fac 2, fac+ 3, fac+ 4, fac 5, fac 6, facw 7, fac+ 8, fac 9, fac+ H, upl	Loams to sands	Poor	Good	Good
Pascopyrum smithii	Western wheatgrass	1, upl 2, facu 3, facu+ 4, facu 5, facu 6, fac- 7, fac- 8, facu 9, facu 0, fac- A, upl	Loam	Good	Good	Good
Paspalum vaginatum	Seashore paspalum	2, obl 6, facw* C, obl H, facw+	Sandy	Poor	Good	

Table TS14I–6 Grasses, legumes, and forbs for soil bioengineering systems—Continued

Scientific name	Common name	Region/ wetland status	Soil	Shade tolerance	Drought tolerance	Flood tolerance
Puccinellia nuttalliana	Alkaligrass	1, fac 3, obl 4, obl 5, obl 7, obl 8, obl 9, obl 0, obl A, facw	Loam	Good	Good	Good
Schizachyrium scoparium	Little bluestem	1, facu- 2, facu 3, facu- 4, facu 5, facu 6, facu+ 7, facu 8, facu 9, facu	Sands to loams	Poor	Good	Poor
Scirpus acutus	Hard-stem bulrush	1, obl 2, obl 3, obl 4, obl 5, obl 6, obl 7, obl 8, obl 9, obl 0, obl	Silt	Poor	Poor	Good
Scirpus maritimus	Cosmopolitan bulrush	1, obl 9, obl 0, obl H, obl	Silt	Good	Good	Good
Scirpus pungens	Common three-square	1, facw+ 2, obl 4, obl 5, obl 6, obl 7, obl 8, obl 9, obl 0, obl	Silt	Fair	Fair	Good
Sorghastrum nutans	Indiangrass	1, upl 2, facu 3, facu+ 4, facu 5, facu 6, facu 7, upl 8, facw	Sands to loam	Fair	Fair	Poor

Table TS14I–6 Grasses, legumes, and forbs for soil bioengineering systems—Continued

Scientific name	Common name	Region/ wetland status	Soil	Shade tolerance	Drought tolerance	Flood tolerance
Spartina pectinata	Prairie cordgrass	1, obl 2, obl 3, facw+ 4, facw 5, facw 6, facw+ 7, facw 8, obl 9, obl	Silt	Fair	Fair	Good
Verbena hastate	Blue vervain Swamp verbena	1, facw+ 2, fac 3, facw+ 4, facw 5, facw 8, facw 9, fac+ 0, facw	Loam	Fair	Fair	Fair
Zizaniopsis miliacea	Giant cutgrass	1, obl 2, obl 3, obl 6, obl	Loam	Poor	Poor	Good

Region code number or letter
1–Northeast (ME, NH, VT, MA, CT, RI, WV, KY, NY, PA, NJ, MD, DE, VA, OH)
2–Southeast (NC, SC, GA, FL, TN, AL, MS, LA, AR)
3–North Central (MO, IA, MN, MI, WI, IL, IN)
4–North Plains (ND, SD, MT eastern, WY eastern)
5–Central Plains (NE, KS, CO eastern)
6–South Plains (TX, OK)
7–Southwest (AZ, NM)
8–Intermountain (NV, UT, CO western)
9–Northwest (WA, OR, ID, MT western, WY western)
0–California (CA)
A–Alaska (AK)
C–Caribbean
H–Hawaii

Indicator categories (estimated probability):
fac Facultative— Equally likely to occur in wetlands or nonwetlands (34–66%)
facu Facultative upland—Usually occur in nonwetlands (67–99%), but occasionally found in wetlands (1–33%)
facw Facultative wetland—Usually occur in wetlands (67–99%), but occasionally found in nonwetlands
obl Obligate wetland—Occur almost always (99%) under natural conditions in wetlands
upl Obligate upland—Occur almost always (99%) under natural conditions in nonwetlands

Frequency of occurrence:
– (negative sign) indicates less frequently found in wetlands
+ (positive sign) indicates more frequently found in wetlands
* (asterisk) indicates wetlands indicators were derived from limited ecological information
ni (no indicator) indicates insufficient information was available to determine an indicator status

Purchasing plant materials

Collection of plant materials in the wild typically results in a good match of locally adapted plants that are available at modest cost. However, this is not always possible, and if local materials are in short supply or are not available or are unsuitable, nursery-grown material can be used instead. An increasing number of nurseries grow cultivars specifically developed for streambank stabilization/soil bioengineering by the NRCS Plant Materials Program. Some nurseries have blocks of plants available on their site, and the customer cuts it themselves and pays by the pound or per stem for the cuttings. Other nurseries offer cuttings suitable for use as pole cuttings, fascines, brush layers, or other applications, and these are priced and sold by the unit. The practitioner should coordinate with the nursery in advance so that these plants match the desired specifications for the project. It is frequently necessary to reserve plant materials in advance by placing a deposit or by arranging to contract-grow or cut the desired materials, including both woody and herbaceous plants. Species selected from nursery-grown stock should be appropriate for the plant hardiness zone (fig. TS14I–7) in which the project occurs. Nurseries sell materials that are grown elsewhere in the country, so it is important to know the provenance of the plant materials or seed that are being purchased.

Bareroot plants

Bareroot trees and shrubs are commonly grown by native-plant nurseries. This form of plant is useful both as direct plantings and when used in selected soil bioengineering measures. Bareroot materials are economical and easy to store, transport, and install. When purchasing bareroot plants, select good quality seedlings with a height of at least 18 inches and a root collar of 3/8 inch. Plants should be firm, and the growing layer underneath the bark should be green. This is tested by scratching off a small area of the bark. The plants should have a substantial root mass about equal in size to the rest of the plant.

Proper storage and handling of bareroot materials is critical to ensure viability. Bareroot plants can be stored for months prior to planting as long as the roots do not dry out, or freeze, and the plant does not leaf out. Store bareroot plants in a cool, damp, dark location. Moist materials such as sawdust, shredded newspaper, long straw or soil can be placed around the plants to prevent the roots from drying out.

Dehydration of the roots is one of the main causes for poor performance. In addition to keeping the roots moist during storage and handling, it is important to ensure good soil-to-root contact, once installed. The use of root gels and absorbent polymers, such as Terra-Sorb®, increase survival rates in drier, coarser textured soils.

Containerized plants

Containerized plants are the most expensive and cumbersome restoration materials, but they have the highest survival rate. Seedlings are grown in containers that vary in size and shape. Each container holds a seedling, soil, and nutrients. These containerized seedlings are planted whenever the soil is unfrozen. Spring and fall are generally the best times. Containerized seedlings have not experienced the root trauma of bareroot stock when they are harvested. Generally, containerized plants have a higher survival rate than bareroot and a lower cost per surviving seedling. They are easier to hand-plant, and they store better and for longer periods than bareroot plants. The use of container stock extends the season for soil bioengineering.

The practitioner should carefully inspect containerized plants prior to accepting delivery. Remove several plants from their pots, and check the roots to be sure they are white and fibrous and conform to the shape of the container. Plants with large, thick, circling roots indicate that the plant has outgrown its container. These need to be rejected and removed from the job site. Conversely, if a significant amount of soil spills out from the pot, revealing a root system that does not conform to the shape of the container, then the plant is too small for the container and should be rejected and removed from the job site.

Plants should be vigorous, healthy, and free of damage to the roots and branches. Containerized plants should also be free of insect infestation, disease, sun-scald, disfigurement, and abrasion. Healthy plant materials are most able to tolerate less than ideal conditions and survive on a restoration site.

Figure TS14I–7 USDA plant hardiness zone map and key

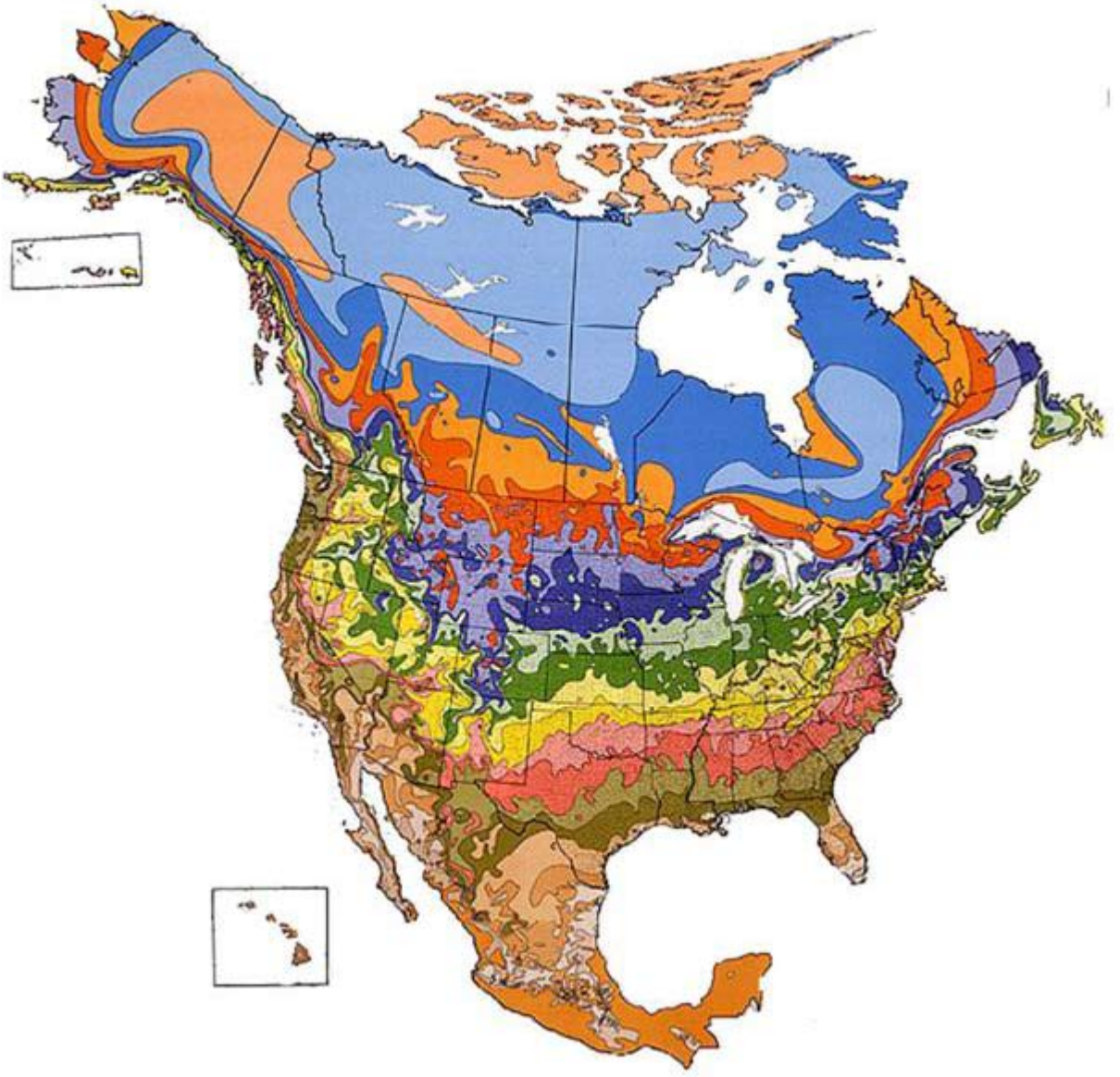


Figure TS14I–7 USDA plant hardiness zone map and key—Continued

USDA zone	Temperature range	Example cities
1	Below -50 F (-45.6 C)	Fairbanks, Alaska Resolute, Northwest Territories (Canada)
2a	-50 to -45 F (-42.8 to -45.5 C)	Prudhoe Bay, Alaska Flin Flon, Manitoba (Canada)
2b	-45 to -40 F (-40.0 to -42.7 C)	Unalakleet, Alaska Pinecreek, Minnesota
3a	-40 to -35 F (-37.3 to -39.9 C)	International Falls, Minnesota St. Michael, Alaska
3b	-35 to -30 F (-34.5 to -37.2 C)	Tomahawk, Wisconsin Sidney, Montana
4a	-30 to -25 F (-31.7 to -34.4 C)	Minneapolis/St.Paul, Minnesota Lewistown, Montana
4b	-25 to -20 F (-28.9 to -31.6 C)	Northwood, Iowa Nebraska
5a	-20 to -15 F (-26.2 to -28.8 C)	Des Moines, Iowa Illinois
5b	-15 to -10 F (-23.4 to -26.1 C)	Columbia, Missouri Mansfield, Pennsylvania
6a	-10 to -5 F (-20.6 to -23.3 C)	St. Louis, Missouri Lebanon, Pennsylvania
6b	-5 to 0 F (-17.8 to -20.5 C)	McMinnville, Tennessee Branson, Missouri
7a	0 to 5 F (-15.0 to -17.7 C)	Oklahoma City, Oklahoma South Boston, Virginia
7b	5 to 10 F (-12.3 to -14.9 C)	Little Rock, Arkansas Griffin, Georgia
8a	10 to 15 F (-9.5 to -12.2 C)	Tifton, Georgia Dallas, Texas
8b	15 to 20 F (-6.7 to -9.4 C)	Austin, Texas Gainesville, Florida
9a	20 to 25 F (-3.9 to -6.6 C)	Houston, Texas St. Augustine, Florida
9b	25 to 30 F (-1.2 to -3.8 C)	Brownsville, Texas Fort Pierce, Florida
10a	30 to 35 F (1.6 to -1.1 C)	Naples, Florida Barstow, California
10b	35 to 40 F (4.4 to 1.7 C)	Miami, Florida Coral Gables, Florida
11	above 40 F (4.5 C)	Honolulu, Hawaii Mazatlan, Mexico

The 1998 Web version of the 1990 USDA Plant Hardiness Zone Map

Balled and burlapped material

Balled and burlapped (B&B) plants can be expensive and cumbersome to establish. However, they provide a larger, more robust plant that is big enough to give some structure to a new planting. B&B trees can provide immediate cover for wildlife, and they moderate hydrology, and protect streambanks. They are used on eroding salmonid streams to reestablish the habitat functions more rapidly.

B&B plants are dug, and the root mass is wrapped in burlap to keep the soil on the roots. They should have solid root balls with enough of the root systems present to support the top growth of the plants. Ensure that the grower has harvested sufficient root mass to support the aboveground biomass.

Before B&B plants are set in the planting hole, it is recommended that the burlap be totally removed to ensure that the roots are allowed to grow uninhibited. B&B plants will need to be braced with supports for the first season until the root system has grown and spread enough to support the tree on its own.

Streambank soil bioengineering techniques

Many types of streambank soil bioengineering treatments have been used throughout the country. A collection of techniques that are broadly applicable have been divided into sections that address the different bank zones.

It is appropriate to modify these treatments to account for site-specific conditions, cost of materials, and material availability. Many variations of these techniques exist. Many of the techniques listed are often combined with other streambank soil bioengineering techniques or with harder, inert structures.

Toe treatments

Coir fascines

This is a manufactured product also known as coir logs or coconut fiber rolls (figs. TS14I–8 and TS14I–9). Coir fascines consist of coconut husk fibers bound together in a cylindrical bundle by natural or synthetic netting and are manufactured in a variety of standard lengths, diameters, and fill densities for different energy environments. Coir fascines are flexible and can be fitted to the existing curvature of a streambank. They provide immediate toe protection and bank stabilization, while trapping sediment within the coir fascine, which encourages plant growth. Coir fascines are well suited for establishing herbaceous materials, and they can be prevegetated prior to installation. A key advantage of this method is the modularization and standardization of the materials that result in relatively predictable and reliable performance. A disadvantage of coir fascines is that they are expensive to purchase and ship. They require additional anchoring systems, which increases the initial costs and installation time.

Materials

- Fascines fabricated from and filled with 100 percent coir (coconut husk) are preferred for streambank stabilization work because they serve as a stable growing medium on which seeds and young plants can become established. This material provides some resistance to damage from ice flows, floating debris, and other impacts, and provides a reinforcing framework for vegetation until the coir filling decays, at which point the plants should be able to protect the banks.
- For most settings, high tensile strength (minimum 200 lb tensile strength) synthetic mesh is desirable for the knotted or braided mesh exterior of the coir fascine. Although coir mesh versions are available, the mesh frequently loses its strength before vegetation can become fully established, making the material vulnerable to failure. Therefore, coir mesh versions are typically used on sites with low stress levels.
- The most sturdy and resistant coir fascines are manufactured with a density of 9 pounds per cubic foot. Where ice, debris, steep banks, and other stress factors are not a problem, lower density materials may offer a more cost-effective alternative.
- The most commonly used size is 12-inch diameter, although they are available in both larger and smaller sizes.
- Coir fascines are typically anchored with wooden stakes or earth anchors with cable assemblies. Soil anchor design and installation are addressed in more detail in NEH654 TS14E.

Figure TS14I–8 Installation of coir fascines: (a) Anchoring; (b) Tying of the coir fascines together (*Photos courtesy of Robbin B. Sotir & Associates, Inc.*)

(a)

(b)

Figure TS14I–9 Stacked coir fascines using woody vegetation (*Photo courtesy of Hollis Allen*)

Installation

- Coir fascines may be installed during any season, provided that the ground can be worked adequately for placement and anchoring. Planting into the coir fascine may be planned for later in a more desirable season, as needed.
- Coir fascines can either be placed so that they help position the toe of a bank, where it was located prior to an erosion event, or in direct contact with the current bank profile. Typically, they are positioned so that the top of the coir fascine is located at the mean water level during the summer growing season. In most cases, this zone best supports herbaceous vegetation. Due to the distance from the plant to the soil, it is imperative that the coir fascine remain wet.
- Coir fascines are frequently planted with 2-inch-diameter plugs of herbaceous species which, preferably, have been rooted in a coir fiber matrix to provide good frictional contact.
- Coir fascines require protection against scouring and flanking that should be addressed in the design.
- The anchoring system must be adequate to seat the coir fascine securely in contact with the adjacent soil. Normally, this means a pair of stakes placed every 2 feet along the coir fascine, one on each side. In cold climates, earth anchors or rope tie-downs are necessary to prevent lifting of the coir fascine as ice forms. Always place wooden stakes between the cable or rope and the coir to keep the cable or rope from cutting clear through the coir fascine. Piercing a high-density coir fascine with stakes should be avoided. The stakes should be driven alongside the coir fascine. The coir fascine is secured by either tightly sandwiching the coir fascine between the stakes or by using ropes or cables to tie around the coir fascine.
- To form a continuous unit, coir fascines must be tied together end to end. This is most convenient to do while the coir fascines are still on dry land, laid out along the top of bank. Strong synthetic rope is used to stitch the ends together, with knots tied at frequent intervals to ensure a reliable connection.
- When coir fascines are stacked to provide coverage of a wider strip of bank, they must be laced together on the edges where they touch. One row of lacing is typically adequate to hold two tiers together, although two rows of lacing will result in a tighter contact between the tiers, which is useful at holding back noncohesive soils. All tiers require appropriate staking or anchoring.
- After anchoring is complete, coir fascines may be planted (fig. TS14I–9). Either live cuttings may be inserted through the coir fascine itself, or 2-inch-diameter plugs may be inserted 6 inches on center along the length of the coir fascine.
- When the coir fascines are stacked, live poles, live cuttings, or rooted plants may be placed on the first (lower) coir fascine, prior to placing the next one above it.

Brush and tree revetments

Brush and tree revetments are nonsprouting shrubs or trees installed along the toe of the streambank (fig. TS14I–10). They slow the stream velocity adjacent to an eroding bank and promote sediment deposition at the toe. This treatment is sometimes referred to as Christmas tree revetments or juniper revetments. The revetment material does not need to sprout and most

Figure TS14I–10 Brush/tree revetment over poles and a brush mattress

species used will not. It is generally recommended that live willows or other quickly sprouting species be planted behind the revetment to assist in capturing sediment and to provide a permanent living cover.

Materials

- Dead/live brush or trees such as junipers, spruce, fir, or hawthorn. Pine trees do not typically have dense and durable enough needles and branches to provide ideal shielding.
- Ties—10- to 2-gauge, galvanized smooth wire, 1/8- to 1/4-inch cable, clamps, gripples, and anchors.
- Anchors—5-foot metal T-posts or 2-inch oak posts are often used. Soil anchors and rock bolsters may also be used. Soil anchor design and installation are described in more detail in NEH654 TS14E.
- Tools—wire cutters, hammer, post pounder, chain saw for cutting brush.

Installation

- Brush and tree revetments can usually be installed throughout the year. However, for safety reasons, it is generally best to avoid high water periods.
- Harvest the trees for the revetment and stage near the site. Use trees with dense branches, such as junipers, because they will collect more sediment. Collect trees or brush and stage at the treatment area.
- Place the first tree one tree length below the downstream end of the treatment area. The stump of the tree should point upstream. Push firmly into the channel bank.
- Install an anchor post on the streamside of the tree adjacent to the trunk at the stump end where it overlaps with the next tree. Secure the tree to the post with three wraps of cable or wire, then clamp. In some situations, it may be easier to install the anchor posts before placing the trees.
- Overlap the next downstream tree trunk into the main branches of the first one by a third of the length of the tree. The stump end of the second tree should lie between the top end of the first tree and the bank. The result is a shingle-like arrangement.
- Wire the two trunks together, leaving the branches loose. Use a minimum of two wraps of cable or wire in two different places on the overlap.
- Install a second anchor post in the middle of the overlap portion of the two trees. Secure the two trees to the post with a minimum of two wraps of cable or wire in two different places on the overlap.
- Continue this process until a continuous row of brush protects the length of the treatment area.
- The trunks of the revetment should be placed at the toe where the bank meets the bed. They should be of a large enough diameter to reach to the average water elevation. In areas of fluctuating water levels, it may be necessary to place a second row of revetment at the high water line to prevent scouring behind the revetment during flood events.
- Fill in the space between the bank and the revetment with live cuttings or fascines to create a dense matrix. This material should be high enough so that they will not be rapidly covered by sediment trapped by the revetments.
- It is important that the revetment extend upstream and downstream past the area being treated to prevent flows from getting behind the revetment. It is advisable to key the upstream and downstream ends of the revetment into the bank and reinforce the key with additional brush or rock.
- To enhance recovery of the treated area, grade or knock down the sloughing streambank on the revetment to create a gentler slope as shown below. Make sure the revetment has enough brush material to catch the soil. If not, add additional brush before shaping the bank. Willow cuttings or other quickly sprouting species should then be installed on the new slope, using treatments such as fascines, brush mattress, vertical bundles, or willow post or pole plantings. This option may damage any existing vegetation.

Rootwad revetments

Rootwads are not soil bioengineering measures themselves, but are useful in supporting soil bioengineering measures installed higher up the slope of the bank. Rootwad revetments (figs. TS14I–11 and TS14I–12) make use of locally available logs and root fans to add physical habitat to streams in the form of coarse woody debris and deep scour pockets (root fan is the root mass which includes large and small roots attached to the main root mass). When placed correctly, rootwads and logs used as footers and headers around the rootwads can form an effective armor along the bank that shields soils, deflects flows away from the near-bank area, and provides roughness to reduce velocities in the near-bank zone. The use of large woody debris is addressed in more detail in NEH654 TS14J. Normally, earth, large rock or cables, and earth anchors are used to stabilize the woody elements. Various shrub and tree plantings are incorporated into the bank and flood plain areas. Since rootwads themselves will not last indefinitely, this treatment depends on a complementary strategy to replant the bank or to allow a healthy riparian corridor plant community to develop in the overbank zone. A key advantage of this method is that most materials are frequently available at low cost on or near the project site. When existing rootwad materials are not available, however, this system may be expensive due to construction collection and transportation costs and may damage the forest environment.

Figure TS14I–11 Rootwads being installed over rock toe and with soil anchors (*Photo courtesy of Mike Martyn*)

Materials

- Rootwad revetments are best created from hardwood tree species in sound condition, free from extensive decay.
- Preferably, log and root fan materials will not be collected from the bank and flood plain areas where they can provide bank stability if left intact.
- To harvest the rootwads, trees are typically pushed down with a bulldozer so that the root fan pops out of the soil attached to the tree trunk. Some cutting of larger surface roots may be required. Root fans of 5 to 12 feet in diameter are most valuable for application in typical streambank treatments, although smaller root fans may be used where they provide adequate coverage to a smaller streambank. A length of bole (or trunk) 10 to 15 feet in length is left

Figure TS14I–12 Rootwad being pushed into the bank

attached to the root fan, creating the rootwad. Remaining trunk of 12 inches or greater diameter is cut into lengths of 10 to 20 feet to form header or footer logs, as needed. The length of the logs is set, based on the desired spacing between rootwads.

- Large rock, cables, and earth anchors are often used to anchor rootwads and header/footer logs into position, and they must be sized to be stable during extreme storm events. Soil anchor design and installation are addressed in more detail in NEH654 TS14E. Rootwads may also be installed over a stone toe, footer logs, or by themselves. The choice of the toe protection depends on site-specific conditions and project purpose.

Installation

- The first step for installing rootwads is to install the toe protection. If soil anchors are to be used, they must be installed and proofed as described in NEH654 TS14E. If a rock toe is to be used, the necessary excavation and placement should be accomplished (fig. TS14I–11). If footer logs are used, they are placed along the toe of the bank with their ends overlapping. Boulders or earth anchors are used to secure them into position. Soil backfill is added to fill any gaps behind the logs.
- Rootwads are positioned in an overlapping, shingle-type arrangement with the trench almost parallel to the direction of flow. The rootwad is angled so that the root fan fits snugly into the bank at the upstream side, and may angle away from the bank on the downstream side. This requires that the bole of the rootwad be embedded into the bank pointing downstream, not perpendicular to the bank (fig. TS14I–12). Typically, the bole is pointing 30 degrees away from the tangent of the curve of the bank in plan view.
- The tip of the bole may be sharpened and forced into the bank, or it may be laid into an excavated trench. The bole is eventually buried so that it can serve as a deadman to stabilize the revetment. Boulders can be placed as needed to securely wedge the rootwad in place. Since the root fans jut into the flow path and are subject to extreme tractive forces, it is essential to secure the bole of the rootwad through careful soil packing, boulder wedging, clumps, or anchoring; otherwise, the rootwads will loosen and eventually fail.
- The header logs are placed above the footer logs, likewise with the joints located behind the root fan. Boulders or earth anchors can be used to secure the header logs in place. Soil fill is packed behind and between the header log.
- Ideally, this treatment will provide coverage up to the bankfull height; although, in some cases, it is necessary to add other additional treatments above the rootwad revetment.
- Any exposed soil above and between the header and footer logs is vulnerable to loss during extreme flow events. Normally, transplanted riparian sod or live staking is used to provide coverage and to establish vegetation for these areas.

Brush spurs

A brush spur is a long, box-like structure of brush that extends from within the bank into the streambed (figs. TS14I–13 and TS14I–14). They function very similarly to stone stream barbs as described in NEH654 TS14H. Brush spurs are sometimes referred to as brush box spurs or deflectors. The purpose of brush spurs is to promote sediment deposition along the toe of the bank. This helps with rebuilding and strengthening an eroding bank, absorbing energy, deflecting flows away from the bank, and habitat enhancement. Brush spurs are low structures and are completely overtopped during channel-forming flow events. They typically project into the channel less than a fifth of the channel width. This treatment requires a moderate to high sediment load of fine material and may not be suitable for areas with high velocities, prolonged inundation, or high debris loads.

Materials

- Brush spurs—live cuttings 3/4 to 2 inches in diameter, 6 to 20 feet long with the branches left intact
- Ties—braided manila, sisal, or prestretched cotton twine, 10- to 12-gauge, galvanized wire, 1/8- to 1/4-inch cable, clamps, or Gripples®

- Anchor posts—6 to 12 feet long and 6 inches in diameter oak posts (use longer posts in areas of looser bed material)
- Soil anchors design and installation are described in NEH654 TS14E.
- Tools—wire cutters, knives, shovels, hammers, post pounder, and chain saws

Installation

- Collect and soak live cuttings as described earlier in this technical supplement. Leave side branches intact.
- Determine alignment and spacing of brush spurs. Spurs are typically installed at an angle of 30 to 45 degrees to the bank facing upstream and act together as a system.
- The top of the spurs should be between the annual low and high water levels and slope down towards the stream channel bed. The rooted end should not extend above the top of the bank.
- Excavate a 2- to 4-foot-wide key or root trench a fifth of the spur length into the bank at the root of each spur as shown in figure TS14I–13(a). Keep this excavation as narrow as possible. The bottom of the trench should be below the bottom of the channel bed at the toe of the bank.
- Install at least two pairs of anchor posts to frame the spur (fig. TS14I–13(b)). One pair should be positioned at approximately one-third of the length of the spur, and the second pair at the approximate two-thirds distance location. The maximum distance between the posts in a pair should be 5 feet. The posts should be spaced apart at the expected width of the spur (2 to 4 feet). The final set of anchor posts should be 3 to 5 feet from the end or nose of the spur. The top of the anchor posts should extend above that of the planned spur by 6 to 12 inches.

Figure TS14I–14 Brush spur after one growing season

Figure TS14I–13 Brush spur being installed

(a)

(b)

- Install soil anchors adjacent to the posts. Soil anchor design and installation are addressed in more detail in NEH654 TS14E.
- Pack live cuttings tightly into the gap between the anchor posts. The butt or basal end of the live cuttings should be placed in the key trench, touching the undisturbed soil at the back of the trench. In areas where a longer spur is needed, overlap a mixture of live and dead material by a minimum of one-half the length of the brush. Secure the material together with two wraps of twine, wire, or cable at approximate 3- to 5-foot centers along the length of the bundle.
- Secure the brush between the posts with a minimum of two wraps of twine, wire, or cable.
- Install poles around the outside edge of the key trench.
- Cover the brush in the key trench with soil, ensuring good soil-to-stem contact. In higher stress areas, stone can be used to reinforce the area where the spur is keyed in. The installation procedure is the same as without stone with the exception of the following.
  - — Excavate a wider trench where the invert of the key trench is 1 to 2 feet below the toe of the bank.
  - — Fill the invert of the trench to the elevation of the bank toe with appropriately sized stone material.
  - — Install the brush spur as described, but fill around the key trench with additional stone.

Live siltation construction or live brush sills

Live siltation construction or live brush sills (figs. TS14I–15 and TS14I–16) are rows of live cuttings inserted into an excavated trench. Siltation in this context means the encouragement of sediment deposition of all particle sizes, not just silt sized. The live cuttings are expected to root and provide additional structural support. Live siltation construction or live brush sills are often used to supplement other treatments to assist with final infilling of scoured areas. Since this is a treatment that is intended to promote sediment deposition, it requires a moderate to high sediment load of fine materials. Live siltation construction is generally not suitable for areas with high velocities or prolonged inundation. Live siltation construction can also function as erosion stops in dry channel beds to resist the formation of rills and gullies, or in bends to resist meander cutoffs. They can also be placed parallel to the stream adjacent to the toe.

Figure TS14I–15 Live siltation construction or live brush sills

Figure TS14I–16 Live siltation construction or live brush sills with rock

Materials

- Live cuttings—3/4 to 3 inches in diameter, 2.5 to 5 feet long
- Tools—machete, clippers, shovel, saw, hammer, backhoe
- Fertilizer and other soil amendments

Installation

- Collect and soak live cuttings for 14 days, or install them the day they are harvested. Leave side branches intact. It is important to use or harvest low-growing species that will remain supple.
- Excavate a trench that is approximately 1 to 3 feet deep. The trench should have a trapezoidal shape when excavation is complete. If the trench is located along a channel, it should be oriented at about a 20- to 30-degree angle against the direction of the flow. The trench should be keyed into the bank 3 to 5 feet.
- Trenches should be approximately 3 to 15 feet apart, depending on the erodibility of the soil, gradient of the channel, and nature of the treatment they are being used to supplement.
- Orienting the growing tips downstream, pack the branches tightly with the basal ends in the excavated trench forming a dense layer of branches. The branches are placed on the downstream 45-degree angle side, with layers of soil in between. Approximately 15 to 25 live cuttings per foot of trench should be used. Be sure the branches are thick and continuous with no gaps. The ends of the branches should protrude beyond the top of the trench by 12 to 36 inches.
- Cover the downstream side of the trench with the live cuttings and soil. Wash in the soil to assure good soil-to-stem contact, then compact the soils by foot.
- Rock may be added on the upstream side of the installation to provide additional strength and protection.
- Consider seeding between the traverses. Use species that will not create competition with the woody vegetation.
- Trim the terminal end or bud to promote root growth. After installation, the area should be watered. Supplemental irrigation may also be required.

Cribwall

A cribwall is a hollow, boxlike structure of interlocking logs or timbers (fig. TS14I–17). The structure is filled with rock, soil, and live cuttings, or rooted plants. The live cuttings or rooted plants are intended to develop roots and top growth and take over some or all of the structural functions of the logs. The maximum height is typically less than 6 feet for untreated timber. Treated timber can be used to construct larger structures. The structures may not be able to resist large lateral earth pressures, and may provide a false sense of security. If used adjacent to a stream, the impact if the structure fails and washes downstream must be considered. It is critical that the toe be set securely below the estimated maximum scour. Excavations over three feet may require shoring.

Materials

- Front and rear long beams—4- to 8-inch-diameter logs or square wooden timbers, approximately 20 feet long. Peeled logs are typically more resistant to rot than logs with bark.
- Cross beams—4- to 8-inch-diameter logs, length equal to anticipated height of the structure
- Live cuttings—3/4 to 3 inches in diameter, 5 to 7 feet long, and/or bareroot plants 18 to 24 inches in length or 1 gallon container stock
- Rebar or spikes—3/8 to 1/2 inch in diameter to secure logs
- 2- to 6-inch rock for the toe foundation
- Fill material—The permeability of soil in cribbing must be less than that of the undisturbed back slope to prevent back pressure, unless a back slope drainage system is installed. Heights of over 5 feet typically require an engineered fill.
- Soil anchors may also be used. Soil anchor design and installation are addressed in more detail in NEH654 TS14E.
- Tools—machete, shovels, clippers, axe, hammer, sledge hammer, saw, and excavator

- Filter fabric to wrap the rock in the toe foundation and back slope drains
- Fertilizer and other soil amendments

Installation

- Collect and soak live cuttings for 14 days or install them the day they are harvested. Leave side branches intact. Remove loose, failed, or failing soil from the face of the slope.
- Excavate loose material to reach a stable foundation. Tilt the excavated toe so that the structure is battered (sloped into the embankment) by approximately 6 inches to 1 foot, or more. A stone toe should be set below the depth of anticipated scour and be placed in front and under the structure.
- Place front and rear long beams approximately 3 to 5 feet apart and parallel to the streambank and each other. The rear beam should be approximately 6 inches to 1 foot below the front beam.
- Place crossbeams perpendicular to the front and long rear beams on 3- to 5-foot centers.
- Allow crossbeams to overlap the front and long rear long beams by 6 inches to 1 foot. Secure with spikes or rebar.
- Fill inside of structure with rock approximately 1 to 3 feet above the channel baseflow. In a small stream system, the rock should only be used in the face of the cribwall to a height of 1 to 2 feet above baseflow. Confine and separate the rock with filter fabric if necessary.
- The next layer or two is typically filled with a 50/50 mix (by volume) of rock and soil. The soil used is typically native soil, but it must be able to drain sufficiently to prevent pressure from building up in the bank. In rare cases, amended soil (native material that has been improved with fertilizers and other nutrients that will improve plant establishment) may be used, especially if the native soil is infertile. However, this can increase the cost and may have negative ecological impacts on the stream.
- Step back succeeding layers so that the cribwall is inclined 10 to 30 degrees from the vertical (1H:4V to 1H:6V).

Figure TS14I–17 (a) Live cribwall under construction; (b) After first growing season

(a)

(b)

- Once logs or timbers are stacked above the existing rock fill, place live cuttings with the basal ends towards the slope and the growing tips towards the stream, or use bareroot or container stock. When live cuttings are used, allow the bud tips to extend 6 to 18 inches beyond the front of the long beams, and ensure that at least 40 percent of the basal ends of the branches extend 6 to 12 inches beyond the long rear cross beams.
- Align the live cuttings so they extend on top of the front long beam and below the long rear beam for a given course.
- Trim the terminal bud so that stem energy will be routed to the lateral buds for more rapid sprouting.
- Typical density is 6 to 12 live cuttings per linear foot, or 3 to 6 bareroot plants per linear foot, or one container plant every 12 to 18 inches.

Fascines

A fascine is a long bundle of live cuttings bound together into a rope or sausage-like bundles (figs. TS14I–18 and TS14I–19). The structure provides immediate protection for the toe. Since this is a surface treatment, it is important to avoid sites that will be too wet or too dry.

The live cuttings eventually root and provide permanent reinforcement. Fascines may be placed farther up the bank to assist in controlling overland flow by breaking long banks into a series of shorter banks. This is described later in contour fascines.

Figure TS14I–19 Installation of live fascines combined with erosion control fabric (*Photo courtesy of Robbin B. Sotir & Associates, Inc.*)

Figure TS14I–18 Assembling fascines

Materials

- Live cuttings—3/4 to 2 inches in diameter, 5 to 15 feet long
- Cord, braided manila, sisal or prestretched cotton twine, or small-gauge, nongalvanized wire
- Dead stout stakes—wedge-shaped wooden stakes, 2 to 3 feet long depending on soil conditions
- Tools—machete, shovels, clippers, hammer, sledge hammer, saw, and chain saw
- Fertilizer and other soil amendments

Installation

- Collect and soak live cuttings for 14 days, or install them the day they are harvested and fabricated. Leave side branches intact.
- Stagger the live cuttings in a uniform bundle built to a length of about 8 feet. Vary the orientation of the cuttings. Use 8- to 10-foot bundles for ease of handling, and transport in a pickup bed. They can also be easily spliced together to create a fascine long enough to fit the particular project site.
- Tie bundles with twine at approximately 2-foot intervals. The bundles should be 6 to 24 inches in diameter, depending on their application.
- Start installation from a stable point at the upstream end of the eroding bank.
- Excavate a trench into the bed of the stream, where the bank meets the bed. The trench should be about a half to three-quarters the diameter of the bundle.
- Align the fascine along the toe of the bank of the eroding section.
- Place the bundle in the trench and stake (use wedge shaped dead stout stakes) directly through the bundle 3 feet on center. Allow the stake to protrude 2 inches above the top of the bundle. To improve depth of reinforcement and rooting, install live stakes (2 to 3 ft in length) just below (downslope) and in between the previously installed dead stout stakes, leaving 3 inches protruding from the finished ground elevation.
- Cover the fascine with soil, ensuring good soil-to-stem contact. Wash it in with water to get around the inner stems of the bundle. Some of the bundle should remain exposed to sunlight to promote sprouting. Use material from the next upbank trench. It may be desirable to use erosion control fabric to hold the soil adjacent to and in between the fascine bundles, especially in wet climates. When using erosion control fabric between the fascine bundles, the fabric is first placed in the bottom of the trench, an inch of soil is placed on top and up the sides of the trench and erosion control fabric, and the fascine bundle is then placed in the trench and staked down (fig. TS14I–19).

Note: Fascines can be oriented perpendicular to the streambank contours. This practice is often called the vertical bundle method. The primary difference between the construction of a vertical bundle and a fascine is that all of the cuttings in a vertical bundle are oriented so the cut ends are in the water. It is particularly applicable in arid and semiarid areas where there is uncertainty in determining the water table.

Bank treatments

Live pole cuttings or live stakes

Live pole cuttings are dormant stems, branches, or trunks of live, woody plant material inserted into the ground with the purpose of getting them to grow (figs. TS14I–20 through TS14I–22). Live stakes are generally shorter material that are also used as stakes to secure other soil bioengineering treatments such as fascines, brush mattresses, erosion control fabric, and coir fascines. However, the terms live stakes and live pole cuttings are often used interchangeably. Both live poles and live cuttings can be used as anchoring stakes. They are live material so they will also root and sprout. Live pole cuttings are 3 to 10 feet long, and 3/4 to 3 inches in diameter. These cuttings typically do not provide immediate reinforcement of soil layers, as they normally do not extend beyond a failure plane. Over time, they provide reinforcement to the soil mantle, as well as surface protection and roughness to the streambank and some control of internal seepage. They assist in quickly reestablishing riparian vegetation and cause sediment deposition in the treated area.

Figure TS14I–20 Preparation of pole cuttings

Figure TS14I–21 Iron punch bar being used to create pilot hole

Figure TS14I–22 Live pole cuttings after one season

Materials

- Live cuttings—3/4 to 3 inches in diameter, 3 to 20 feet long
- Tools—machete, clippers, dead blow hammer, saw, chain saw, loppers, and rebar

Installation

- Cleanly remove all side branches and the top growth. Cut the basal (bottom) end to a 45-degree angle, or sharpen into a pointed end. The top end should be cut flat. At least two buds or bud scars should be present above the ground in the final installation, depending on the surrounding vegetation height. The live cuttings should be taller than the surrounding vegetation to ensure that they are not shaded.
- Collect and soak the live cuttings for 14 days, or install them the day that they are harvested and fabricated.
- Use a punch bar or hand auger to create a pilot hole that is perpendicular to the slope. The depth of the hole should be 2/3 to 3/4 the length of the live cutting. Make the hole diameter as close to the cutting's diameter as possible to obtain the best soil-to-stem contact. The hole should be deep enough to intercept the lowest water table of the year or a minimum of 2 feet.
- To achieve good soil-to-stem contact, fill the hole around the pole with a water-and-soil slurry mixture. Add soil around the cutting as the water percolates into the ground and the soil in suspension settles around the cutting. Another method is to tamp soil around the cutting with a rod. Throw a small amount of soil in the hole around the cutting and tamp it down to remove all air pockets. This is similar to installing a wooden fence post.
- Install the pole into the ground at a right angle to the slope face. Use a dead blow hammer to tap the cutting into the ground. Insert the cutting at a 90-degree angle to the face of the slope. Ensure that the sharpened basal end is installed first.
- Place stakes on 2- to 4-foot spacing in either a random pattern or triangular grid for most shrub species. Spacing depends on species, moisture, aspect, and soil.

Dormant post planting

Dormant post plantings are large cuttings of live, woody plant material inserted into the ground (fig. TS14I–23). Typically, these are 5 to 20 feet long and have diameters ranging in size from 3 to 8 inches. These dormant live post cuttings provide some immediate, but limited, reinforcement of soil layers if they extend beyond a failure plane. The live post cuttings are intended to root and provide soil reinforcement and subsurface protection, as well as supply roughness to the streambank and offer some control of internal seepage. They assist in quickly reestablishing riparian vegetation and cause sediment deposition in the treated area.

Materials

- Dormant live post cuttings—3 to 8 inches in diameter, 3 to 20 feet long. May use posts up to 10 feet long with auger installation, depending on auger length
- Tools—machete, clippers, hammer, punch bar, and saw. May also include chain saw, loppers, power auger, hand auger, waterjet, and mechanized stingers (fig. TS14I–23).
- Fertilizer and other soil amendments

Installation

- Cleanly remove all side branches and the top growth. Cut the basal end to a 45-degree angle or sharpen to a point. The top end should be cut flat. At least two buds or bud scars should be present above the ground, depending on the surrounding vegetation height. The live post cuttings should be taller than the surrounding vegetation to ensure that they are not shaded.
- Collect and soak live post cuttings for 14 days, or install them the day they are harvested.
- Use a punch bar, hand, or power auger to create a pilot hole that is perpendicular to the slope. The depth of the hole should be two-thirds to three-fourths the length of the stake. Make the hole diameter as close to the cutting's diameter as possible to obtain the best soil-to-stem contact. The hole should be deep enough to intercept the lowest water table of the year or a minimum of 2 feet.

Figure TS14I–23 Installation of dormant posts with stinger

- Push or lightly tap the post into the ground perpendicular to the slope, so that the sharpened basal end is inserted first.
- To achieve good soil-to-stem contact, fill the hole around the post with a water and soil slurry mixture. Add soil around the cutting as the water percolates into the ground and the soil in suspension settles around the cutting. Another method is to tamp soil around the cutting with a rod. To use this approach, throw a small amount of soil in the hole around the cutting and tamp it down to remove all air pockets. Repeat this in layers. This is similar to installing a wooden fence post.
- Place the dormant post on 2- to 4-foot spacing in a random pattern or triangular grid. Spacing is dependent on species, moisture, aspect, and soil characteristics.

Contour fascines

Contour fascines are another use of fascines to assist in controlling overland flow by breaking long banks into a series of shorter banks (fig. TS14I–24). (See fascine description under the section entitled Toe treatments). They may be placed on the bank along the contour, or on an angle to facilitate (capture and direct) drainage. The structure provides immediate protection against surface erosion, due to its orientation (approximately perpendicular, even at an angle) to the slope face and its porous barrier-like installation. This treatment may provide immediate protection against shallow-seated slope failure, as both live cuttings and dead stout stakes are incorporated into the measure. The live cuttings are intended to root and provide additional reinforcement to the soil mantle.

Materials

- Live cuttings—3/4 to 2 inches in diameter, 5 to 15 feet long
- Ties—cord, braided manila, sisal or prestretched cotton twine, or small-gauge, nongalvanized wire
- Dead stout stakes—wedge shaped, 2 to 3 feet long, depending on soil conditions
- Tools—machete, shovels, clippers, hammer, sledge hammer, saw, and chain saw
- Fertilizer and other soil amendments

Installation

- Collect and soak cuttings 14 days, or install them the day they are harvested and fabricated. Leave the side branches intact.
- Stagger the live cuttings in a uniform bundle built to a length of about 8 feet. Vary the orientation of the cuttings. Use 8- to 10-foot bundles for easy to handling, and transport in a pickup bed. They can also be easily spliced together to create a fascine long enough to fit the particular project site.
- Tie bundles with twine at approximately 1- to 2-foot intervals. The bundles should be 6 to 24 inches in diameter, depending on their application. Typically, smaller diameter bundles are used.
- The installation process begins at the toe of the bank and proceeds towards the top of bank.
- Remove loose, failed, or failing soil from face of the bank, and generally smooth the face to facilitate installation procedures.
- Align the fascine along the contour for dry banks. Place the fascine bundle at an angle ranging from 30 to 60 degrees along wet slopes to facilitate (capture and direct) drainage. On upper banks adjacent to a stream and along outside meanders, it may be useful to align the fascines at an angle to reduce the likelihood of scour and rilling around installed bundles.

Figure TS14I–24 Fascines installed at an angle over a riprap toe

- Excavate a trench approximately three-quarters the diameter of the bundle. In rare cases, it may be desirable to place 1 inch of amended soil (native material that has been improved with fertilizers or other nutrients that will improve establishment) in the bottom and up the sides of the trench. However, this can increase project costs and may have negative ecological impacts on the stream.
- Place the bundle in the trench and hammer in the wedge shaped (dead stout) stakes directly through the bundle 3 feet on center. The top of the stakes should protrude 2 inches above the top of the bundle. To improve depth of reinforcement and rooting, install live cuttings (2 to 3 ft in length) just below (downslope) and in between the previously installed dead stout stakes, leaving 3 inches protruding from the finished ground elevation.
- Cover the brush with soil, ensuring good soil-to-stem contact. Some of the bundle should remain exposed to sunlight to promote sprouting. Use material from the next upbank trench. It may be desirable to use erosion control fabric to hold the soil adjacent to and in between the fascine bundles especially in wet climates. When using erosion control fabric between the fascine bundles, the fabric is first placed in the bottom of the trench, an inch of soil is placed on top and up the sides of the trench and erosion control fabric, and the fascine bundle is then placed in the trench and staked down as previously described. Since this is a surface treatment, it is important to avoid sites that will be too wet or too dry. Table TS14I–7 (NRCS 1996b) provides information on how to install the trenches based on the slope of the bank along the stream.

Fascines can be oriented perpendicular or at an angle to the streambank contours to provide an immediately roughened surface for erosion control. The fascines can also be arranged in a chevron pattern to create a pole drainage system for wet slopes and intercept ground water seepage (Gray and Sotir 1996).

Joint plantings

Joint plantings or vegetated riprap are cuttings of live, woody plant material inserted between the joints or voids of riprap and into the ground below the rock (fig. TS14I–25). Joint planting cuttings are 30 to 48 inches long, and from 3/4 to 2 inches in diameter. These live cuttings typically do not provide immediate reinforcement of soil layers, as they normally do not extend beyond the failure plane. The live cuttings are intended to root and develop top growth providing several adjunctive benefits to the riprap. Over time, these installations provide reinforcement to the soil on which the riprap has been placed, as well as providing roughness (top growth) that typically causes sediment deposition in the treated area. Some control of internal seepage is also provided. These joint planting installations assist in quickly reestablishing riparian vegetation. Joint plantings are frequently used on the lower part of the bank.

Materials

- Joint plantings—live cuttings 3/4 to 2 inches in diameter and 2.5 to 4 feet long. They should be long enough so that at least 1 foot of the cutting will extend into the ground below the riprap.
- Tools—machete, clippers, dead blow hammers, sledge hammer, rebar, saw, chain saw, loppers, and rebar

Table TS14I–7 Recommended spacing of fascines

Bank H:V	Bank distance between trenches in feet	Maximum bank length in feet
1:1 to 1.5:1	3–4	15
1.5:1 to 2:1	4–5	20
2:1 to 2.5:1	5–6	30
2.5:1 to 3:1	6–8	40
3:1 to 4:1	8–9	50
4:1 to 5:1	9–10	60

Installation

- Cleanly remove all side branches and the top growth from the cuttings. Cut the basal end to a 45-degree angle, or sharpen to a point. The top end should be cut flat. At least two buds or bud scars should be present above the ground in the final installation, depending on the surrounding vegetation height. The live cuttings should be taller than the surrounding vegetation to ensure that they are not shaded.
- Collect and soak the live cuttings for 14 days, or install them the day they are harvested and fabricated.
- Make a pilot hole by hammering in a piece of rebar between the rock. A steel stinger can also be used. Carefully extrude the rebar and tamp in the joint planting stem. Insert the basal end first.
- To achieve good soil to stem contact, fill the hole around the cutting with a water and soil slurry mixture.
- Plant live cuttings on 1.5- to 2-foot spacing in a random pattern or triangular grid. Spacing depends on species, moisture, aspect, and soil characteristics.

Brush layering

Brush layering consists of alternating layers of live cuttings and soil (figs. TS14I–26 and TS14I–27). The cuttings protrude beyond the face of the slope approximately 6 to 18 inches. The installed live cuttings provide immediate frictional resistance to shallow slides, similar to conventional geotextile/geogrid reinforcement. The protruding stems serve to break long slopes into a series of shorter slopes to decrease runoff erosion. The cuttings are intended to root and provide additional reinforcement to the soil. This treatment provides immediate protection against surface erosion and shallow-seated slope failure. This measure is limited to shallow-cut bank excavations, and needs to be started above a stable foundation.

Materials

- Live cuttings—3/4 to 3 inches in diameter and 3 to 6 feet long. The branches must be long enough so that the cut basal ends of the branches touch the back of the excavation, and the growing tips protrude 6 to 18 inches from the face of the slope.
- Tools—machete, shovels, mattock, clippers, saw, and hammer
- Fertilizer and other soil amendments

Figure TS14I–25 (a) Completed installation of joint planting; (b) Early in first growing season (*Photo courtesy of Robbin B. Sotir & Associates, Inc.*)

(a)

(b)

Figure TS14I–26 (a) Excavation of the brush layer bench; (b) Cutting placement (*Photo courtesy of Robbin B. Sotir & Associates, Inc.*)

(a)

(b)

Figure TS14I–27 (a) Completed installation of brush layers; (b) Results after two growing seasons (*Photo courtesy of Robbin B. Sotir & Associates, Inc.*)

(a)

(b)

Installation

- Collect and soak the live cuttings for 14 days, or install them the day they are harvested. Leave the side branches intact. Collect brushy material for this technique.
- Remove loose, failed, or failing soil from face of the bank.
- Begin excavation and installation above a stable toe structure. Like riprap, this measure requires a stable foundation.
- Excavate benches along the contour, 2 to 3 feet wide.
- Benches should be sloped 15 to 25 degrees down from the front outer edge to the back of the excavation.
- Place branches in an overlapping, crisscross configuration. Typically 12 to 24 stems per linear foot of constructed bench (measured on the contour), depending on the size of material and branching.
- Orient the stems so that the basal ends touch the back of the undisturbed, excavated bank. Approximately a fourth of the branch stem should extend beyond the completed bank face.
- Install the brush layer in three courses: layer 1, cuttings oriented to the left; layer 2, cuttings oriented to the right; and the cuttings in the final layer point straight out towards the stream. Place a few inches of soil between each layer of branches. The soil layers should be compacted by foot or with a manually directed tamper to remove air pockets. In some circumstances, amended soil (soil that has been nutrient tested and fertilized, lime has been added to enhance growth) may be used in these layers. Each course has 5 to 15 live cuttings. The completed brush layer measure is made up of 15 to 45 branches per foot. The determination of density is by the amount of available sunlight, soils, steepness, moisture, and cutting material available. Repeat until desired thickness is reached. Use the soil material from the next upbank terrace to fill the one beneath.
- Trim the terminal bud so that stem energy will be routed to the lateral buds for more rapid sprouting of roots and stems.

Construct according to the spacing shown in the table TS14I–8 (USDA NRCS 1996b).

Brush mattress

A brush mattress is a layer of live cuttings placed flat against the sloped face of the bank (fig. TS14I–28). Dead stout stakes and string are used to anchor the cutting material to the bank. This measure is often constructed using a fascine, joint planting, or riprap at the toe, with live cuttings in the upper mattress area. The branches provide immediate protection from parallel streamflow. The cuttings are expected to root into the entire bank face and provide surface reinforcement to the soil.

Materials

- Live cuttings—3/4 to 1 inch in diameter. The cuttings should be approximately 2 feet taller than the bank face. This will allow the basal ends to be placed in or at the edge of the water. Up to 20 percent of the cuttings can be dead material to add bulk.
- Dead stout stakes—wedge shaped, 1.5 to 4 feet long, depending on soil texture
- Ties—string, braided manila, sisal or prestretched cotton twine, or galvanized wire
- Tools—machete, shovels, clippers, hammer, sledge hammer, punch bar, saw, and machine to shape the bank
- Fertilizer and other soil amendments

Installation

- Collect and soak the live cuttings for 14 days, or install them the day they are harvested. Leave side branches intact.
- Cut a 2- by 4-inch board diagonally and at desired length to create the dead stout stakes.
- Excavate the bank to a slope of 1V:2H or flatter. The distance from the top of the slope to the bottom of the slope is typically 4 to 20 feet. Excavate a 1-foot-wide and 8- to 12-inch-deep trench along the toe.
- Drive the dead stout stakes 1 to 3 feet into the ground up the face of the prepared bank. Space the installation of the dead stout stakes on a grid that is 1.5 to 3 foot square. Start the lowest

Table TS14I–8 Spacing for brush layers

Bank H:V	Bank distance (ft) between benches for wet slopes	Bank distance (ft) between benches for dry slopes	Maximum bank length (ft)
2:1 to 2.5:1	3	3	15
2.5:1 to 3.5:1	3	4	15
3.5:1 to 4:1	4	5	20

Figure TS14I–28 (a) Brush mattress being installed; (b) Brush mattress after one growing season

(a)

(b)

row of dead stout stakes below bankfull width or a fourth of the height of the bank. The tops of the dead stout stakes should extend above the ground 6 to 9 inches. Live cuttings may also be mixed with the dead stout stakes, and tamped in between to add deeper initial rooting. However, the live cuttings cannot generally be driven-in as securely as the dead stout stakes and should not be relied upon solely for anchoring the brush mattress.

- Lay the live cuttings up against the face of the bank. The basal ends of the cuttings are installed into the trench with the growing tips oriented upbank. The live cuttings' side branches should be retained and should overlap in a slight crisscross pattern. Depending on the size of the branches, approximately 8 to 15 branches are installed per linear foot of bank.
- Use a fascine or some form of anchoring along the bottom portion of the brush mattress to ensure the basal ends of the live cuttings are pressed against the bank.
- Stand on the live cuttings and secure them by tying string, cord, wire, braided manila, sisal, or prestretched cotton twine in a diamond pattern between the dead stout stakes. Short lengths of tying material are preferred over long lengths. In the event of a failure, only a small portion of the treatment would be compromised if short lengths are used. Otherwise, there are risks of losing larger portions of the project if long lengths of tying material are used to anchor the cuttings to the dead stout stakes.
- After tying the string to the stakes, drive the dead stout stakes 2 to 3 inches further into the bank to firmly secure the live cuttings to the bank face. This improves the soil-to-stem contact.
- Wash loose soil into the mattress between and around the live cuttings so that the bottom half of the cuttings is covered with a 3- to 4-inch layer of soil.
- Backfill the trench with soil or a suitable toe protection such as rock.
- Trim the terminal bud at the top of bank so that stem energy will be routed to the lateral buds for more rapid root and stem sprouting.

Branch packing

Branch packing consists of alternating layers of live cuttings and soil to fill localized slumps or gullies (fig. TS14I–29). The branches protrude beyond the face of the bank. The live cuttings reinforce the soil similar to conventional geotextile/geogrid reinforcements. The stems provide immediate frictional resistance to shallow slides. Dead stout stakes are used to anchor the live cuttings or the rooted plants and provide initial support to the soil. The cuttings or rooted plants are intended to provide additional soil reinforcement within the fill area and into the surrounding gully sides and bank.

Materials

- Live cuttings—3/4 to 3 inches in diameter, cut to a length so that the cut end of the branches touch the undisturbed soil at the back of the void, and the growing tips protrude 6 to 18 inches from the face of the bank. Alternatively, or together with the cuttings, 24- to 36-inch-long bareroot plants may be used.
- Dead stout stakes—2 by 4 inches by 5- to 8-foot-long board, cut diagonally
- Tools—machete, shovels, clippers, saw, and sledge hammer
- Fertilizer and other soil amendments

Installation

- Collect and soak the live cuttings for 14 days, or install them the day they are harvested. Leave the side branches intact.
- Remove loose, failed, or failing soil from the face of the bank slump or gully.
- The installation should begin at the toe of the bank and proceed to the top of the bank.
- Construct a bench on contour. The width of the bench should approximate the depth of the gully, or the bench can be excavated so that it tilts back into the bank.
- The constructed bench should slope down into the bank at a 15- to 25-degree angle.
- Drive the dead stout stakes 3 to 5 feet into the ground in a square configuration over the formed base bench. The tops of the wooden

stakes should extend to the projected surface of the completed bank. Space the wooden stakes 1 to 2 feet apart. Alternately, live poles can be used.

- In wet areas, install a layer of rock 6 to 9 inches deep, which has been wrapped in filter fabric prior to installing the live cuttings.
- Scarify the sides of the gully or slump surface.
- Place 6 to 12 brushy live cuttings between the wooden stakes in an overlapping, fan-like configuration. Typically, 30 to 50 stems will be required per linear yard of bench.
- Orient the stems so that the basal ends touch the back and sides of the gully or slump. Approximately a fourth of the branch stem should extend beyond the completed bank.
- Backfill with a few inches of amended soil in between the installed branches and tamp by foot to ensure good soil-to-stem contact.
- Add another 6 to 8 inches of amended soil. Repeat until the desired thickness is reached. Once the soil layer is 6 to 12 inches deep, place another layer of branches over the terrace and repeat until the slump or gully is filled.
- Trim off the terminal bud so that the available stem energy will be routed to the remaining lateral buds and encourage more rapid sprouting of roots and stems.

Vegetated reinforced soil slope or vegetated soil lifts with geogrids

A vegetated reinforced soil slope (VRSS) system is made up of layers of soil wrapped in synthetic geogrid or geotextile with live cuttings or rooted plants installed in between the wrapped soil layers (figs. TS14I–30 through TS14I–33). As with brush layering or branch packing, the branches or rooted plants protrude beyond the face of the bank. The live cuttings contribute to soil reinforcement along with the geogrid. VRSS can be used to stabilize steep banks (1H:1V or greater). While this technique is not as structurally sound as a retaining wall, it is a good soil bioengineering alternative to hard engineered structures for steep sites.

Figure TS14I–29 (a) Branch packing (using live poles) under construction; (b) One growing season later

(a)

(b)

Figure TS14I–30 Fill placement within VRSS (*Photo courtesy of Robbin B. Sotir & Associates, Inc.*)

Figure TS14I–31 Geogrid wrapping of soil lift (*Photo courtesy of Robbin B. Sotir & Associates, Inc.*)

Figure TS14I–32 Completed VRSS (*Photo courtesy of Robbin B. Sotir & Associates, Inc.*)

Figure TS14I–33 VRSS development after 4 years (*Photo courtesy of Robbin B. Sotir & Associates, Inc.*)

Materials

- Live cuttings—3/4 to 3 inches in diameter, 3 to 15 feet in length, so that the basal end of the live branches touch the undisturbed soil at the back of the excavation and the growing tips protrude from the face of the bank by 6 to 18 inches
- The selection of synthetic geogrid or geotextile is based on the required parameters for stability against rapid drawdown, wedge failures, and circular failures.
- Dead stout stakes or rebar to anchor the geogrid in place
- Gravel drainage materials or in-place drainage systems, if needed
- Burlap or geosynthetic filter fabric to retain the fines
- Wood for temporary batter boards
- Fertilizer and other soil amendments
- Plastic ties, hog rings, or string

Installation

- Collect and soak the live cuttings for 14 days, or install them the day they are harvested. Leave the side branches intact.
- Start the excavation below the channel bed. Install a foundation below the anticipated scour elevation. This foundation may be composed of materials such as rock or wrapped-rock layers.
- Install a temporary batter board along the front edge of the previously constructed bench. Its location will determine the steepness and shape of the finished bank face.
- Starting on a bench of amended soil (soil that has been nutrient tested and fertilizer, lime has been added to enhance growth) above the installed foundation toe protection, ensure that the constructed bench slopes back into the bank, making the back of the constructed bench at least 6 to 12 inches lower than the front edge.
- In poorly drained soils, a drainage layer of gravel, manufactured in-place drainage systems, or a gravel, sand, or soil mix may be included to promote and improve internal drainage of the bank.
- Install the geogrid according to the manufacturer's instructions. Position the geogrid from the back of the constructed bench excavation, across the bench, and up the temporary batter board, with the remainder draping down the bank towards the stream. Securely anchor the geogrid in place at the back of the bench and along the front with rebar or dead stout stakes.
- Install burlap or geosynthetic fabric inside the geogrid at the temporary batter board. This may be held in place with plastic ties, hog rings, or string. The purpose of this fabric is to hold the fine soils until the live cuttings or rooted plants are established.
- Place fill soils on the geogrid layer to the specified depth, compacting in 6-inch lifts to 80 to 85 percent Standard Proctor Density. When the soil has been placed to a lift height of 12 to 18 inches, pull the loose geogrid back over the front and top of the soil lift. Use a lever bar or the knuckle of an excavator to drag the geogrid towards the back edge of the bench to achieve adequate tension. Anchor the geogrid using dead stout stakes. It is important that this newly created bench and the ones to follow above it also slope to the back into the bank.
- Lay live cuttings on the constructed bench with the cut basal ends in the back of the trench and the growing tips oriented and protruding over the front edge. The tips of the branches should protrude 6 to 18 inches beyond the front edge of the bench. Lay the live cuttings in three layers with a few inches of soil in between each. Lay the first layer so that tips point to the left, tips in the second layer point to the right and the third layer points out towards the stream. Typically, 5 to 15 cuttings per linear foot of bank are used, depending on the species, growing conditions, and physical parameters of the site. Container plants may also be used in place of live cuttings.
- Repeat these steps to add successive lifts of live cuttings or rooted plants, geogrid, and soil.
- The top bench is finished off so that the geogrid is buried under a soil layer with erosion control

fabric, long straw mulch and seed, or other materials used, as needed, to finish the final grade.

Brush wattle fence

Treatments are intended to promote sediment deposition and protect the bed from erosion (fig TS14I–34). They are typically installed in multiple rows along flood plains and areas adjacent to banks. Wattle fences are rows of live stakes or poles with live wattling materials woven in a basket-like fashion. The cuttings eventually root and provide a permanent living structure.

Materials

- Live cuttings—2 to 4 inches in diameter, 3 to 4 feet long
- Wattling materials—flexible branches that are 3/4 to 1 inch in diameter and 4 to 10 feet long
- Tools—machete, shovels, hammer, punch bar, clippers, and saw

Installation

- Collect and soak the live cuttings. Collect and soak wattling as described earlier. Leave side branches intact. It is important to use low-growing species that will remain supple.
- Excavate a trench that is 1 to 2 feet deep. If the treatment is located along a channel, it should be oriented at a 20- to 30-degree angle against the direction of the flow and be keyed into the bank.
- Trenches should be 10 to 50 feet apart, depending on the erodibility of the soil and gradient of the channel.
- Use a punch bar or stake to create a pilot hole at the base of the trench. The pilot hole should have a minimum depth of 1 foot below the invert of the trench.
- Tap the stake into the ground so that the sharpened basal end is inserted first. Leave about 12 inches of the live cutting exposed above the top of the trench.
- To achieve good soil-to-stem contact, fill the hole around the cutting with a water and soil slurry mixture. Add soil around the cutting. As the water percolates into the ground, the soil in suspension will settle around the cutting. Another method is to tamp soil around the cutting with a rod. To do this, throw a small amount of soil into the hole around the cutting and tamp it down with a rod to remove all air pockets. This procedure is similar to installing a wooden fence post.

Figure TS14I–34 (a) Wattle fence immediately after construction; (b) 1 year later

(a)

(b)

- Install additional live cuttings at 1- to 2-foot intervals along the proposed location of the wattle wall. If the wattling materials available do not flex easily, the distance between the live cuttings can be adjusted accordingly.
- Weave the flexible wattling material between the live cuttings in a basket-like fashion. To begin, take a length of wattling material and weave it horizontally to the left of the first live cutting, then to the right of the second live cutting, then to the left of the next. Do this over the entire length of the wattle wall. Gently compress, so this layer makes contact with the ground. Begin the next layer by weaving the wattling material in the opposite direction; begin by weaving the material to the right of the first live cutting, then to the left of the second, and so on. Once the second layer is woven, gently compress so it touches the first layer, ensuring a tight weave. Continue weaving in this manner up to the top of the wattle wall.
- Backfill the trench and tamp the soil. After installation, the area should be watered. Supplemental irrigation may also be required as the plants become established.

Top of bank/flood plain treatments

Brush trench

A brush trench is a row of live cuttings that is inserted into a trench along the top of an eroding streambank parallel to the stream (figs. TS14I–35). The live cuttings form a fence that filters runoff and reduces the likelihood of rilling. The live cuttings eventually root and provide a permanent living structure. Brush trenches are often used to supplement other soil bioengineering treatments.

Materials

- Live cuttings—3/4 to 3 inches in diameter, 2.5 to 5 feet long
- Tools—machete, clippers, shovel, saw, hammer, and excavator
- Fertilizer and other soil amendments

Installation

- Collect and soak the live cuttings for 14 days or install them the day they are harvested. Leave the side branches intact. It is important to select low-growing species that will remain supple.

Figure TS14I–35 (a) Brush trench after installation; (b) 1 year later

(a)

(b)

- Install appropriate bank and toe protection prior to excavating the brush trench.
- If a moderate amount of runoff currently flows over the bank, consider using a low berm along the top of the bank and directing the flow to a stable outfall away from the bank.
- Excavate a trench that is 10 to 12 inches wide and 1 to 2 feet deep. The trench should be no less than 1 foot back from the top of the bank so that it does not weaken the bank.
- Pack the branches tightly with the basal ends down, forming an intertwined mat. Make sure that the basal ends touch the bottom of the trench. Install 8 to 15 live cuttings per linear foot of trench. The branches protruding from the top of the trench should be taller than the height of competing vegetation.
- Avoid gaps in the vegetation.
- Fill in around the live cuttings with soil, then wash in to assure good soil-to-stem contact. All gaps between the plant materials within the trench should be filled with soil.
- Cut off the terminal end or buds to promote root growth. After the installation is completed, water the entire area. Supplemental irrigation may also be required as the vegetation becomes established.

Other techniques

Wattle fence as an erosion stop

A wattle fence can be used to deter erosion in ditches or small dry channel beds to resist the formation of rills and gullies (fig. TS14I–36). Wattle fences are rows of live stakes or poles with live cuttings woven in a basket-like fashion. The live cuttings eventually root and provide a permanent living structure. During planning and selection of wattling material, consider the potential for excessive growth clogging the channel. The planted material should be of a species that will remain supple. This treatment is not typically suitable for areas with high velocities, prolonged inundation, or significant headcuts.

Materials

- Live cuttings—2 to 4 inches in diameter and 2 to 3 feet long
- Wattling materials—flexible branches that are 3/4 to 1 inch in diameter and 4 to 10 feet long
- Tools—machete, shovels, hammer, punch bar, clippers, and saw
- Fertilizer and other soil amendments

Installation

- Collect and soak the live cuttings for 14 days, or install them the day they are harvested. Leave the side branches intact.
- Excavate a trench 1 to 2 feet deep across the dry channel or ditch.
- The trench should be keyed-in or extended 1 to 3 feet into the sides of the channel or ditch and should curve upslope.
- Use a punch bar or stake to create a pilot hole at the base of the trench. The pilot hole should have a minimum depth of 1 foot below the invert of the trench.
- Tap the live cutting into the ground so that the sharpened basal end is inserted first. Approximately two-thirds to three-fourths of the live cutting should be below the top of the trench. In addition, the top of the stakes should not be higher than one-third of the channel depth.

Figure TS14I–36 Brush wattle fence to deter erosion in a gully

- Fill the hole with water and soil slurry. Tamp the ground around the live cutting.
- Install additional live cuttings at 1- to 2-foot intervals along the proposed location of the wattle wall. If the wattling materials available do not flex easily, the distance between the live cuttings can be adjusted accordingly.
- Weave the flexible wattling material between the live cuttings in a basket-like fashion. To begin, take a length of wattling material and weave it horizontally to the left of the first live cutting, then to the right of the second live cutting, then to the left of the next. Do this over the entire length of the wattle wall. Gently compress so this layer makes contact with the ground. Begin the next layer by weaving the wattling material in the opposite direction. Begin by weaving the material to the right of the first live cutting, then to the left of the second, and so on. Once the second layer is woven, gently compress so it touches the first layer, ensuring a tight weave. Continue weaving in this manner up to the top of the wattle wall.
- The center of the wattle should be lower than the sides to reduce the likelihood of bank erosion. The sides of the wattling should be keyed-in 1 to 3 feet into the sides of the ditch or channel.
- Backfill the trench and tamp the soil. After the installation is completed, water the entire area. Supplemental irrigation may also be required as the vegetation becomes established.
- Key stones into the bed of the channel below the wattle structure for a minimum length of two times the height of the structure.

Crimping and seeding

Crimping is a surface roughening treatment that secures straw to the surface (fig. TS14I–37). It is a temporary surface treatment that protects and promotes the establishment of permanent grasses and vegetation. It can be accomplished with heavy equipment or by hand.

Materials

- Straw—avoid moldy or compacted straw
- Seeds
- Tools—shovels

Installation

- Determine approximate contour lines for installation along the slope. The contour lines should be separated by 2 to 3 feet.
- Push the shovel into the ground along the contour lines to a depth of approximately 8 inches. With the shovel still in the ground, push the shovel forward then pull it backwards to make a V-shaped indentation in the soil.
- Distribute straw along the tops of the V-shaped indentations.
- Using the shovel, push the straw into the V-shaped indentations leaving 1 to 3 inches of straw protruding above the ground surface.
- Tamp the ground by foot to close the indentation around the straw.
- Seed the area and water it.
- Place a 2- to 4-inch layer of straw between the contours. Supplemental irrigation may be required.

Figure TS14I–37 Crimped straw

Adjunctive measures

Erosion control fabric

Detailed presentation of the full range of erosion control fabrics that are available is beyond the scope of this document. However, many of these materials can help to augment or complement soil bioengineering measures for bank protection (figs. TS14I–38, TS14I–39, and TS14I–40). Various erosion control fabrics are produced from natural and synthetic materials such as erosion control blankets made of straw, wood excelsior, woven coir, or combinations of these and turf reinforcement mats produced from nondegradable, synthetic, three-dimensional fibers. Jute mesh and coir mesh are most always used on projects where wildlife habitat is a consideration because snakes, birds, and other small animals do not become entangled in these meshes as they would in nylon netting used to bind erosion control blankets. Although none of these erosion control products are designed to withstand the stress of concentrated water flow, they are useful along the upper sections of tall banks, overbank areas, or spots where overbank drainage is called for. Sometimes they can be combined with other materials, such as live cuttings or fascines, to achieve effective bank stabilization.

Materials

- Erosion control fabric—Select a product based on the manufacturer's recommendations and on shear-stress values and velocities: metal pins, plastic pegs, biodegradable polymer stakes, dead stout stakes, or live cuttings.
- Seed
- Fertilizer (use minimally where needed)

Installation

- Distribute seed and fertilize before installing erosion control blanket or turf reinforcement mat.
- Erosion control fabrics may be oriented in either direction related to the bank. However, it is important to follow the manufacturer's recommendations.
- Follow the manufacturer's recommendations for securing the edges of the fabric and suggested percent of overlap. Typically, this is accomplished with a trench and backfilling. However, other techniques such as fascines and revetments have been used to secure the edges.
- Use a quantity, pattern, and style of stakes, pins, or pegs in accordance with manufacturer's recommendations.

Integrating soil bioengineering and structural treatments

Rock is often used as a component of streambank stabilization projects. It is often used where:

- long-term durability is needed
- water velocities are high
- long periods of inundation are present
- there is a significant threat to life and property

The sizing of rock for riprap should be approached with caution, as it can be expensive and can give a false sense of security if not applied appropriately. Techniques for stone sizing are provided in NEH654 TS14C. Additional issues to consider include, but are not limited to:

- filter layer
- bank slope
- height
- thickness
- length
- tiebacks
- scour

For more information on these issues, refer to NEH654 TS14K.

Figure TS14I–38 Cutting coir fabric (*Photo courtesy of Sonia Jacobson, NRCS*)

Figure TS14I–39 Live cuttings installed in fabric

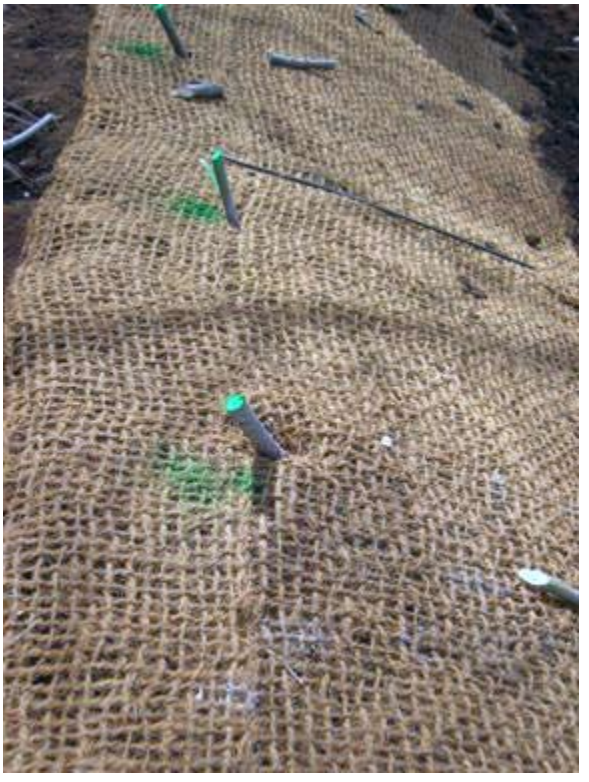


Figure TS14I–40 Combining fascines and fabric

Soil bioengineering techniques for specific climate conditions

Hot climate issues

The southern regions of the United States have warm winters which reduce the dormancy periods of plants. Furthermore, these areas have rainy winters, which produce frequent flowing water in many intermittent streams. Warmer temperatures and short dormant periods, coupled with the limitations imposed by rain, make constructing streambanks in the winter difficult and increase costs if flow diversion is necessary (fig. TS14I–41).

Plant dormancy is critical to the success of soil bioengineering techniques when live cuttings are used because materials must be harvested and installed while they are dormant. A plant becomes dormant because of changes in environments (Lang et al. 1985), normally decreasing temperature and day length (Wareing 1969). The dormancy period begins when the plant has lost its leaves and ends when new leaf buds appear along branches and stems. In general, plants in the USDA plant hardiness zones 8, 9 and 10 (fig. TS14I–7) become dormant in December and break dormancy from March to early March, depending on the latitude. This short dormancy period restricts the installation for live cuttings to a 2- to 3-month installation timeframe.

To extend the dormancy period of live cuttings, Li et al. (2004) published a study using a cold storage to extend the dormancy of live cuttings. The research was conducted in Bryan, Texas, which is in USDA Plant Hardiness zone 8b. The average annual minimum temperature ranges between 15 and 20 degrees Fahrenheit (–6.7 and –9.4 °C). Black willow (*Salix nigra*) cuttings measuring 1/2 to 1 1/2 inches in diameter were used for the study. The cuttings were harvested in March and stored in a refrigerator, maintaining a temperature range from 34 to 40 degrees Fahrenheit. While in storage, the basal ends of the cuttings were submerged in water to maintain vitality. Opaque plastic was used to block light. The cuttings were soaked prior to planting. The trial plots were planted in three separate installations during the months of March, April, and May. A single layer of cuttings was used employing the brush layering technique.

Li et al. (2004) reported survival rates of 81 percent for the cuttings installed in March, and 44 percent for materials planted in both April and May. These findings ranked as satisfactory when compared with survival rates of 40 to 70 percent reported by Gray and Sotir (1996). In conclusion, the cold storage method described appears to extend *Salix nigra's* dormancy period. The cold storage method could be considered viable in areas where warm, wet winters might otherwise rule out the use of soil bioengineering practices.

Cold climate issues

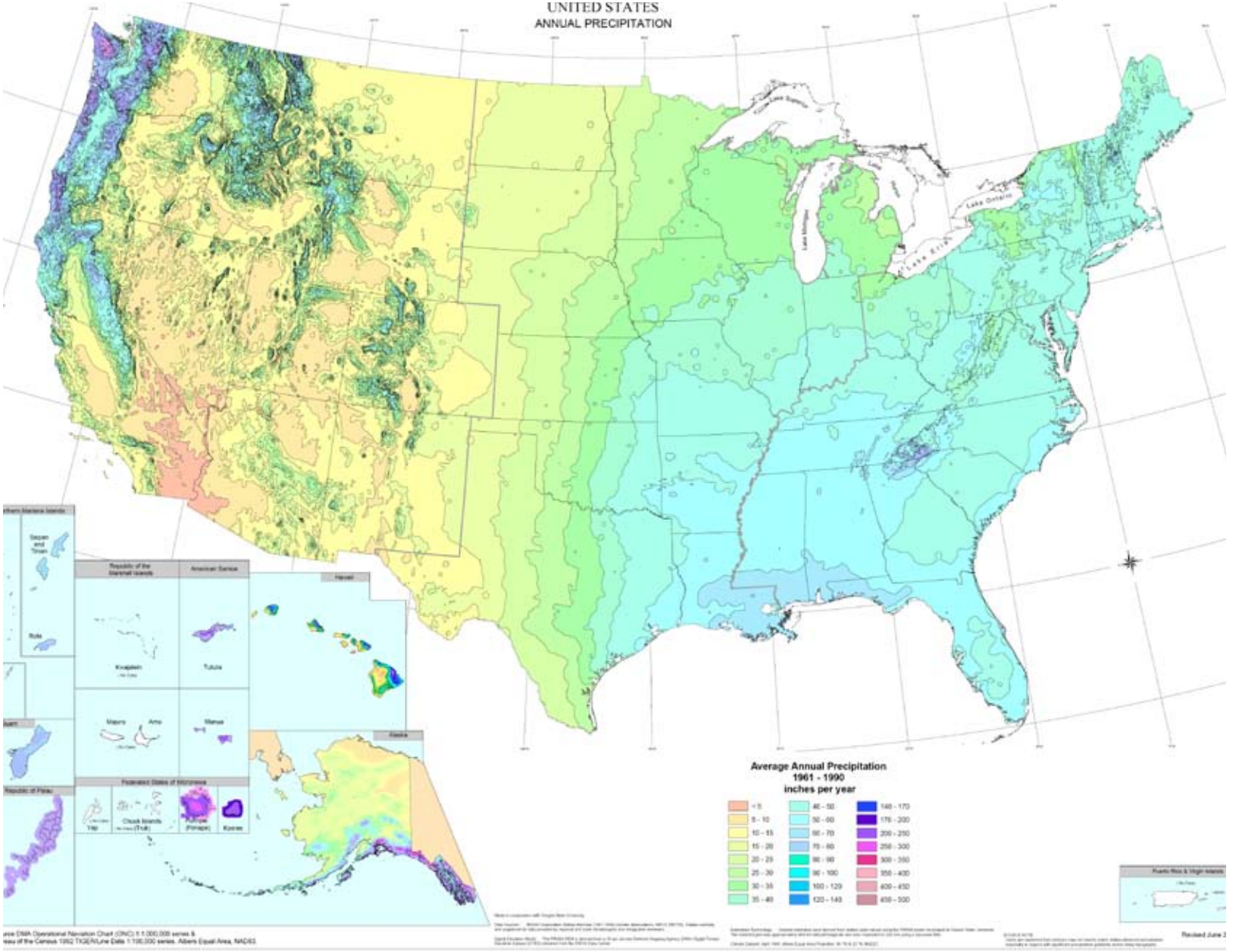
Cold climates place few constraints on the use of soil bioengineering treatments, which have been used widely in alpine areas and northern regions of Europe for hundreds of years. Many willows and other readily sprouting species are well adapted to cold conditions, and they perform excellently.

One of the main problems in cold climates is the impacts of ice flows on new installations and on developing plant stands. Many willows, dogwoods, and alders, for instance, can be sheared off by rafts of ice. This impact normally does not damage the vitality of the bank plantings, which easily resprout, but it can seriously dislodge other elements used in treating the banks.

Where ice is a factor, typically in USDA plant hardiness zone 4 or colder, it is important to use rigorous bank stabilization measures. A key to selecting streambank stabilization treatments that can withstand ice impacts is to provide full initial coverage of the bank with a durable treatment system. It is also critical to install greater quantities of well-adapted plants. They will be able to self-repair after being sheared off by ice. Live cuttings, live siltation constructions, tree revetments, and brush mattresses have been known to withstand ice impact in the toe and bank zones better than rootwad revetments, log cribs, and coir fascines. However, on poorly cohesive soils, freeze-thaw action can dislodge surface treatments, and measures that are more deeply embedded below the frost line may prove more stable.

Although planting season in cold climates is longer than in hot climates, there are some limitations that

Figure TS14I–41 U.S. annual precipitation map



affect how they are used. In warm areas, plantings can be installed throughout the winter season. This work comes to a halt in cold climates as soon as the ground freezes to the point that it is no longer workable. A shallow frost that can be broken by excavation equipment does not interfere with construction of streambank stabilization treatments or the placement of plant materials.

In winter conditions, plants face a risk of drought. Although water may be present in the soil, it is normally frozen and inaccessible to plants. The air is typically dry, and windy conditions easily lead to desiccation of plants. The best method to combat problems of cold weather desiccation is to bury up to 95 percent of the length of dormant cuttings, prune excess growth from other woody plant species, and apply a thick layer of mulch.

The structural elements of a bank stabilization treatment are typically placed first, and the plantings are added in late spring once the ground has thawed and the highest flow period, including ice break-up, has passed.

In the most extreme ranges of cold regions, conditions may not be suitable for treatments that use plant materials. This can be checked by locating healthy reference plant communities to serve as proof that the species present are able to grow under the same conditions on a nearby site. These sites often may be suitable as donor sites for collection of woody or herbaceous species for use in plantings.

One of the easiest mistakes to make is to obtain plant materials from a commercial nursery source or convenient wild collection source, then attempt to install them at higher elevations or in a more northern locale. Without proper hardening off, these plantings will often succumb to frost, dessication, diseases, and pests. It is usually acceptable to procure plants from a cold area and use them in a warmer location, but not the reverse.

High precipitation issues

It is important to have a detailed understanding of the rainfall, soils, and stream hydrology and hydraulics. Generally, the plant materials are similar to the average temperate areas. Some of the biggest differences involve how the overbank area and upper bank must be managed to properly accommodate concentrated flow from intensive runoff events. It is practical to plan on adding a berm at the top of the bank, with a swale parallel to the bank to carry water to a chosen armored point, where it can safely be carried over the bank to the stream.

Woody plants should be selected to include the widest possible array of species adapted to the region as pests, such as fungi and insects, are most likely to set in when moisture levels in the upper soil horizons remain high, and foliage, bark, and buds rarely dry out fully. Having a diverse planting plan makes it less likely that pests will destroy a planting completely and helps to keep any pest impacts far more localized. Often, seed and mulch must be used to establish vegetation on disturbed areas that would be able to regenerate on their own in drier climates.

At times, erosion control fabrics must be used to secure seed in place long enough to germinate and prevent rill formation on steep banks. Controlling how water flows over the bank is critical, as is construction phase erosion and sediment control, and restoration of all access roads and staging areas that may have been disturbed. Without proper care, these areas may collect water, and concentrated flow may cause new problems. Fortunately, areas with high precipitation tend to develop into lush and prolific revegetation once the plantings are established.

Since soil saturation can occur frequently in high precipitation areas, it is important to make sure that streambank stabilization measures address slope failure mechanisms, such as shallow slumping or deeper slides, rather than focusing on surface erosion protection alone.

Low precipitation issues

Areas with low precipitation are regions or climates where moisture clearly limits the production of vegetation (Society of Range Management 1989). In arid climates, the precipitation/potential evapotranspiration (P/PET) ratio is greater than or equal to 0.05 and less than 0.20. These regions receive less than 10 inches of average annual precipitation. In semiarid climates, the P/PET ratio is greater than or equal to 0.20 and less than 0.50. These regions receive between 10 to 20

inches of average annual precipitation (Convention on Biological Diversity 1999). Most rain falls in the spring and fall, with little in the summer months. Flooding will usually occur in the spring or early summer. High water often lasts for about a month (sometimes less in the southwestern parts of the United States), depending on climate, rainfall, snow pack, and elevation. Once the high water has passed, the water level will decrease through the summer. This often leaves the toe zone exposed, particularly on smaller river systems. In arid and semiarid regions, water removal from dams, diversions, or irrigation pumping will often exacerbate the problem.

The main limiting factor in a low-precipitation area is water availability. The amount and seasonal availability of water is important for the long-term survival and plant range of adaptation. Knowing the lowest water table level for the year and ensuring the live cuttings and plants reach that point is crucial to their survival. Plants may require additional moisture during the establishment period. In fact, many plants can survive with much lower precipitation amounts if they are established with supplemental moisture (Hoag and Fripp 2002). In low-precipitation areas, drought tolerance is a major consideration when selecting a plant species.

In some areas of the western United States where irrigation of cropland is important, stream channels are used to convey water from a reservoir to irrigated farmland. This causes the water level to be higher in the middle of the summer when water levels should be low, and establishing vegetation in the bank zone becomes problematic.

Supplemental watering

Supplemental deep watering is typically required if the lowest water table cannot be accessed by the roots of live cuttings. For many streambank stabilization projects, temporary irrigation is a practical way to ensure that plants become established in droughty soils, arid regions, or if there have been delays in the project's schedule that have pushed the planting operations beyond the seasonal beneficial rains. Supplemental watering should be adjusted as the plantings become established. Initially, watering should be frequent and of short duration. As the plants get bigger and more roots have developed, the watering frequency should be less often and of longer duration to encourage deeper rooting.

In some cases, when major construction is part of the design, inexpensive drip tape can be installed in the rocks or layers to water the plantings. This tape can be used until the plants are large enough to survive without supplemental watering, and then the tape can be abandoned. This method will generally reduce the cost of the project, and the soaker tape can be placed directly in the root zone of the plants to provide more efficient watering.

Excessive irrigation can be damaging to a project. Interestingly, some areas with average or low precipitation can experience the problems associated with high precipitation areas due to poor design or management of irrigation. If a temporary irrigation system is not carefully designed, excessive amounts of water can be delivered to the bank. This can cause loss of seed and mulch, gullying of the bank, and may trigger major slumping in saturated soils. If using poorly adapted irrigation equipment, design highly sturdy treatments or perform a detailed evaluation of the pump, piping, and sprinkler equipment that will be used and the frequency and duration of watering to prevent excessive artificial precipitation.

Water-retaining soil amendments

Plant mortality is often around 20 percent under good conditions and as high as 80 percent in arid areas or where drought conditions occur and soil fertility is poor. Because of this, there is growing interest in the use of soil amendments that retain and release water over time.

Water-absorbent polymers are polyacrylamide-based granules designed to absorb up to 400 times their own weight of water and release it slowly back to plant roots over a period of months or even years. The granules are recharged by precipitation, irrigation, or hand watering. Fine and medium-grade consistencies of the product are sold. Fine-grained granules are mixed with water and applied to bareroot seedling roots prior to planting. Medium-grained granules are saturated and then either mixed with backfill or placed directly in the planting hole.

A solid water product made of 98 percent water and 2 percent food-grade cellulose and alum is also available. Soil bacteria degrade the cellulose, releasing the water slowly and continuously. The number of applications needed for plants to become established has not

yet been fully documented, but is likely influenced by factors such as depth to ground water, frequency of seasonal rainfall, temperatures, soil type, plant species, elevation, and drying winds. This product is sold under the name DriWater®. The efficacy and cost-effectiveness of water-retaining soil amendments in arid and semiarid regions are being assessed by the Environmental Laboratory at the USACE Engineer Research and Development Center (Fischer 2004).

Installation equipment and tips

Dead blow hammer

When installing poles by pounding them into the ground, it is helpful to use a sand-filled dead blow hammer or a rubber mallet. This simple and readily available tool helps prevent splitting of the cutting, mushrooming of the top end, and other forms of damage to the plant, thereby increasing its odds of survival and growth. If a live cutting is cracked during installation, it should be cut at least a half-inch below the crack to reduce the possibility of insect infestation. If the height of competing vegetation is an issue, it may be necessary to plant additional cuttings in close proximity.

Stinger (metal)

The stinger, a metal tool to plant unrooted hardwood cuttings of willow and cottonwood species for riparian or shoreline erosion control or rehabilitation, was designed and built specifically for planting into rock riprap (figs. TS14I–42). In the past, unrooted, woody vegetation has been planted into rock riprap, but planting methods have concentrated on inserting the cuttings in the ground first and dumping rock on top of them or planting through riprap with a steel bar or waterjet (Schultze and Wilcox 1985). These methods are not very efficient and have not achieved great success. The stinger, however, builds on these methods and utilizes the power of a backhoe to plant larger diameter and longer unrooted cuttings than was possible before. The stinger can plant unrooted cuttings through rock riprap with minimal effort to better stabilize the rock. This method allows the placement of cuttings above the ice layer where they will not be torn out by the force of the ice. The method also improves the aesthetics of riprap.

The stinger fits on the end of a backhoe arm in place of the bucket. It is constructed by welding a long round bar to a support frame. The support frame is attached to the backhoe arm, using the same pins as the bucket, after the bucket is removed. The upper hydraulic ram on the backhoe arm moves the bar forward and backward so the holes can be punched at almost any angle. See the specification sheet and drawing for actual design. The entire attachment weights about 900 to 1,000 pounds and can be transported either attached to the backhoe arm or in a pickup truck. It was designed to be heavy enough to punch a hole down through the spaces between large rock riprap into moist to wet soil underneath. Once the stinger reaches the soil under the rock riprap, it is pushed in deep enough to make a hole that allows the placement of cuttings in moist soil.

The willow or cottonwood pole is inserted part way into the hole. A metal cap is placed over the top of the cutting and the tip of the stinger is placed on the top of the cap. The backhoe operator then pushes the stinger down, pushing the cutting into the hole. Only 1 to 5 feet of the cutting should remain above the rock surface. The majority of the cutting (2/3 to 3/4 of the length) should be in the ground.

The stinger can plant 3 to 6 inches in diameter by 4- to 12-foot-long unrooted willow and cottonwood cuttings directly through riprap. This size cutting has had excellent establishment success when two key planting guidelines are followed: the cuttings should be planted deep enough to be in permanently moist soil; and the cutting tops should extend 1 to 5 feet above the high water level.

For reservoirs used for irrigation purposes, cuttings should be planted 1 vertical foot below the high waterline in the spring of the year for best results. Plant the cuttings when the water level has dropped 2 vertical feet or more below the high waterline. If plantings are planned on reservoirs that are operated differently, care should be taken to ensure that the cuttings are in moist soil during the growing season, but not inundated longer than 1 month. Once established, cuttings can be inundated for longer periods of time.

If shoreline erosion control is the primary purpose of the planting, always plant in layers using different types of willow or cottonwood species. Shrub-type willows should be planted first, and tree-type willows or cottonwoods should be planted farther up the bank. The shrub-type willows intercept the wave first and absorb some of its erosive energy. Shrub-type willows have more flexible stems that will bend and not break. Tree-type willows or cottonwoods have less flexible stems, but have deeper root systems and larger trunk diameters that can withstand more wave energy (Hoag and Ogle 1994).

If the planting site has been riprapped, plant one row of shrub-type willows about 4 to 6 feet apart and one row of tree-type willows or cottonwoods about 5 to 8 feet up the bank on a 10- to 12-foot spacing. The spacing depends on the type of maintenance that is planned for the planting site. Plant at a wider spacing if equipment will be used to pull rock riprap back up the bank as part of a regular maintenance schedule.

If the planting site has not been riprapped and has a vertical slope, common in riparian corridors, plant each layer with a narrower spacing and the cuttings closer together to provide better protection for the exposed soil. Shrub-type willows have been planted as close as 1 to 2 feet apart, while the tree-type willows have been planted as close as 5 to 6 feet apart.

The primary limiting factor for establishing cuttings is moisture. The key to good establishment is placing the cuttings into permanently moist soil where competition from the roots of the surrounding vegetations is significantly decreased (Hoag, Young, and Gibbs 1991).

When planting unrooted cuttings into rock riprap, vertical cutbanks, or eroded streambanks, insert them at a 45-degree angle to the water surface. This will protect the cuttings from damage caused when the bank above the cutting sloughs off and crashes down onto the stem. This sloughing can cause a vertically planted cutting to break off. This technique also reduces the damage the cutting could sustain from heavy wave action, floating debris, or floating ice chunks.

A maintenance schedule is very important for the first 2 years following the planting. Dead cuttings should be replaced as soon as possible to prevent holes in the vegetative armor that could allow excessive wave energy to impact the shoreline. The longer the period between planting and replacement, the higher the potential erosion hazard to the shoreline or streambank (Hoag, Young, and Gibbs 1991).

Waterjet hydrodrill

The waterjet hydrodrill or waterjet stinger was specially designed to use high-pressure water to hydrodrill a hole in the ground to plant unrooted hardwood cuttings into riparian revegetation (figs. TS14I–43 and TS14I–44). Typically, cuttings are installed so that the bottom of the cutting is about 1 foot into the lowest water table. A waterjet hydrodrill is a useful tool for creating a planting hole with adequate depth to the water table. It is especially useful in semiarid regions where the water table may be 3 to 6 feet below the surface. This device has also been used in areas of high precipitation to accelerate the installation of large numbers of cuttings.

This device consists of a stainless steel nozzle welded to the end of a 3/4-inch pipe. A valve is fixed to the top to control flow. A T-handle is welded near the top to aid in the planting operations. The probe is connected by a garden hose to a high-pressure pump. A pressure relief valve is included on the pump for safety. The requirements for the pump include:

- gasoline powered
- small enough to be transported
- minimum 80 pounds per square inch output
- 120 gallons per minute output
- minimum vertical lift of 18 feet

The waterjet is operated by placing the nozzle against the ground and turning on the valve. As the water jets out, the waterjet probe slowly works its way into the ground. If it hits a hard layer, it may slow or stop, but the jet should eventually work through it. If obstructions are encountered, the user will need to wiggle the jet back and forth until the water can find a way around it. Once the desired depth is reached, the user should pull the waterjet out of the hole while continuously rocking it back and forth to create a larger hole.

It is important that the operator not allow significant amounts of sediment to bubble up out of the hole while

Figure TS14I–42 Stinger

Figure TS14I–43 (a) Water jet nozzle; (b) Stinger

(a)

(b)

Figure TS14I–44 (a) Water jet pump; (b) Equipment on trailer

(a)

(b)

drilling. The more sediment that is allowed to bubble out, the more sediment that will have to be replaced after the water moves into the surrounding soil. The cuttings should be pushed into the hole immediately after it has been jetted, to avoid having it collapse or fill with sediment. The waterjet does not work well in large gravels and cobbles. It works best in fine-textured soils. It will work in sands if the cutting is pushed into the ground at the same time as the probe.

Muddying-in

Good soil-to-stem contact is critical for successful establishment of most dormant unrooted cuttings. This can be achieved by muddying them in. Muddying-in the cuttings means pouring a slurry mix of water and soil into the hole around the cutting stem. The slurry mix will flow around the cutting, completely displacing any air pockets and creating good soil-to-stem contact. As the water percolates into the surround soil, the soil that is in the slurry will settle tightly around the stem, improving rooting success. Using the waterjet can accomplish the same thing.

Holding ponds

All dormant cuttings benefit from being soaked prior to installation. Ideally, this will be for 14 days. The soaking process can occur in an existing pond, backwater zone of a river, a small plastic-lined pond, or anything that will hold water. The goal is to fully submerge the dormant cuttings. Soaking the cuttings allows the plant tissues to fully hydrate. It also causes the root buds to start growing. The roots will emerge from the bark in about 14 days. The tender emerged roots will rub off when the cuttings are planted, so remove the cuttings from the water just before the roots emerge. Once planted, the cuttings will root into the soil much faster than they would if they have to absorb water from the surround soil (fig. TS14I–45).

If the entire cutting is soaked in a cold water pond, preferably in shady conditions, it can prolong the dormant period, allowing a project to proceed effectively even when the construction schedule is lagging. The entire cutting should be submerged while soaking especially as the air temperature rises. Lastly, placing the cuttings in a holding pond facilitates inventorying of plant materials to be assembled and safely stored, allowing an efficient and uninterrupted process of collection and installation (NEH654 TS14I–46).

Figure TS14I–45 Cuttings with basal ends submerged in a pond (*Photos courtesy of Robbin B. Sotir & Associates, Inc.*)

Sealing or marking paint

Once dormant cuttings are collected, it is difficult for most people to accurately identify one species of plant from another and keep track of which end is up on a cutting. One of the simplest measures for addressing this need is to have various colors of marking paint available for coding plants and tagging the top (shoot) end of the cuttings (NEH654 TS14I).

Bundle the cuttings, and tap the tops on the ground to ensure the tops are even. Pour about 1 inch of a mix of 50 percent water and 50 percent latex paint into a flat pan or bucket. Dip the top inch of the bundle briefly into the paint/water mix, and stack the bundles to dry. This treatment does not harm the plants. Its purpose is to prevent desiccation and make it easier to identify the species once they are planted. In addition, it reduces the chance that the live cuttings will be planted upside down.

Painting the tops of the cuttings is handy for a construction inspector who can far more readily spot the cuttings, and it helps the crews keep track of where to plant them. It also is indispensable in monitoring to assess how well the plantings have developed.

Figure TS14I–46 Soaking willow cuttings at Fox Creek, Driggs, ID

Construction scheduling

Many of the soil bioengineering treatments outlined here depend on installation of inert elements, as well as plant materials. To accommodate the inevitable delays in construction project scheduling, it is useful to realize that the structural phase can often be done first, during a season that is not amenable to planting, and the planting can be scheduled immediately thereafter or added the following year.

Plant protection

One benefit of soil bioengineering treatments is their ability to provide habitat by serving as a food source, but too much of a good thing can be destructive to the success of the project.

Newly planted shrubs along the riverbank may be grazed by deer and other herbivores. This becomes an issue in harsh climate, such as cold, dry climates, where vegetation is scarce. Although most stands of plants can tolerate being trimmed down to the ground once a year (assuming they have initially rooted and become established onsite), continual grazing pressure may exceed the ability of plants to maintain the health of their root system and regenerate.

There are a number of different options to protect the cuttings until they have rooted or to protect mature riparian woody plants. Often, fencing that surrounds either the entire planting, or sections of it, is the best solution. After 3 to 5 years, the plants no longer require that protection. Similarly, tree plantings in riparian corridors frequently come under pressure from deer or small mammals that eat their bark at the snow level. Wire rabbit mesh or commercially available plant protection sleeves can prevent this damage (fig. TS14I–48).

Particular care and attention should be exercised when using tall plant protection sleeves in very hot areas because they can act as super hot greenhouses that will cook the tender plants inside. In areas that have moose and elk, 6-foot-high horse fence that is tied into 3- to 4-foot circles and placed around the trees will prevent them from putting their heads over the top of the fencing and eating the apical bud. The circle should be wide enough to prevent wildlife from eating any branches that are close to the sides.

Figure TS14I–47 (a) Cottonwood cuttings being dipped into a mixture of paint and water to seal the tops; (b) Cuttings that have been sealed with paint

(a)

(b)

Figure TS14I–48 (a) Tree cage built out of 6-foot-high horse fence; (b) Example of a tree protection sleeve

(a)

(b)

Where beaver are a problem, cover the bottom of the cutting with paint that has sand added to it. When a beaver starts to chew on the bark and gets a mouthful of paint and sand, further browsing is normally deterred.

Where livestock or wildlife are known to be present, fencing to exclude them is the best defense. Livestock fencing works well for horses, sheep, and cattle, but it is normally not high enough to exclude deer. A double row of higher fencing (8 ft or more) can be effective. Chemical repellents that are commercially available work to some degree, but should not be relied upon by themselves. They must also be recharged on a regular basis, especially after heavy rainfall. Muskrat and beaver can be excluded from a site by using heavy-gauge, welded wire or hardware cloth, which can be buried into the ground or used as a covering below the topsoil layer. Metal posts should be used to support the wire, since beavers will gnaw through wood. For beaver exclusion, the fencing should be at least 4 feet high (higher if deep snow is deposited during the winter). Goose fencing must break up the site into small cells, typically no larger than 6 feet by 10 feet in area and must be made of multiple-strand string or wire fencing to prevent them from landing inside the cells. Chicken wire should be placed around the base of the cells to prevent the geese from swimming or walking under the strings. Preventing or responding to problematic levels of herbivory can be costly. Use local knowledge and experience to effectively design and maintain protective measures.

Soil compaction

Few standards exist for determining ideal design parameters for soil compaction when installing vegetation for stabilization and erosion control purposes on slopes and banks. Geotechnical engineers regularly recommend the highest practical soil compaction based on data correlating soil density with increased mechanical strength. Agronomists, on the other hand, recommend minimal soil compaction because compacted soils impede the growth and development of crops, forests, and native plant communities.

Generally, a compaction rate of between 80 and 85 percent of the standard Proctor maximum dry density optimizes slope stability with vegetation development and growth (Goldsmith, Silva, and Fischenich 2001). A soil compacted between 80 and 85 percent Proctor will not provide a significant engineering function to the stability of slopes, but it will provide a suitable environment for roots to grow.

There is usually some delay between the introduction of vegetation and the start of its active growth. If the slope is in a critical condition at this stage, a high degree of compaction may protect the slope against failure, but root growth may be restricted. In this situation, the geotechnical requirements should be addressed using some initial safeguard against failure such as biodegradable and synthetic geotextiles, live or dead wooden stakes, metal pins or spikes, soil nails, or a retaining structure. This provides a temporary engineering function until vegetation takes root and grows.

Planting plans

The restoration of vegetation for the entire riparian zone is essential for improving a range of wildlife and aquatic habitats. Such efforts require that the dynamic processes of establishment, growth, and succession of riparian plant communities be allowed to occur. Overbank, transitional, and upland areas are often restored using a variety of plant stock types. Vegetation experts (plant specialists, landscape architects, agronomists, botanists, and biologists) must inventory the existing vegetation (including noxious species) and gather information about the soils and migratory paths. They combine this data with the project's goals and objectives prior to making decisions about species selection, plant stock types, planting density, and wildlife habitat value. The information gathered should also consider the types and sizes of vegetation on adjacent property and whether these will impact the proposed planting plan. Any existing vegetation that is to remain on the site should be identified and protected during the construction process. Invasive species may also need to be eradicated.

If seedlings for broadleaf species will be used, ensure that they are a minimum of 3/8-inch caliper size, measured 1 inch above the root collar. Coniferous species must have good balance between top and root. Seedlings should be a minimum of 3/8-inch caliper size, measured 1 inch above the root collar, and should be about 2 to 3 years old and at least 18 inches tall. They

can be planted manually or mechanically. The hole should be deep enough so that when the plant is in place the root crown (where the root cells meet the stem cells) is level or slightly below the finished grade. Vegetation should be set plumb in relation to surrounding topography.

Over planting with seedlings of shade intolerant pioneer species that grow quickly can close the canopy rapidly and may accelerate succession. Seedlings of shade-tolerant dominants can be introduced to this mix so that with time, they will replace the pioneering species. Over planting with canopy species helps reduce the prevalence of browsers that would otherwise devour the smaller specimen-dominant species. Seedlings vary in price depending on species, source, and whether they are bareroot or containerized ($2.50–$12 per plant for containerized material versus $0.35–$1.50 per plant for bareroot plants).

Container-grown seedlings have better survival rates and greater root mass than their bareroot counterparts, making them better equipped to deal with drought. While they can be shipped most of the year, installation should be done within their prescribed planting times. Controlling herbaceous material after planting operations and during establishment is critical to the overall survival rate of seedlings; therefore, long-term maintenance costs must be factored into the project budget.

Bareroot stock is generally easier to plant, cheaper, and more available than containerized stock. Weed control and initial site preparation are very important for successful establishment of bareroot stock.

Where instantaneous results are desired (high-use recreation sites or projects adjacent to urban areas), large containerized and balled-and-burlapped (B&B) stock should be used. The public's perception of success or failure of a project may be based solely on aesthetic qualities. These stock types are the most expensive. Prices start at $8 per plant (2005). Installation costs vary from $10 to $30 per plant. The most popular planting size is 1 1/2- to 2 1/2-inch caliper, measured 6 inches above the root crown. This size makes them larger, heavier, and more awkward to install on most streambank projects, when compared to bareroot stock. However, they will also speed up the establishment of wildlife and fish habitat, restore missing riparian functions, and improve overall aesthetic appearance. Adding in a few large containerized or B&B plants as specimen plants and concentrating on using smaller containerized plants or bareroot plants around them will give the planner the best of both worlds.

After the planting plan is developed, the next step is to mark plant locations in the field. Plant materials should be arranged randomly unless mowing will be used to control herbaceous material. For large-scale restoration projects, it is not necessary to mark the location of each individual plant, as long as the planting crew has a general understanding of the planting plan. This is not the case for high visibility projects or when working with crews who have little experience in restoration work. Planting season varies depending on species and geographic location. Generally, woody-stemmed materials are planted when the vegetation is dormant, from leaf fall in the late fall, to bud break in the spring. When planted in late fall, the roots of deciduous vegetation continue to grow as long as soil temperature remains above 45 degrees Fahrenheit. Soils must remain moist; otherwise, a severe winter storm will kill the vegetation. Installation generally starts at the toe and progresses up the bank.

All the efforts, knowledge, and resources invested in today's planting plan may not resemble tomorrow's plant community. Stream restoration and improvement requires a long-term perspective. Natural disturbance regimes influence the functions of riparian vegetation until the vegetative communities become stable. Through succession, the landscape becomes more refined by becoming more integrated, diversified, and complex.

Soil bioengineering projects often install a monoculture that consists mainly of the pioneering species. Depending on project details, it may be possible to accelerate the vegetative succession process by selecting a few species that would typically be found in a later successional phase. This is difficult since many late successional-stage plants need early successional-stage plants to create the right conditions like soils, nutrients, microbial populations, and shade for those species to establish. Given the complexities and uncertainties about the use of vegetation, a plant specialist is the most knowledgeable person on the design team to develop a planting plan and decide which plants are best suited to a particular site.

Monitoring and maintenance

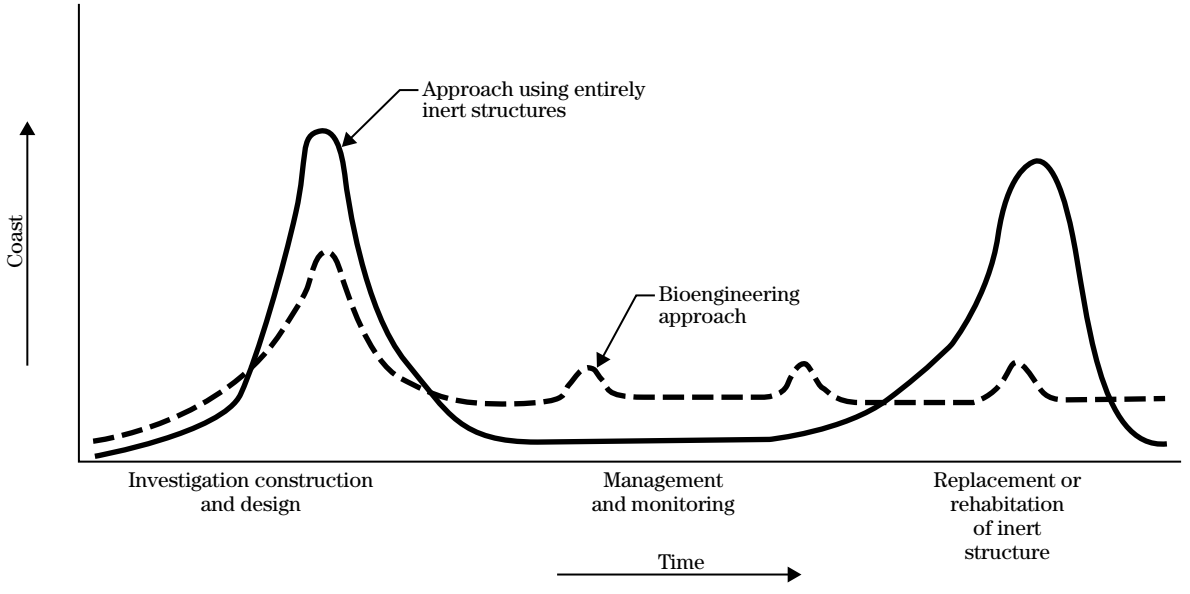
While soil bioengineering projects tend to be self-renewing and grow stronger with time, project areas require periodic monitoring and maintenance, particularly during the establishment stage. Maintenance is especially important on highly erosive sites. Maintenance could include removal of debris and elimination of invasive or undesirable species, as well as replanting vegetation in spot areas. The idealized plot in figure TS14I–49 (Coppin and Richards 1990) compares the cost of traditional inert bank protection to soil bioengineering approach. The plot illustrates that a soil bioengineering approach requires some expenditures for monitoring and maintenance, while an inert structural approach has higher initial costs, minimal or no maintenance, but eventual replacement (Allen and Leach 1997). The plot also illustrates that the reconstruction costs of a soil bioengineering approach are often significantly less than those associated with inert structures.

The success of a soil bioengineering streambank stabilization project obviously depends on the establishment and growth of the vegetative component. Allen and Leach (1997) noted that it is important to monitor soil bioengineering projects after project completion to assure plant survival and development. For example, supplemental irrigation may be necessary for exceptionally dry conditions. A fungicide or insecticide may need to be applied if insects or disease are an issue. Beaver, geese, livestock, moose, elk, and other herbivores may also eat the plants in a streambank soil bioengineering project. The loss of a predetermined percentage of the planting may be used to trigger a requirement for remedial planting.

If a moderate storm occurs before establishment of the vegetative component of a streambank soil bioengineering project, there is a potential for significant damage to the project. In fact, depending on the nature of the stream and the project, this damage may be severe enough that the vegetative component of the project may not recover. Therefore, it is recommended that most soil bioengineering projects be inspected after moderate flows, as well as on a periodic basis. These inspections are often enough to determine if remedial action will be necessary.

One of the most common problems identified with newly installed bioengineered treatments is herbivory, or consumption by plant-eating animals. At times, Canada geese or muskrats may decimate a new her-

Figure TS14I–49 Illustration of expenditure profiles for soil bioengineering and inert structures



baceous planting, or beaver may trim every shrub and tree sprout down to ground level. This comes as a shock and disappointment when it occurs, especially after completing a project or even after a robust initial growing phase. Most woody plantings rebound quickly from such impacts, and therefore, can be considered indications of beneficial habitat use. Many herbaceous plantings also rebound well, but if unrooted or repeatedly grazed down to the ground, the damage can be permanent. If this is a possibility, it may be advisable to provide a measure in the plans for inspection and replacement of lost material.

Conclusion

Streambank soil bioengineering is the use of living and nonliving herbaceous and woody plant materials in combination with natural or synthetic support materials for slope stabilization, erosion reduction, and vegetative establishment. This technique has a rich history and uses plants and sometimes inert material to increase the strength and structure of the soil. The use of streambank soil bioengineering treatments is increasing in popularity for a number of reasons: improved aesthetics, increased scrutiny by regulatory agencies, improved water quality benefits, restored fish and wildlife habitat, and decreased costs.

The long-term goal of many streambank soil bioengineering stabilization projects is to mimic natural conditions within a natural or newly altered regime. Unaltered channels in their natural environments can be expected to move and erode during large storms. Therefore, where the goal is to allow the system to remain natural, the bank will likely not be static, and periodic bank erosion should be expected. This condition can be contrasted to more urban situations where the proposed conditions of the channel typically do not allow for bank erosion. In these cases, the selected streambank soil bioengineering methods incorporate hard or inert elements that can handle higher velocity flows and to limit the flexibility of the protected bank.

Many types of soil bioengineering treatments can be used to stabilize streambanks and can withstand varying shear limits and velocities. Streambank soil bioengineering treatments are a viable alternative to hard structures, as long as the risks are clearly understood and planned for. Understanding the riparian planting zones is particularly important to ensure that the vegetation is planted in the right zone.

Technical Supplement 14J

Use of Large Woody Material for Habitat and Bank Protection

(210–VI–NEH, August 2007)

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Cover photos: *Top*—Logs and rootwads may be designed to protect eroding streambanks.

Bottom—Large woody material is an important ecological component of many streams in the United States.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14J

The Use of Large Woody Material for Habitat and Bank Protection

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Technical Supplement 14J

The Use of Large Woody Material for Habitat and Bank Protection

Introduction

Large woody materials (LWM) have been used for river training and stabilization for centuries. Many of the earliest river training structures built on large rivers in the United States included willow mattresses, brush mattresses, or wooden pilings driven into the bed. More recent efforts include tree revetments and other structures featuring large wood that were placed in the Winooski River, Vermont, in the 1930s, as part of a successful comprehensive watershed stabilization project (Edminster, Atkinson, and McIntyre 1949; U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) 1999a). A wide-ranging federally funded streambank protection research and demonstration program in the 1970s included several field trials of LWM-based protection schemes (U.S. Army Corps of Engineers (USACE) 1981). Most of these installations produced favorable short-term results for erosion control and in terms of costs, although some projects were damaged by ice (Henderson 1986).

In the 1970s, George Palmiter developed a suite of techniques involving repositioning LWM for controlling erosion and high-frequency flooding along low-gradient, medium-sized rivers clogged with debris and sediment. His approach featured use of hand tools and small power equipment (Institute of Environmental Sciences, Miami University, 1982; National Research Council 1992). A 1986 evaluation of 137 log habitat structures in the Northwest revealed high rates of damage and failure (Frissell and Nawa 1992).

During the 1990s, increasing appreciation of the importance of large wood in natural riverine ecosystems triggered efforts to design structures that emulated the form and function of naturally occurring, stable accumulations of wood, particularly in rivers of the Pacific Northwest (Abbe, Montgomery, and Petroff 1997; Hilderbrand et al. 1998). However, rootwad composites, which are currently among the most popular types of large wood structures, do not resemble any commonly observed large wood formations.

Area of applicability

LWM structures are intended to provide habitat and stabilization until woody riparian vegetation and stable bank slopes can be established. LWM decays within a few years unless it is continuously submerged. Therefore, structures made entirely or partially of woody materials are not suited for long-term stabilization unless wood is preserved by continuous wetting or with chemicals. Woody structures are best applied to channels that are at least moderately stable, have gravel or with finer bed material, and need wood for habitat. More detailed criteria are summarized in table TS14J–1 (adapted from Fischenich and Morrow 2000).

Woody material structures, like most bank protection, are not suited for reaches with active bed degradation. Streams not transporting sediments or steep, high-energy systems transporting large cobbles and boulders are usually not good candidates for woody material structures. Although there are many examples of woody material projects, the basis for design is somewhat limited by a lack of quantitative data for design, performance, and environmental effects. Furthermore, many of the most important design variables are regional or site specific. An overview of published values computed or assumed for key design variables is provided in table TS14J–2. This table is intended to provide an impression of the limitations of current design criteria, and suggested design values are presented. Long-term performance information is limited (Thompson 2002; USDA NRCS 1999a). Accordingly, wood structures are not well suited for high-hazard, high-risk projects.

Environmental and habitat considerations

Although early interest in the use of wood structures for stream stabilization was driven by the need for low-cost approaches, current understanding includes consideration of the important role that woody materials play in creating and providing the diverse conditions typical of aquatic habitats (Gurnell et al. 2002). Knowledge regarding geomorphic and ecological functions of wood in rivers is rapidly increasing. Considerable evidence suggests that streams across North America were dominated by inputs and large accumulations of woody materials prior to European settlement (fig. TS14J–1).

Table TS14J–1 Limitations on applicability of large wood structures

Variable	Considerations
Habitat requirements	Provides physical diversity, cover, velocity shelter, substrate sorting, pool development, undercut banks, and sites for terrestrial plant colonization using natural materials
Existing LWM density	Absent or depressed relative to similar nearby reaches that are lightly degraded
Sediment load	Generally not suitable for high-energy streams actively transporting material larger than gravel. LWM structures may be rapidly buried in high sediment load reaches, diminishing their aquatic habitat value, but accelerating recovery of terrestrial riparian habitats
Bed material	Anchoring will be difficult in hard beds such as cobble, boulder, or bedrock
Bed stability	Not suitable for avulsing, degrading, or incising channels. The best situations include areas of general or local sediment deposition along reaches that are stable or gradually aggrading. Deposition induced by LWM structures may be stabilized by planted or volunteer woody vegetation, fully rehabilitating a naturally stable bank by the time the placed woody materials decay (Shields, Morin, and Cooper 2004). Unlike some of the other structures, rootwads often create scour zones, not deposition
Bank material	LWM structures placed in banks with >85% sand are subject to flanking
Bank erosion processes	Not recommended where the mechanism of failure is mass failure, subsurface entrainment, or channel avulsion. Best when toe erosion is the primary process
Flow velocity	Well-anchored structures have been successfully applied to situations with estimated velocities —2.5 m/s (D'Aoust and Millar 2000). Rootwad installations have withstood velocities of 2.7 to 3.7 m/s (Allen and Leech 1997). Engineered logjam (ELJ)-type structures withstood 1.2 m/s in a sand-bed stream (Shields, Morin, and Cooper 2004)
Site access	Heavy equipment access usually is needed to bring in and place large trees with rootwads
Conveyance	LWM structures can increase flow resistance if they occupy significant parts of the channel prism (Shields and Gippel 1995; Fischenich 1996)
Navigation and recreation	LWM should not be located where they will pose a hazard or potential hazard to commercial or recreational navigation. Potential hazards are greatest for structures that span the channel
Raw materials	Suitable sources of trees needed nearby
Risk	Not suited for situations where failure would endanger human life or critical infrastructure

Table TS14J–2 Published values for design variables for LWM structures

Quantity	Used for	Typical values	Source
Density of wood in g/cm^3 (lowest, or worst-case condition[1/])	Buoyant force computation	0.4 to 0.5 0.5 0.4 to 0.5	Shields, Morin, and Cooper (2004) D'Aoust and Millar (2000) D'Aoust and Millar (1999)
Drag coefficient	Drag force computation	0.7 to 0.9 Up to 1.5 0.4 to 1.2 1.0 1.2 to 0.3 (tree) 1.2 (rootwad)	Shields and Gippel (1995) Alonso (2004) Gippel et al. (1996) Fischenich and Morrow (2000) D'Aoust and Millar (2000) D'Aoust and Millar (1999) D'Aoust and Millar (1999)
Design life for wood, yr	Planning	5 to 15	Fischenich and Morrow (2000)
Soil strength	Analysis of loads/ anchoring provided by buried members	Soil forces on buried members neglected in order to be conservative. Range of values based on soil types	Shields, Morin, and Cooper (2004)

1/ Worst case conditions presume well-dried wood. Dry wood rapidly absorbs water and may increase its density by 100% after only 24-hr submergence (Thevenet, Citterio, and Piegay 1998). However, critical conditions, especially along smaller streams, are likely to occur before wood has had time to fully absorb water.

Figure TS14J–1 Large historical logjams of LWM, Great Raft, Red River, LA

Native communities of plants and animals depend on habitats provided by wood. Large wood has been observed to support step-pool morphology, generate local scour and deposition, and even to create dams and trigger avulsions on streams of all sizes. Natural wood accumulations reduce flow-through velocity at baseflow (Shields and Smith 1992), facilitating retention of organic materials for processing by lower levels of the food web. Woody material is an important substrate for benthic macroinvertebrates (Wallace and Benke 1984) and provides diverse pool habitat, cover, and velocity refugia for fish and other animals. Visual cover from predators is important for fish in many stream ecosystems. Terrestrial and amphibious animals use instream wood for basking and perching. Riparian plants often rapidly establish on deposition associated with woody material. Habitat rehabilitation projects often feature addition of woody materials to streams, primarily for habitat reasons and only secondarily for erosion control or channel stabilization (Fischenich and Morrow 2000). Local effects of wood structures (whether they induce scour or deposition) depend on structure design and site variables.

Design

Design of woody material structures should follow a geomorphic and ecological assessment of the watershed and a similar, more detailed assessment of the reach or reaches to be treated including an analysis of existing conditions and anticipated responses related to stability, as well as habitat diversity. Site assessments are described in more detail in NEH654.03.

Types of LWM structures

Existing designs for large wood structures may be grouped into a few basic configurations, as shown in table TS14J–3. Only general concepts are presented, as numerous variations are found. Combinations of woody materials with stone and living plant materials are common. The first three types shown in table TS14J–3 are intermittent structures, while the last three provide continuous protection along an eroding bank. Rootwads may be placed at spaced intervals or in an interlocking fashion so they may be considered either intermittent or continuous types. The design and construction of rootwads and tree revetments are also addressed in NEH654 TSTS14I. Intermittent structures provide greater aquatic habitat diversity than continuous protection. Existing design criteria for engineered log jams (ELJ) were developed based on experience in wide, shallow, coarse-bed streams in the Pacific Northwest. Application of these concepts to streams with relatively deep channels, sand beds, and flashy hydrology requires considerable modification (Shields, Morin, and Cooper 2004). Figure TS14J–2 depicts LWM (also known as large woody debris) where it is an impediment to flow or navigation, as illustrated in figure TS14J–2. Woody materials have been shown to be an integral part of stream ecosystems. However, LWM such as this can also be used for restoration purposes.

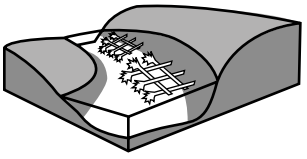
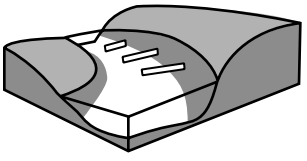
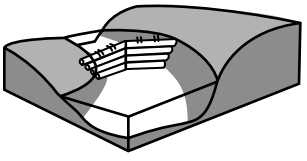
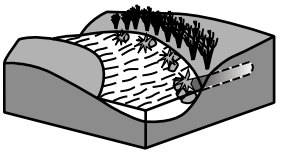
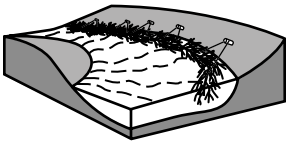
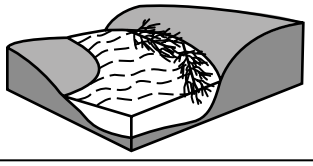
Selecting a type of structure

Configuration of a LWM structure should be selected using similar criteria that are employed for selecting any approach for stream stabilization or habitat rehabilitation:

- The configuration should address the dominant erosion processes operating on the site (Shields and Aziz 1992).
- Key habitat deficiencies (lack of pools, cover, woody substrate) should be addressed.

Figure TS14J–2 White River, IN, with large woody debris *(Photo courtesy of USGS)*

Table TS14J–3 Classification of large wood instream structures

Configuration	Sketch	Description	Strengths	References
Engineered logjams		Intermittent structures built by stacking whole trees and logs in crisscross arrangements	Emulates natural formations. Creates diverse physical conditions, traps additional debris	Abbe, Montgomery, and Petroff (1997); Shields, Morin, and Cooper (2004)
Log vanes		Single logs secured to bed protruding from bank and angled upstream. Also called log bendway weir	Low-cost, minimally intrusive	Derrick (1997); D'Aoust and Millar (2000)
Log weirs		Weirs spanning small streams comprised of one or more large logs	Creates pool habitat	Hilderbrand et al. 1998; Flosi et al. (1998)
Rootwads		Logs buried in bank with rootwads protruding into channel	Protects low banks, provides scour pools with woody cover	
Tree revetments or roughness logs		Whole trees placed along bank parallel to current. Trees are overlapped (shingled) and securely anchored	Deflects high flows and shear from outer banks; may induce sediment deposition and halt erosion	Cramer et al. (2002)
Toe logs		One or two rows of logs running parallel to current and secured to bank toe. Gravel fill may be placed immediately behind logs	Temporary toe protection	Cramer et al. (2002)

- The finished project should function in harmony with the anticipated future geomorphic response of the reach.
- Economic, political, institutional, and construction access issues should be considered.
- Suitable materials must be available for reasonable cost.
- Safety issues for recreational use of the completed project reach should be addressed, if appropriate.
- Structures like weirs or spurs that protrude into the flow tend to create greater habitat diversity than those that parallel banks, like revetments, with attendant effects on fish (Shields, Cooper, and Testa 1995).

Dimensions for intermittent LWM structures

The geometry of intermittent (spur-type) LWM structures may be specified by crest angle, length, elevation, and spacing. Spur-type structures are addressed in more detail in NEH654 TS14H.

The crest angle (angle between a line normal to the approach flow vector and the weir crest) may be set at 15 degrees upstream from a line drawn perpendicular to flow to promote deflection of overtopping flow away from eroding banks. Based on results of straight channel flume tests, Johnson, Hey, et al. (2001) suggested that stone spur-type structures be angled upstream so that the angle between the bank and the crest is between 25 degrees and 30 degrees. However, the angles can approach 90 degrees on straighter channels. Wood members embedded in the bank with their butts or rootwads pointing upstream may gain stability as drag forces tend to push them into the bank.

Crest length for structures that do not span the channel may be based on a projected value for the equilibrium width of the channel. Alternatively, crest length may be based on a target flow conveyance for the design cross section. A step-by-step procedure for spacing these structures is provided in NEH654 TS14H.

In incised channels, crest elevations for ELJ-type structures must be high enough so that the sediment berms that form over the structures stabilize existing near-vertical banks. Stable bank heights and angles may be based on geotechnical analyses or empirical criteria based on regional data sets. Castro and Sampson (2001) suggest crest elevation be set equal to that of the channel-forming flow stage. Conversely, Derrick (1997) suggests that even very low structures can exert important influence on flow patterns. All other factors being equal, local scour depths tend to be greater for higher structures.

Spacing between intermittent wood structures should be great enough to provide segments of unprotected bankline between structures to reduce cost and to create physical habitat diversity (Shields, Cooper, and Knight 1995), but also prevent flanking and structural failure. Spacing for intermittent structures is normally expressed as a multiple of the length of the structure from bank to riverward tip, measured perpendicular to the approach flow (projected crest length or effective length). Sylte and Fischenich (2000) suggest that spacing be three to four times the projected crest length for bends with $R_c/W > 3$ (radius of curvature/bankfull width), decreasing to 0 for $R_c/W < 2.5$. Tortuous channels can be problematic. Shields, Morin, and Cooper (2004) suggested that ELJ-type structures should be spaced one and a half to two times the crest length apart, following criteria for traditional training structures presented by Petersen (1986).

The embedment length or dimension for bank key-in for structures that are partially buried in the bank varies with bank height, soil type, and stream size. The key-in should be sufficient to maintain the position of the rest of the structure throughout its design life and should be greater for frequently overtopped and highly erodible banks (Sylte and Fischenich 2000).

Force and moment analysis

Some workers have developed engineering design procedures for wood structures that considered all of the important forces acting during design events, thus allowing design of anchoring systems that produced given factors of safety (Abbe, Montgomery, and Petroff 1997; D'Aoust and Millar 2000; Shields, Morin, and Cooper 2004). Forces that may be considered in such an analysis include buoyancy, friction between the woody structure and the bed, fluid drag and lift, and geotechnical forces on buried members. Simplified approaches with inherent assumptions are available, including one in NEH654 TS14E.

Buoyant force—The buoyant force is equal to the weight of the displaced water volume. The net buoyant force, $\vec{F}_b$, is equal to the difference between the weight of the structure and the weight of displaced water:

$$\vec{F}_b = \left[\rho_{wood} V_{wood} - \rho_{water} V_{water}\right]\vec{g} \qquad \text{(eq. TS14J–1)}$$

where:
ρ = density
V = volume
$\vec{g}$ = the gravitational acceleration vector in the vertical direction

For a fully submerged structure,

$$V_{wood} = V_{water} = V \text{ and } \vec{F}_b = \left(\rho_{wood} - \rho_{water}\right)V\vec{g} \qquad \text{(eq. TS14J–2)}$$

Wood structures may have complex geometries, which makes determination of volume difficult, particularly for partially submerged structures. Computations may be simplified by assuming that logs are cylinders or cones, adopting advantageous coordinate systems, and treating rootwads and boles as separate elements (Braudrick and Grant 2000; Shields, Morin, and Cooper 2004). Alternatively, a volume computed from the outside dimensions of the structure may be multiplied by a porosity factor to allow for air spaces. Thevenet, Citterio, and Piegay (1998) suggested that this factor is 10 percent for wood jams and 7 percent for shrubs.

If the wood structure may be approximated by a triangular prism of height, h, and with a uniform specific weight $\gamma_{structure}$, a simple solution for the depth, d_{wn}, at which the structure becomes neutrally buoyant (buoyant forces =gravitational forces) may be computed using:

$$\frac{\gamma_{structure}}{\gamma_w} = \frac{d_{wn}}{h}\left(2 - \frac{d_{wn}}{h}\right) \qquad \text{(eq. TS14J–3)}$$

where:
γ_w = specific weight of water

Friction—The movement of large wood structures by sliding along the bed will be resisted by a frictional force, $\vec{F}_f$, with magnitude equal to the normal force, $\vec{F}_n$, times the coefficient of friction between the woody material and the bed.

$$\vec{F}_f = \mu_{bed}\vec{F}_n \qquad \text{(eq. TS14J–4)}$$

In the absence of measured data, Castro and Sampson (2001) assumed that $\mu_{bed} = \tan\theta$, where θ is the friction angle for the bed sediments. However, it should be noted that the normal force, $\vec{F}_n$, approaches zero as depth increases and the structure approaches neutral buoyancy. Therefore, $\vec{F}_f$ may be effectively zero for design conditions.

Drag—The drag force on an LWM structure may be computed using the equation

$$\vec{F}_d = \frac{C_D A \gamma_w \left[\vec{U}_o \times \vec{U}_o\right]}{2g}\vec{c} \qquad \text{(eq. TS14J–5)}$$

where:
$\vec{F}_d$ = drag force
C_D = drag coefficient
A = area of structure projected in the plane perpendicular to flow
$\vec{U}_o$ = approach flow velocity in the absence of the structure
$\vec{c}$ = unit vector in the approach flow direction

A woody material structure may be treated as a single body, rather than as a collection of individual cylinders (Gippel et al. 1996). For structures located on the outside of bends, the cross-sectional mean velocity should be increased by a factor of 1.5 to allow for higher velocities on the outside of bends (USACE 1991b). Drag coefficients may be computed using an empirical formula (Shields and Gippel 1995), and typically range from ~0.7 to 0.9 (table TS14J–2). Drag coefficients for cylinders placed perpendicular to the flow reach values as high as 1.5 for cylinders that are barely submerged due to forces associated with the formation of standing waves (Alonso 2004). Drag coefficients for geometrically complex objects like LWM structures vary less with angle of orientation to the flow than for simple cylinders and tend to fall in the range of 0.6 to 0.7 (Gippel et al. 1996). Alonso (2004) fit the following regression formulas to laboratory data and suggested that it might be used to compute the drag coefficient, C_D:

$$C_D = W \times \left[1 - 0.35\exp\left(\frac{-4G}{d}\right)\right] \times \left[1.062 + 2\times10^{-6}R_e - 3\times10^{-12}R_e^2 + 2\times10^{-18}R_e^3\right] \qquad \text{(eq. TS14J–6)}$$

where:
G = distance from the bottom of the log to the bed
R_e = cylinder Reynolds number, $\frac{Ud}{\nu}$

where:
U = magnitude of the approach flow velocity
d = diameter of the log
ν = kinematic viscosity of the water
W = factor to account for the increase in drag due to surface waves, and may be given by

$$W = -0.28 \ln\left[\frac{z}{d}\right] + 1.4 \qquad \text{(eq. TS14J–7)}$$

when z/d < 4, and W = 1 when z/d > 4,

where:
z = distance from the log centerline to the water surface

Drag forces are expected to rapidly diminish with time during the first few high-flow events as patterns of scour and deposition reshape the local topography (Wallerstein et al. 2001).

Lift—The lift force, $\vec{F}_L$, on an LWM structure may be computed using the equation

$$\vec{F}_L = \frac{C_L A \gamma_w \left[\vec{U}_o \times \vec{U}_o\right]}{2g} \vec{e} \qquad \text{(eq. TS14J–8)}$$

where:
C_L = lift coefficient
$\vec{e}$ = unit vector normal to the plane containing primary flow direction, $\vec{c}$, and the transverse axis of the structure

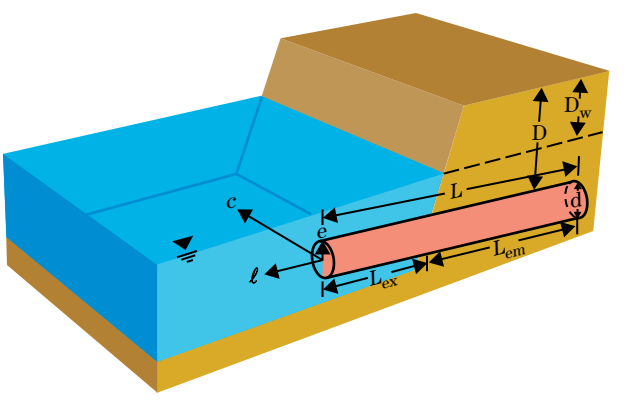
The lift coefficient on a single cylinder placed perpendicular to the flow is greatest (~0.45) when the cylinder is in contact with the bed and declines to near zero when the gap between the bottom of the cylinder and the bed exceeds one half times the cylinder diameter (Alonso 2004). As with drag, lift forces likely rapidly diminish as patterns of scour and deposition reshape the local topography (Wallerstein et al. 2001). Except for rare situations, lift may be neglected in design of LWM structures.

Geotechnical forces—The resistive forces due to passive soil pressure acting on buried portions of logs are direct reactions to fluid forces. A simplified analysis is presented here. A more detailed treatment that includes sloping banks and a nonhorizontal water table is presented by Wood and Jarrett (2004) and provides the basis for an associated Excel® worksheet. The following equations (Gray 2003) assume that the:

- log is embedded horizontally in the streambank
- top of the bank is horizontal
- bank is composed of homogeneous, isotropic soil with specific weight γ_{soil}, friction angle ϕ and cohesion c
- ground water table elevation in the bank is approximately equal to the stream surface elevation, which is high enough to fully submerge the log (fig. TS14J–3)
- bank slope is assumed to be near vertical
- the log is assumed to be frictionless

The log has a length = L, diameter d, and is buried a distance D below the top bank and a horizontal depth L_{em} (embedment length). The passive soil resistance distribution is assumed to be triangular with its maximum value at the bank face and decreasing linearly to zero at the embedded tip of the log. This implies that the resultant passive resistance force acts on the log a distance of $2/3L_{em}$ from the embedded tip. The active earth pressure force is assumed to be small, relative to the passive force.

Figure TS14J–3 Definition sketch for geotechnical forces on buried log



The vertical loading on the log due to the weight of the soil above it will be given by:

$$F_{soil} = \sigma'_v L_{em} d \qquad \text{(eq. TS14J–9)}$$

where:

$$\sigma'_v = (D - D_w)(\gamma_{soil} - \gamma_{water}) + D_w \gamma_{soil} \qquad \text{(eq. TS14J–10)}$$

where:

γ_{soil} = moist or total unit weight of the soil above the log

Alternatively, F_{soil} may be computed using equations developed to compute soil loading on conduits buried in ditches. When the ditch width is no greater than three times the log diameter,

$$F_{soil} = C_d \sigma'_v B_d^2 \frac{L}{D} \qquad \text{(eq. TS14J–11)}$$

where:

B_d = width of the ditch

C_d = a coefficient that captures the interaction between the ditch walls and the fill

$$C_d = \frac{\left[1 - e^{\frac{-0.38D}{B_d}}\right]}{0.38} \qquad \text{(eq. TS14J–12)}$$

$$\text{for } \frac{D}{B_d} < 2 \text{ and} \qquad \text{(eq. TS14J–13)}$$

$$C_d = \frac{D}{B_d} \qquad \text{(eq. TS14J–14)}$$

$$\text{for } \frac{D}{B_d} \geq 2 \qquad \text{(eq. TS14J–15)}$$

The two approaches for computing F_{soil} converge for ditches with widths just slightly greater than the log diameter.

Assuming friction between the soil and log is negligible, the passive soil pressure force, $\vec{F}_p$, is given by

$$\vec{F}_p = 0.5 \sigma_p L_{em} d \qquad \text{(eq. TS14J–16)}$$

where:

σ_p = passive soil pressure

is given by

$$\sigma_p = \sigma'_v K_p + 2c\left(K_p\right)^{0.5} \qquad \text{(eq. TS14J–17)}$$

where:

K_p = coefficient of passive earth pressure

is given by

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) \qquad \text{(eq. TS14J–18)}$$

If unknown, soil cohesion, c, may conservatively be assumed to equal 0. Riparian soils are often noncohesive, and cohesion in cohesive soils is effectively 0 when soils are saturated.

Moments—The driving moment, $\vec{M}_d$, about the buried tip of the embedded log is given by the vector sum

$$\vec{M}_d = \left[\left(\vec{F}_d + \vec{F}_L\right)\left(L_{em} + \frac{L_{ex}}{2}\right) + \vec{F}_b\left(\frac{L}{2}\right)\right] \times \vec{l} \qquad \text{(eq. TS14J–19)}$$

where $\vec{l}$ is the unit vector along the axis of the buried log and positive in the direction away from the buried tip and $L_{ex} = L - L_{em}$. The resisting moment, $\vec{M}_d$, will act opposite the driving moment and is given by the vector sum

$$\vec{M}_r = \left[\vec{F}_{soil}\left(\frac{1}{2}\right)L_{em} + \vec{F}_p\left(\frac{2}{3}\right)L_{em} + \vec{F}_c L_c\right] \times \vec{l} \qquad \text{(eq. TS14J–20)}$$

where $\vec{F}_c$ is the restraining force due to anchor cables or ballast, and L_c is the appropriate moment arm about the buried tip of the embedded log.

Ballast and anchoring

Forces and moments due to anchors may be added to the other forces acting on the LW structure to compute factors of safety. The factor of safety with respect to forces, F_{sf}, is the ratio of the magnitude of the resultant of the resisting forces to the magnitude of the resultant of the driving forces with separate factors of safety computed for the vertical (y) and horizontal (x, streamwise) directions.

$$F_{sf_y} = \frac{F_{soil_y} + F_{py} + F_{cy}}{F_b + F_L} \qquad \text{(eq. TS14J–21)}$$

$$F_{sf_y} = \frac{F_{soil_x} + F_{px} + F_{cx}}{F_D} \qquad \text{(eq. TS14J–22)}$$

M_r acts opposite M_d, and both vectors act along a horizontal axis through the embedded tip of the log. Therefore, the factor of safety with respect to moments, F_{sm}, is simply the ratio of their magnitudes:

$$F_{sm} = \frac{M_r}{M_d} \qquad \text{(eq. TS14J–23)}$$

Anchoring systems should be designed to achieve factors of safety greater than 2 due to the high level of uncertainty in computations for imposed forces. Anchoring approaches include placing ballast (soil, cobbles, boulders) on or within the structure, embedding part or all of the large wood in the bank or in a stone structure, and using cable, marine rope, or chain to secure the structure to boulders, soil anchors (NEH654 TS14E), stumps, trees, deadmen, or pilings (Cramer et al. 2002; Fischenich and Morrow 2000). When logs or woody elements are used as ballast, it is important for the designer to consider the implications of the wood rotting and becoming lighter. When boulders or bed material are used for ballast, buoyant, drag, and lift forces on the ballast rock must be considered in the force balance (D'Aoust and Millar 2000). An electronic spreadsheet may facilitate this calculation.

Logs in complex structures may be attached to one another or to boulders by drilling holes through the logs and pinning them together with steel rebar. Epoxy adhesive has also been used for attaching logs. Abbe, Montgomery, and Petroff (1997) favor an approach that may be termed passive anchoring (Cramer et al. 2002), in which the shape, weight, ballast, and placement of a structure are adequate to resist movement in events up to the design flow. Passively anchored structures may be comprised of wood members that are attached to one another, but not to external anchors. Passive anchoring is not recommended for high hazard situations, sites with vulnerable infrastructure downstream, or sites where structures will be frequently overtopped.

Materials

Minimum dimensions, species, and sources for woody materials should be specified during design. Cramer et al. (2002) suggest the following guidelines for size of trees and rootwads:

Dimension	Minimum size
Rootwad diameter	Bankfull discharge depth
Trunk diameter	0.5 × bankfull discharge depth
Tree length	0.25 × bankfull discharge width

Clearly, wood materials this large are not always available. Onsite sources are always most economical; importing large materials can be extremely costly. However, benefits to the stream ecosystem must be weighed against the impacts of clearing and grubbing on existing terrestrial habitat. Complex woody material structures that feature numerous branches and high stem density locally decrease flow velocity, inducing sediment deposition. Accordingly, materials should be selected that have numerous branches, being careful not to break or remove branches during construction. Clearing within the stream corridor should be avoided, but bar scalping may be advisable in certain cases to provide temporary relief of outer bank erosion in a sharp bend. Resulting woody materials (willow rootwads and stems) may be used in structures to trigger rapid revegetation.

Species that are decay resistant are preferred, such as eastern red cedar (*Juniperous virginiana*), western red cedar (*Thuja plicata*), coastal redwood (*Sequioa sempervirens*), Douglas-fir (*Pseudotsuga* spp.), or bald cypress (*Taxodium distichum*). Rapidly decaying species, such as cottonwood (*Populus* spp.), pines native to the Southeast (*Pinus echinata* and *Pinus taeda*), and alder (*Alnus* spp.), should be avoided. However, as noted, use of freshly cut or grubbed willow or cottonwood trees may be desirable for quick revegetation in structures that are partially buried. Comments on decay rates are provided in table TS14J–4.

Decay rates are climate dependent, due to the requirements of the fungi responsible for aerobic decomposition of wood. Rates increase with increasing temperature and precipitation. Scheffer (1971) developed the following index for comparing potential decay rates of aboveground wood structures in different climatic regions of the United States.

Table TS14J–4 Comparison of desirability of various tree species for stream structures

Species	Durability (assuming wetting and drying)	Source of information[1/]
Cottonwood (*Populus* spp.)	Poor	Johnson and Stypula (1993)
Alder (*Alnus* spp.)	Poor	Johnson and Stypula (1993)
Maple (*Acer* spp.)	Fair (will survive 5 to 10 yr)	Johnson and Stypula (1993)
Hemlock (*Tsuga* spp.)	Least durable of conifers	Johnson and Stypula (1993)
Sitka spruce (*Picea sitchensis*)	Excellent	Johnson and Stypula (1993)
Douglas-fir (*Pseudotsuga* spp.)	Excellent (will survive 25 to 60 yr) 32–56 yr	Johnson and Stypula 1993); Harmon et al. (1986)
Western red cedar (*Thuja plicata*)	Most desirable (will survive 50 to 100 yr)	Johnson and Stypula (1993)
Yellow-poplar (*Liriodendron tulipifer*a)	0.4 yr	Harmon et al. (1986)
Aspen (*P. tremuloides*)	5 yr	Harmon et al. (1986)
White fir (*A. concolor*)	4 yr	Harmon et al. (1986)
Norway spruce (*Picea abies*)	~30 yr	Kruys, Jonsson, and Stahl (2002)
Conifers (*P. sitchensis*, *T. heterophylla*, *P. menziesii*, *T. plicata*)	Half-life of ~20 yr	Hyatt and Naiman (2001)
Black locust, red mulberry, Osage orange, Pacific yew	Exceptionally high heartwood decay resistance	Simpson and TenWolde (1999)
Old growth baldcypress, catalpa, cedars, black cherry, chestnut, Arizona cypress, junipers, honeylocust, mesquite, old growth redwood, sassafras, black walnut	Resistant or very resistant to heartwood decay	Simpson and TenWolde (1999)
Young growth baldcypress, Douglas-fir, western larch, longleaf old growth pine, old growth slash pine, young growth redwood, tamarack, old growth eastern white pine	Moderately resistant to heartwood decay	Simpson and TenWolde (1999)
Red alder, ashes, aspens, beech, birches, buckeye, butternut, cottonwood, elms, basswood, true firs, hackberry, hemlocks, hickories, magnolia, maples, pines, spruces, sweetgum, sycamore, tanoak, willows, yellow-poplar	Slightly or nonresistant to heartwood decay	Simpson and TenWolde (1999)

1/ Information from Johnson and Stypula (1993) is qualitative and unsubstantiated. Evidently, these comments pertain to the region of King County, Washington. Harmon et al. (1986) provide a review of scientific literature dealing with decomposition rates of snags and logs in forest ecosystems. The times from Harmon et al. (1986) represent the time required for 20 percent decomposition (mineralization) of a log based on exponential decay constants obtained from the literature. Fragmentation of logs in streams due to mechanical abrasion would accelerate the decay process, as would more frequent wetting and drying. Kruys, Jonsson, and Stahl (2002) provide data on decay of fallen and standing dead trees in a forest in mid-northern Sweden. Hyatt and Naiman (2001) provide data on residence time of large wood in Queets River, Washington. Simpson and TenWolde (1999) provide data for evaluating wood products, not whole trees.

$$\text{Climate index} = \frac{\sum_{\text{Jan}}^{\text{Dec}} \left[(T - 35)(D - 3) \right]}{30} \qquad \text{(eq. TS14J–23)}$$

where:

T = mean monthly temperature (°F)

D = mean number of days in the month with 0.01 inch or more of precipitation

The summation represents the sum of products for all of the months of the year. The sum is divided by 30 to make the index fall between 0 and 100 for most of the United States. For example, Scheffer computed values of 82.5, 44.8, and 22.0 for Atlanta, Georgia; Des Moines, Iowa; and Casper, Wyoming, respectively. This implies that a wood structure would last about four times longer in a climate typical of Wyoming than one typical of Georgia, all other factors being equal.

Synthetic LWM for stream work is available commercially (Bolton et al. 1998). These products are engineered to compare favorably with natural materials in terms of durability or habitat value. However, they may be less effective in terms of habitat creation or more costly than natural materials. Cost comparisons should consider full project life cycles.

Cost

Costs for LWM structures are heavily influenced by site variables and material sources. Cramer et al. (2002) provide typical cost ranges for large wood of $500 to $750 per tree with rootwad and $200 to $300 per tree without rootwad. These figures include material, hauling to the site, excavation, spoilage, and installation. Additional cost information is summarized in table TS14J–5.

Maintenance

LWM structures should be viewed as temporary measures to trigger desirable natural changes in channels and banks. Accordingly, structures gradually degrade and break down. However, structures should be maintained until planted or invading woody plants have succeeded in establishing in the treated area. A relatively high level of maintenance is necessary if initial configurations are to be maintained for more than a few years. Annual low-water inspections are advisable, with particular attention to anchoring systems, decay status of woody materials, hazards to downstream infrastructure, and erosion patterns. Habitat monitoring may be qualitative, but field measurement of water depth, width, and velocity (Shields, Knight, Morin, and Blank 2003) is preferable. Photo documentation and cross-sectional and thalweg surveys are most helpful in detecting changes. Cramer et al. (2002) recommend additional inspections following any event that equals or exceeds the 1-year flow during the first 3 years following construction.

Table TS14J–5 Reported costs for stream stabilization and habitat enhancement structures

Year	Location	Protected bank length, m	Unit cost[1/], $/m	Comments	Source
1987	Nestucca River and Elk Creek, OR	1,960	24	119 woody debris structures using 99 mature conifers placed for habitat objectives, not stabilization	House and Crispin (1990)
1990–91	North Fork Porter Creek, WA	500	165	Five different log configurations anchored with cables and boulders for habitat purposes only	Cederholm et al. (1997)
1990–91	North Fork Porter Creek, WA	500	13	60 trees > 30 cm diameter cut felled into stream from banks and tethered to stumps with cable for habitat purposes only	Cederholm et al. (1997)
1994	Buffalo River, AR		66	Cedar tree revetments and willow rootwads planted in ditches. Two of 13 sites have not performed well	Personal communication, David Mott, National Park Service
1996	Cowlitz River, WA	430	47	Engineered logjams. Includes estimate for value of donated materials	Abbe, Montgomery, and Petroff (1997)
1996	Bayou Pierre, MS	240	117	Eight tree-trunk bendway weirs spaced 30 m apart. Weirs consisted of two to four trees per weir cabled to 0.15-m steel pipes driven into bed. Riprap-protected keys. Two structures failed, others have performed well	Personal communication, Larry Marcy, U.S. Fish and Wildlife Service
1988–97	Six urban gravel bed streams, Puget Sound, WA	2,960	493	Anchored and unanchored LWM added for flood control, sediment/erosion control and habitat enhancement	Larson, Booth, and Morley (2001)
1998	Various, MO		72 [2/]	Double row tree revetment installed using heavy equipment	Personal communication, Brian Todd, State of Missouri
1999	Bitterroot River, MT		80	Rootwads	Brown and Gray (1999)
2000	Little Topashaw Creek, MS	1,500	80	72 LWM structures in small, sand-bed stream. Unit cost = $95/m when willow planting is included	Shields, Morin, and Cooper (2004)
2000	Various		40–200	Rootwads	Sylte and Fischenich (2000)
2002	Various		40–80	Roughness trees	Cramer et al. (2002)
2002	Various, WA		70–200	Log toe	Cramer et al. (2002)
1995–2002	Various, PA		79–213 [3/]	Rootwads	Wood (2003)

1/ Costs are for the construction contract and do not include design and contract administration. Construction materials, mobilization, and profit are included.
2/ Upper end of range provided by original source
3/ An emergency project that included importing fill to replace a 10 m high bank cost $591/m

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Streambank Armor Protection with Stone Structures

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Cover photo: Interlocking stone structures may be needed to provide a stable streambank.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

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Streambank Armor Protection with Stone Structures

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Streambank Armor Protection with Stone Structures

Purpose

Structural measures for streambank protection, particularly rock riprap, have been used extensively in support of stream restoration designs. Stone continues to be an important component of many stream restoration and stabilization projects, where stone or rock provides the needed weight or erosion protection, as well as providing a needed foundation for other design elements. This technical supplement is intended to provide field staffs with an understanding of some of the basic principles, design considerations, and techniques used to treat streambank erosion with rock. Design considerations that are applicable to any structure involving the use of stone are addressed. The use of stone as part of soil bioengineering and to complement instream habitat is also addressed.

Introduction

Stone has long been used to provide immediate and permanent stream and river protection. It continues to be a major component in many of the newer and more ecologically friendly projects, as well. The use of stone in a stream restoration design is a function of the engineering and ecological requirements of the final design. While the term stone can also be used to refer to a unique size of material (between cobbles and boulders), it is used interchangeably in this technical supplement with the term rock. Herein, these terms refer to large, engineered, geologic material used as an integral part of the restoration design.

This technical supplement describes some of the typical applications of both integrated streambank stabilization systems and stand-alone riprap treatments. It is recognized that stone and rock are also used to create desired habitat elements, but this technical supplement focuses primarily on the design of stone treatments for streambank stabilization and protection. Basic principles, stone requirements, design considerations, and techniques used to treat streambank erosion with rock are all described. While much of the guidance described herein was developed for application of stone riprap revetments, it is also applicable for other designs involving rock.

Benefits of using stone

Structural measures are designed to withstand high streamflows and provide adequate protection as soon as installation is complete. Rock may be readily available to most sites, but where it is not, alternative structural measures are designed based on the local cost of available materials (concrete, steel, manufactured materials, wood). Established techniques exist for rock design and construction. Rock riprap measures have a great attraction as a material of choice for emergency programs where quick response and immediate effectiveness are critical.

Rock riprap is needed for many streambank stabilization designs, especially where requirements for slope stability are restrictive, such as in urban areas. It is one of the most effective protection measures at the toe of an eroding or unstable slope. The toe area generally is the most critical concern in any bank protection measure. The primary advantages of stone over vegetative approaches are the immediate effectiveness of the measure with little to no establishment period. The use of stone may offer protection against stream velocities that exceed performance criteria for vegetative measures.

Stone considerations

Not all rocks are created equal. A variety of important stone design characteristics and requirements exist that must be accounted for to successfully use rock in the stream.

Stone size

The stone used in a project, whether it is part of a combined structure or used as a traditional riprap revetment, must be large enough to resist the forces of the streamflow during the design storm. A stone-sizing technique appropriate for the intended use must also be selected. Many established and tested techniques are available for sizing stone. Most techniques use an estimate of the stream's energy that the rock will need to resist, so some hydraulic analysis is generally required. Guidance for stone sizing techniques is provided in NEH654 TS14C.

Stone shape

Some methods use different dimensions to characterize stone size. The critical dimension is the minimum sieve size through which the stone will pass. Some techniques assume that riprap is the shape of a sphere, cube, or even a football shape (prolate spheroid). To avoid the use of thin, platy rock, neither the breadth nor the thickness of individual stones is less than a third of its length. The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) riprap specifications allow riprap to be a spheroid three times as long as it is thick (L/B = 3). Note that the shape of most riprap can be represented as the average between a sphere and a cube. An equation for an equivalent diameter of riprap shaped between a cube and a sphere is:

$$D = \left[\frac{2 \times W}{\gamma_s \times \left(1 + \frac{\pi}{6}\right)} \right] \quad \text{(eq. TS14K–1)}$$

where:
W = weight of the stone, lb
γ_s = density of the stone, lb/ft^3
D = equivalent diameter, ft

This relationship may be helpful if a conversion between size and weight is necessary for angular riprap with this shape.

Riprap should be angular to subangular in shape. Field experience has shown that both angular (crushed limestone) and rounded rock (river stones) can be used for riprap protection with equal success, but shape differences do require design adjustments. Rounded rock does not interlock as well as angular rock. Generally, rounded rock must be 25 to 40 percent larger or more in diameter than angular rock to be stable at the same discharge.

Stone gradation

Stone gradation influences resistance to erosion. The gradation is often, but not always, considered by the technique used to determine the stone size. In general, specifications typically include two limiting gradation curves. The design becomes more conservative as the coarser upper gradation limit is used. A question that should be answered as part of the design is whether a standard gradation, which could be considerably bigger than a special gradation, would be cheaper to build. U.S. Army Corps of Engineers (USACE) EM 1110–2–1601 (USACE 1991b) contains standardized gradations for riprap placement in the dry, low-turbulence zones. One set of standard gradations are those used by the USACE. This method assumes the specific gravity of a stone, G_s = 2.65 and a stone shaped as a sphere. Another approach is to specify American Society for Testing and Materials International (ASTM) D6092 for standard gradation requirements.

For most applications, the stone should be reasonably well graded (sizes are well distributed) from the minimum size to the maximum size. Onsite rock material may be used for rock riprap when it has the desired size, gradation, and quality. A well-graded distribution will have a wider range of rock sizes to fill the void spaces in the rock matrix. The stone gradation influences the design and even the need for a filter layer or geotextile. Further information on the design, use, and application of geotextiles is provided later in this technical supplement, as well as in NEH654 TS14D.

There are exceptions to this well-graded requirement. For instance, a steep slope rock chute will have a higher stable discharge if the rock is poorly graded (all rock is the same size). However, once this poorly graded material starts to fail, it will fail more rapidly than a well-graded material.

Stone quality

Rock quality or durability is important for the long-term success of any streambank protection project that uses riprap. In most applications, the rock must last for the life of the project. The stone should be sound and dense, free from cracks, seams, and other defects that would tend to increase deterioration. Poor quality rock can break down or deteriorate into smaller pieces, thereby reducing the effective diameter. This breakdown can be due to physical, chemical, and mechanical factors. Physical factors include freeze-thaw cycles or, in some cases, capillary action. An example is shown in figure TS14K–1. A chemical reaction with the runoff water can also cause the stone to break down. Rough handling during delivery and placement can mechanically fracture rock into smaller pieces. Interbedded layers of weaker material can also cause accelerated rock break down.

Stone density

The unit weight of stone (γ_s) typically ranges from 150 to 175 pounds per cubic foot, and different quarries will usually provide material with different unit

weights. Designs should be based on realistic unit weights for the project area. If $G_s = \gamma_s / \gamma_w = 2.65$, then $\gamma_s = 2.65 \times 62.4$ pounds per cubic foot (density of water) = 165.36 pounds per cubic foot (a normal design assumption for rock density). NRCS specifications for riprap allow a minimum $G_s = 2.50$. Note that specific gravity is also shown as ρ in some specifications.

A rule of thumb is that for a 5-percent decrease in the unit weight of riprap ($G_s = 2.65 \rightarrow 2.50$), the design diameter would need to be about 10 percent larger than that originally designed, to resist the same forces.

Stone inspection

Rock used for riprap should come from approved sources. Sufficient testing should be performed to ensure that durability requirements are met for the expected service conditions and for the life of the project. In lieu of adequate test records on rock quality, a record of successful performance of the identical material for at least 5 years, and with similar site conditions, may be used as documentation of appropriate quality for some applications. Specific rock quality requirements are provided in NRCS Material Specification #23.

Mechanisms should be in place to ensure that a characteristic size or weight used in the design is actually delivered and placed at the project. When the project is constructed, the stone must be checked to ensure that the delivered stone size and material properties meet design requirements. Visual examinations can be misleading, so physical sampling should be conducted if the project involves a significant investment or is of high risk. A rock sample should be large enough to ensure a representative gradation and to provide test results to the desired level of accuracy (ASTM D5519).

Figure TS14K–1 Capillary breakdown of stone

Design considerations

Stabilizing channel banks is a complex problem and does not always lend itself to precise design. The success of a given installation depends on the judgment, experience, and skill of the planners, designers, technicians, and installers. Several important issues that must be considered for the successful design of projects that depend on the rock performance are briefly described.

Filter layer

Where stone is placed against a bank that is composed of fine-grained or loose alluvium, a filter layer or bedding is often used. This filter layer prevents the smaller grained particles from being lost through the interstitial spaces of the riprap material, while allowing seepage from the banks to pass. This filter layer needs to be appropriately designed to protect the in-place bank material and remain beneath the designed stone or riprap. Therefore, the gradation is based in part of the gradation of the riprap layer and the bank material. The filter layer typically consists of a geosynthetic layer or an 8-inch-thick layer of sand, gravel, or quarry spalls. For design of appropriate filters under rock riprap, refer to NEH633.26.

Banks with fine-grained silts or sands may require a geotextile to provide separation and filtration under riprap. Geosynthetics are covered in more detail in NEH654 TS14D, as well as in Design Note #1 and Material Specification 595 for the design and material considerations for geotextiles. A useful reference for geotextile design considerations is the American Association of State Highway and Transportation Officials (AASHTO) M28.

Some soil bioengineering techniques do not function well under geotextiles, and placing holes through the geotextile for plantings may provide a seepage path that would weaken the structure. This may require a

trade-off analysis to balance the advantages of incorporating soil bioengineering against the advantages of an intact geotextile filter. Finally, there will also be cases where the banks may have sufficient gravel or cobbles, so that neither bedding nor geotextiles are needed.

Bank slope

Many stone sizing techniques also require information about the bank slope. In addition, a geotechnical embankment analysis may impose a limit on the bank slope. The recommended maximum slope for most riprap placement is 2H:1V. Short sections of slopes at 1.5H:1V are sometimes unavoidable, but are not desirable. Most rock cannot be stacked on a bank steeper than 1.5H:1V and remain there permanently. For riprap placement of 1.5H:1V and steeper, grouting of the rock to keep it in place must be strongly considered. Alternative measures, such as gabion baskets, are well suited to steep banks. Also, flatter slopes increase the opportunity for vegetation establishment.

Height

Stone should extend up the bank to a point where the existing vegetation or other proposed treatment can resist the forces of the water during the design event. In a soil bioengineering project, a stone revetment typically does not exceed the elevation of the level of the channel-forming flow event. However, there are exceptions where it is advisable to extend the riprap to the top of the bank.

Thickness

Different stone-sizing techniques may have different assumptions concerning the blanket thickness. The thickness of the placed rock should equal or exceed the diameter of the largest rock size in the gradation. In practice, this thickness will be one and a half to three times the median rock diameter (D_{50}). A typical minimum thickness is the greater of 0.75 times the D_{100} or one and a half times the D_{50}. The ability to use vegetative methods within a riprap revetment is diminished by additional riprap depth. While posts have been installed in revetments up to 4 feet thick, live cuttings or joint planting within a riprap thickness larger than 24 inches has had limited success.

Length

The revetment should significantly overlap the eroding area. The starting point needs to be well protected, properly keyed into the bank, and located sufficiently upstream of the major point of streamflow attack. Starting the treatment upstream helps prevent the streamflow from getting behind the structure and progressively eroding and undermining the protection. Likewise, if the bank protection does not extend sufficiently past the critical area of attack to a point where the streamflow is safely guided back into the primary channel, severe erosion can occur and start progressive failure in an upstream direction.

Where it is not possible to begin and end a structural revetment at a stable area, it is recommended that a stone revetment be extended a minimum distance of one channel width upstream and one and a half channel widths downstream of the eroded area. However, this limited treatment area has a higher risk of failure.

Tiebacks

Tiebacks or key-ins are used to reduce the likelihood of high flows concentrating behind stone slope protection. Tiebacks are used on both the upstream and downstream ends of a stone revetment. A typical rule of thumb for the depth to key into the bank is the bank height plus the anticipated scour depth. On long stone revetments, intermediate tiebacks are often used to ensure the reach integrity. Also, it is suggested that key-ins not be positioned at 90 degrees to the flow, but rather at an angle (30 to 45 degrees to the direction of flow) into the bank. Keying at an angle reduces sudden transitions of flow at the beginning and end of the revetment, and if the stream migrates, the key-in will act as a deflector.

Scour

Toe scour is the most frequent cause of failure in streambank armor protection projects. Scour can be long term, general, and local. More information on scour is provided in NEH654 TS14B.

The greatest scour depths generally occur on the outside and lower portion of curves. Scour depths may increase immediately below and adjacent to structural protection due to the higher velocity section of a stream adjacent to the relatively smooth structure surface. This may undermine the structure and result in failure.

Common methods for providing toe protection are:

- placing the stone to the maximum expected scour depth

- placing sufficient stone along the toe of the revetment to launch or fall in, and fill any expected scour
- providing a sheet-pile toe to a depth below the anticipated depth of scour or to a hard point
- paving the bed

The most commonly employed method is to extend (or key-in) the bank protection measures down to a point below the probable maximum depth of the anticipated bed scour. Where the project involves a significant investment for the protection of valuable property, potential scour can be calculated using the procedures described in NEH654 TS14B. Where there is less of an investment, approximations can be employed. A typical rule of thumb for a minimum key-in depth is one and a half times the riprap thickness or a minimum of 2 feet below the existing streambed. This practical solution generally gives good protection against undermining. Designers can review reliable data on local scour in the area, regional data, or use local experience in determining this minimum depth.

Ice and debris

River ice can have a major impact on riprap protection. Ice and debris increase the stresses on riprap by impact and flow concentration. Ice attached to stone may also dislodge stone and decrease blanket stability. Ice rafting, lifting or plucking, raft impact damage, ice raft push, and velocity increase below ice jams can all cause problems. Detailed discussions of these issues are available (Vaughan, Albert, and Carlson 2002; USACE EM 1110–2–1612, 1999).

A general rule of thumb for riprap subject to attack by large floating debris is that thickness should be increased by 6 to 12 inches, accompanied by an appropriate increase in stone size. Riprap damage from debris impacts is usually more extensive on banks with steep slopes. Therefore, streams with heavy debris loads should be not have armored slopes steeper than 1V:2.5 H (USACE EM 1110–2–1601, 1994f).

Vandalism

Many rock treatments are composed of a relatively thin layer of stone, and unauthorized removal of selected stones from the rock matrix can cause serious problems. Stone is often removed from projects for landscaping and other personal uses. Monitoring and maintenance activities should be in place to protect the project, minimize vandalism, and provide timely repair. Where vandalism is expected, it may be advisable to use larger stone than that required for stability to reduce the likelihood of removal by hand.

Placement of rock

Rock should be placed from the lowest to the highest elevation to allow gravitational forces to minimize void spaces and help lock the rock matrix together. It is important that riprap be placed at full-course thickness in one operation. Final finished grade of the slope should be achieved as the material is placed. Care should be taken not to segregate or group material sizes together during placement. Allowing the stone to be pushed or rolled downslope will cause stone size segregation. See ASTM D6825 on placement of riprap revetments.

An advantage of using riprap structures is that materials are generally readily available, and contractors with appropriate equipment and experience can be found. However, careful consideration should be given early in the design process to the stone installation method. Two commonly employed installation methods are described below.

Dumped rock riprap

This method of protection may be necessary where access to the streambed is limited or for emergency situations. Streambank work using dumped rock requires a source of low-cost rock. Access roads must be available near the stream channel, so that rock can be hauled to the streambank and either dumped over the bank or along the edge. If the job requires large quantities of rock, the operation must be set up to accommodate regular deliveries to the job site. In some cases, the banks may be too weak to support a loaded truck, thereby preventing dumping of rock directly over the streambank. In such cases, the rock may be dumped as close to the edge as possible and pushed over the edge with a bulldozer or front-end loader. Larger rock should be placed at the bottom of the revetment work to provide a stable toe section. The use of a front-end loader may be useful to select rock by size and push it over the bank.

This type of placement usually results in a poor gradation of material due to material segregation, requiring more volume to make up for the lack of gradation. While this type of bank protection requires more stone per square yard of bank protection than machine-placed riprap, it generally requires less labor and equipment operating hours.

Machine-placed riprap

This type of riprap is placed using a track-mounted backhoe or a power crane with a clam shell or orange peel bucket. The riprap is placed on a prepared slope of the streambank to a minimum design thickness of 12 to 18 inches. The larger stones are placed in a toe trench at the base of the slope. This method requires an experienced equipment operator to achieve uniform and proper placement. The toe or scour trench can be dug with the backhoe or clam shell as the machine moves along the slope. The machine can do the backfilling with rock in the same manner.

The bank sloping or grading generally is accomplished with a backhoe or sometimes a Gradall®. If a power crane is used, a dragline bucket must be used with the crane for slope grading. A perforated dragline bucket works best because it allows excess water to drain from the bucket.

Appropriate bedding and/or geotextile can be installed after the grading and slope preparation are completed. The primary function of these materials is for filtration—to prevent movement of soil base materials through the rock riprap. Bedding is normally placed by dump truck and spread to the desired thickness with a backhoe bucket, a front-end loader, or a small dozer. Geotextile must be placed by hand, secured in place as recommended by the manufacturer, consistent with site specifications. It is important that the geotextile be placed in intimate contact with the base to preclude voids beneath the geotextile. Under larger stone, a coarse bedding may be placed on the geotextile to assure that the geotextile stays in contact with the subbase. In some locations, geotextiles may also be used as a reinforcement in very soft foundation conditions. As previously noted, there will also be situations where the banks may have sufficient gravel content, so that neither bedding nor geotextiles are needed.

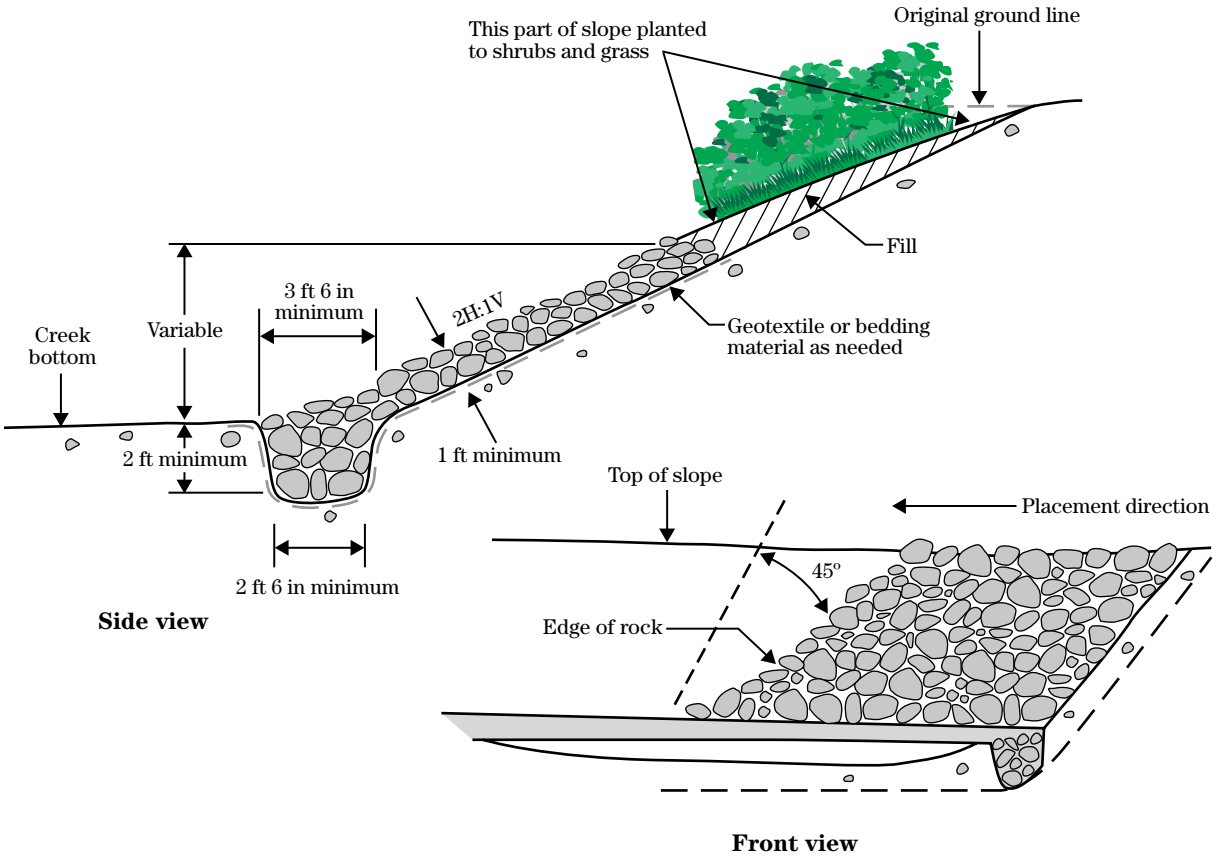
Riprap should be placed to provide a reasonably well-graded and dense mass of rock with a minimum of voids and with the final surface meeting the specified lines and grades. The larger stones should be placed in the toe trench or well distributed in the revetment. The finished stone protection should be consolidated by the backhoe bucket or other acceptable means so that the surface is free from holes, noticeable projections, and clusters or pockets of only small or only large stones.

Riprap placement should begin at the toe trench and progress up the slope maintaining the desired rock placement thickness as the work proceeds. After the toe trench has been filled to the original stream bottom level, the operator should build a wall or leading edge with the riprap, which is the full layer thickness. That thickness should be maintained throughout the placement of the riprap. The wall should be maintained at about a 45-degree angle from a transverse line down the slope, as the placement progresses from the initial starting point at the streambed and progresses up and across the slope (fig. TS14K–2).

Riprap rock should be handled and placed to the full layer thickness in one operation so that segregation is minimized and bedding or geotextile materials used under the riprap are not disturbed after the initial rock placement. Adding rock to the slope or removing it after the initial placement is not practical and generally produces unsatisfactory results. Dumping stone from the top and rolling it into place should also be avoided. This type of operation causes segregation and defeats the purpose of a rock gradation. Running on the riprap slope with track equipment, such as a bulldozer or rubber tire mounted front end loader, should also be avoided. It can damage the rock mass already in place. This operation can also tear the geotextile or damage the bedding by displacing material throughout the rock course. Tamping of the rock with the backhoe bucket can sometimes be used effectively to even up the surface appearance of riprap placement and further consolidate the rock course.

It is advisable to have a test section when riprap is being placed over geotextile to check for geotextile puncturing. After the riprap is placed, it is removed, and the geotextile is evaluated.

Figure TS14K–2 Typical riprap section



Treatment of high banks

The application of rock riprap protection on streambanks that are too high to be practically sloped can be accomplished using the following two methods:

- embankment bench
- excavated bench

Embankment bench method

The embankment bench method provides a reasonable approach to stabilize steep banks with little or no disturbance at the top of the slope and minimal disturbance to the streambed. The method also lends itself to an appropriate blend of structural, soil bioengineering, and vegetative stabilization treatments. This method, or some variation of it, is the most practical and preferred method of treating high, eroding streambanks.

The embankment bench method involves the placement of a gravel bench along the base of the eroding bank (fig. TS14K–3). The elevation of the bench should be set no lower than the height of the opposite bank and, where practicable, 1 to 2 feet higher. This gravel bench provides drainage and protection at the base of the bank and a stable fill to support the structural toe protection. It also provides a working space for the equipment to place the toe protection, which is most often rock riprap or a combination of riprap and soil bioengineering practice.

The embankment bench method requires that the convex side (low bank) of the channel be shaped by excavation of channel bed materials, normally bar removal, to compensate for the reduction in area taken by the bench projection. Offsite materials could be used for the bench in lieu of channel bed materials, but costs would be higher, and the resultant channel restriction could endanger the project. The high bank is generally left in its natural state and appropriately vegetated to assist stability. Some sloughing of the bank onto the prepared bench may occur before a good vegetative cover is established. Willows and other soil bioengineering materials can be established on the bench to help stabilize the toe of the bank and provide vegetative cover. By joint planting in the rock or by sediment accumulation and volunteer vegetation, the bench often can become a self-sustaining solution.

Excavated bench method

The excavated bench method (fig. TS14K–4) is used in situations similar to the embankment bench. The excavated bench method does not require the gravel fill material or enlarging of the channel to compensate for the encroachment of the bench area. Instead, it involves shaping the upper half or more of the high bank to allow the formation of a bench to stabilize the toe of the slope. This is accomplished in a manner which leaves the upper part of the excavated slope at least in no worse shape than it was before the excavation. This solution is rarely practical, but may be necessary in cases where stream access is restricted or not allowed. It may also be a solution on lower banks where the excavation quantity is relatively small.

Surface flow protection

The damage to high banks is often exacerbated by surface runoff. If this is not treated, any protection at the toe may be damaged. High banks subject to damage by surface water flow can be protected by using diversion ditches constructed above the top slope of the bank. Water from active seepage in the high banks should be collected by interceptor drainage and conveyed to a safe outlet. Trees or other vegetative materials in a buffer strip along the top of the bank can be used to help control the active seepage by plant uptake and transpiration. Some soil bioengineering designs can also include ancillary drainage as a function.

Treatment of bedrock controlled streams

Channels with exposed bedrock or ledgerock along the invert or streambank toe inverts require special methods to assure that the toe of the riprap can be anchored and will remain in place. The use of steel dowels and precast toe blocks are two methods that have been successfully implemented in such conditions.

Figure TS14K–3 Embankment bench method

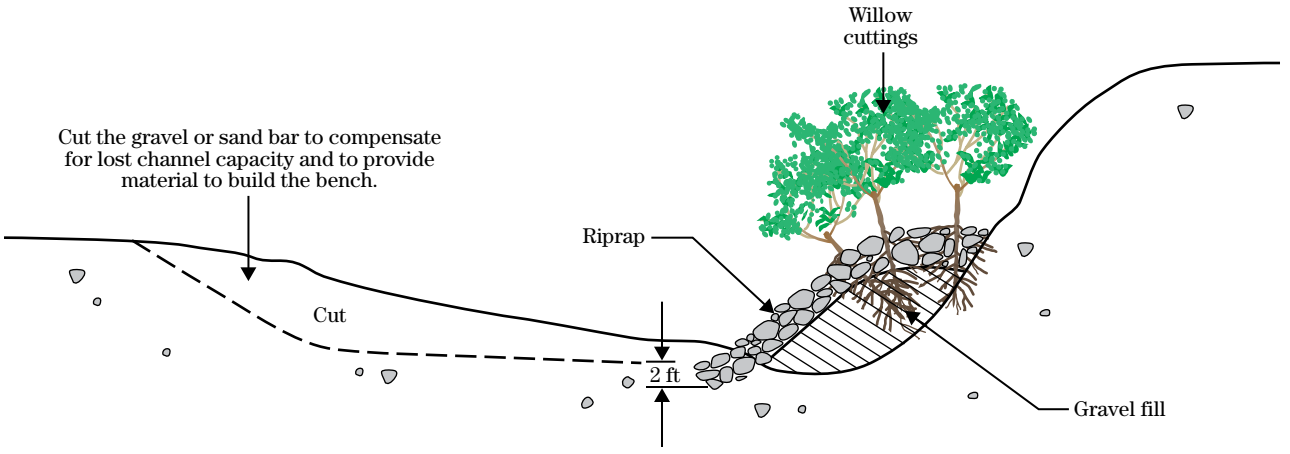
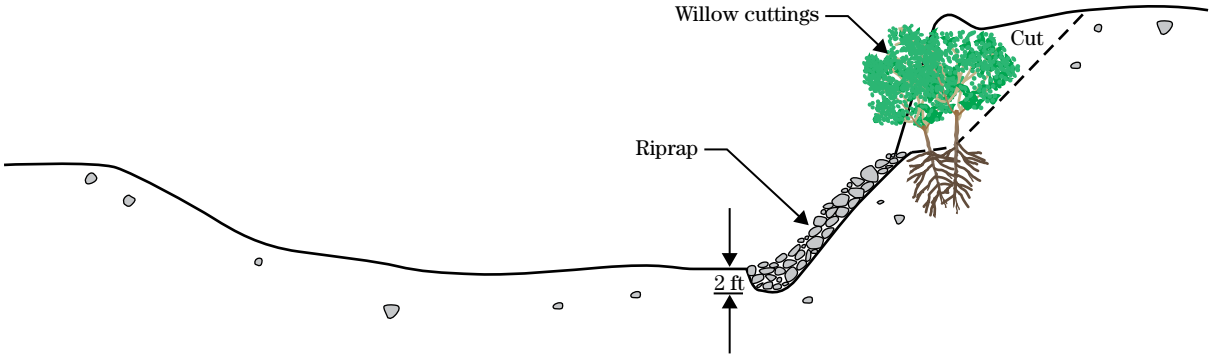


Figure TS14K–4 Excavated bench method



Steel dowel method

This method uses No. 8 or No. 6 steel reinforcing rods, depending on the size of the rock riprap. These rods are typically about 3 feet long and are grouted in place in holes that have been drilled into the bedrock (fig. TS14K–5). This method requires the larger rock be placed along the outer edge of the toe. The steel dowels are placed in position downslope against the large rocks that act as key stones in the toe to support the remainder of the rock riprap on the slope above. A modification of this approach is to drill holes into the toe rock and fit the stones over the steel dowels.

Precast toe blocks method

This method uses precast concrete blocks (fig. TS14K–6) to anchor the bottom row of riprap. The precast blocks should be 12 inches square and 5 feet long. Reinforcing rods extend 12 inches from each end of the blocks to form loops. These steel loops are placed so that they encircle steel bars which are drilled into the bedrock and grouted in place. The steel bars should be a minimum of 3 feet long and 1 inch in diameter (No. 8 bars). Where a 3-foot bar is used, a minimum of 2 feet should be grouted into the rock streambed. Because the blocks are of uniform length, bars are grouted in place on 6.5-foot centers. A template should be used when drilling holes to ensure proper spacing of the steel bars. The precast blocks are easily placed using a power crane. Wood planks should be used to protect the concrete blocks during the placement of the stone to avoid damaging the blocks by dropping stones on them. In channel sections where the bed is uneven, the steel loops may be bent so that they anchor to the steel bars properly.

Figure TS14K–5 Steel dowel method

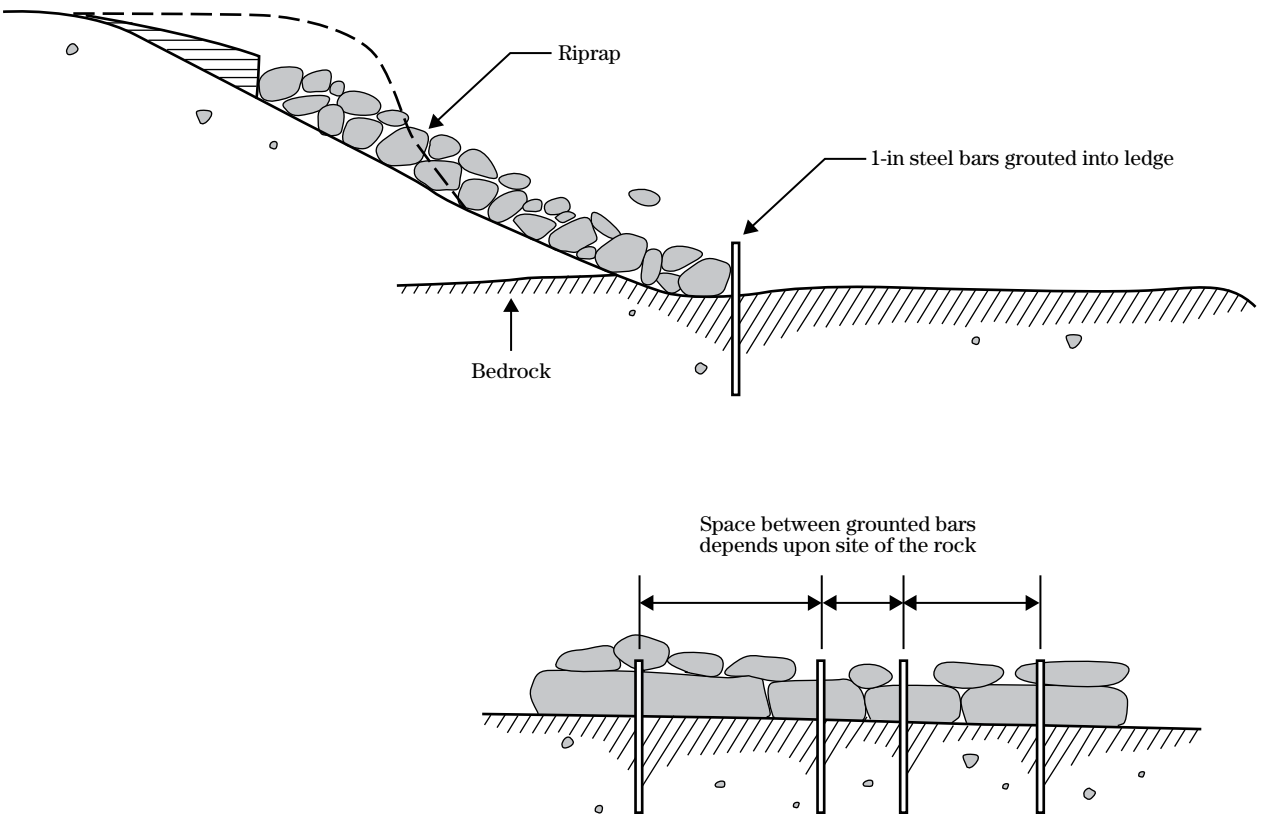
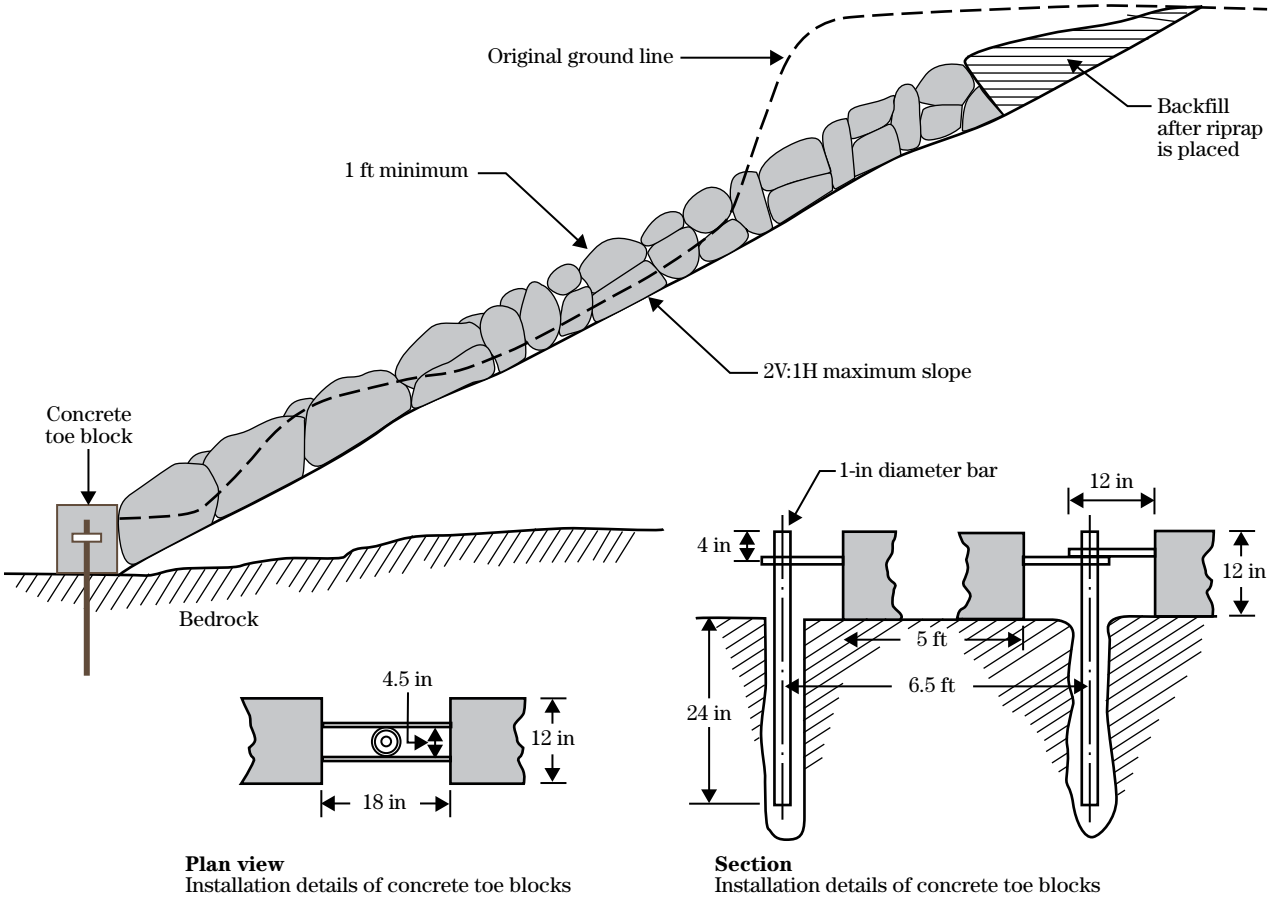


Figure TS14K–6 Precast toe block method



Plan view
Installation details of concrete toe blocks

Section
Installation details of concrete toe blocks

Other structural treatments

There are many structural streambank treatment techniques which involve the use of riprap. Several are briefly described, and others are described elsewhere in NEH654.14.

Stone with soil bioengineering

Combining rock with soil bioengineering treatments can achieve benefits from both techniques. Soil bioengineering is covered in more detail in NEH654 TS14J. The inert rock material often provides immediate toe protection, while the living plant materials protect, reinforce, and stabilize the banks.

Figure TS14K–7 shows a stone toe and live poles. The stone is keyed into the bed below an anticipated scour depth. Live poles can be installed with the aid of a waterjet stinger.

Figure TS14K–8 shows a brush layer being installed over a stone toe. Since the stone is not keyed into the bed, additional stone is placed in the toe. As the bed is scoured adjacent to the bank protection, this additional stone is available to fall into the scour hole.

Figure TS14K–9 shows a vertical bundle being installed under a stone toe. The bundles are placed in trenches which are then filled with soil. This minimizes potential damage to the live material during stone placement, as well as maximizes soil-to-stem contact.

Longitudinal peak stone toe

Longitudinal peak stone toe (LPST) involves the placement of a windrow of stone in a peak ridge along the toe of an eroding bank. The top of the stone is typically one-third to two-thirds of the bank height (Biedenharn, Elliott, and Watson 1997). LPST is particularly applicable where the upper bank is fairly stable, and the erosion is due to mass wasting from the toe of the bank. This technique protects the toe, while allowing the upper bank to stabilize on its own.

The main advantage of this technique is cost savings. An LPST is designed by specifying a weight or volume of rock to be placed along the length of the project reach, rather than finished elevations or dimensions.

Figure TS14K–7 Stone toe and live poles

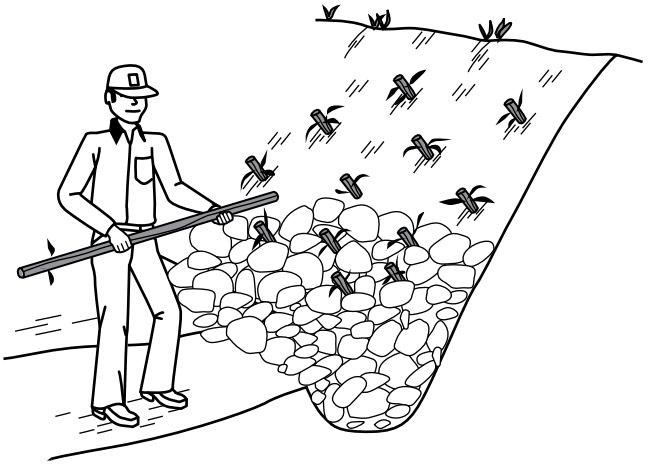


Figure TS14K–8 Brush layer over stone toe

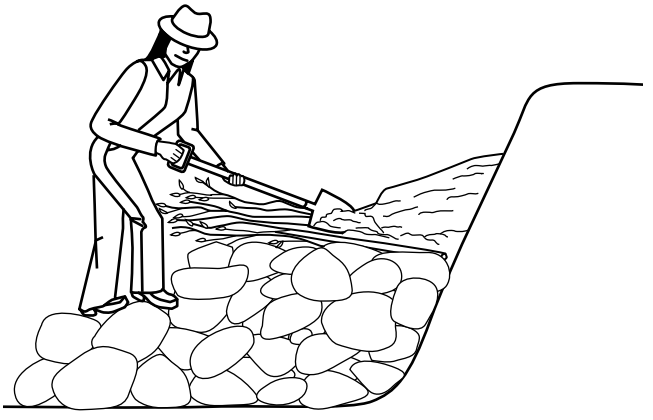


Figure TS14K–9 Vertical bundle and stone toe

On moderate-sized tributaries along the Mississippi River, typical applications can be 1 to 2 tons per linear foot, resulting in a triangular peak between 3 and 5 feet above the streambed (Biedenharn, Elliott, and Watson 1997). Usually, this simple technique is constructed by dumping stone from the bank. Since neither a filter layer nor geotextile fabric is used, a self-filtering, well-graded quarry run stone is specified. This technique depends on the rapid establishment of vegetation landward from the stone. Therefore, it is important to minimize disturbance of natural vegetation during installation, and it may be advisable to consider the addition of soil bioengineering practices.

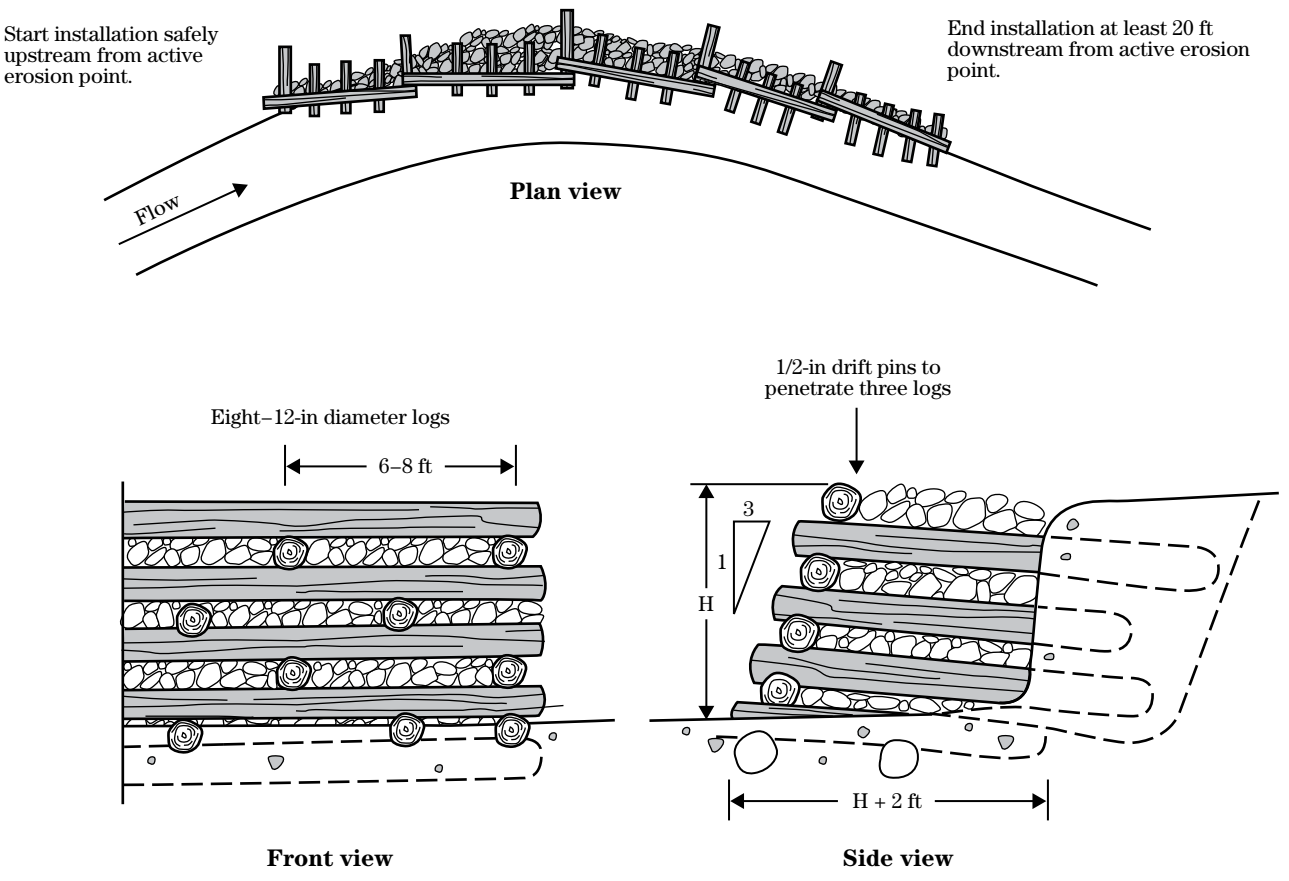
An LPST is often enhanced with the inclusion of woody debris and stone spurs along the length. These encourage deposition along the toe, create edge habitat, and move the higher velocity flow away from the bank.

Timber and rock cribbing

Timber cribbing backfilled with rock and coarse gravel is a traditional bank protection technique. This type of protection was popular many years ago when hand labor was more readily used in streambank protection. It has held up reasonably well, but becomes difficult to repair and maintain with age. Figure TS14K–10 illustrates a method of timber and rock cribbing.

The construction of a timber and rock crib requires considerable hand labor, and its useful life depends on the length of time the logs will hold the rock in place before rotting. As with gabions, the cribbing allows for the protection of unstable banks with stones that would be too small if used in a riprap revetment. While not exactly duplicating a riprap revetment, similar design characteristics are required for its design, such as scour, filtration, drainage, and length.

Figure TS14K–10 Timber and rock cribbing



Wire mesh gabions

Gabions offer important advantages for bank protection. They can provide vertical protection in high-energy environments where construction area is restricted. Gabions can also be a more affordable alternative, especially where rock of the needed size for riprap is unavailable. Gabion wire mesh baskets can be used to stabilize streambank toes and entire slopes. Gabions can also be compatible with many soil bioengineering practices. Gabions come in two basic types: woven wire mesh and welded wire mesh.

Woven wire mesh is a double-twisted, hexagonal mesh consisting of two wires twisted together in two 180-degree turns. Welded wire mesh has a uniform square or rectangular pattern and a resistance weld at each intersection. Within these two types there are two styles of gabions: gabion baskets and gabion mattresses. Baskets are 12 inches or more in height, while mattresses typically range from 5 to 12 inches in height.

Gabion baskets can be particularly effective for toe stabilization on problem slopes. They provide the size and weight to stay in place, with the further advantage of being tied together as a unit. Baskets can be installed in multiple rows to increase stability and provide a foundation for other measures above them. Gabion mattresses are best suited for revetment type installations, channel linings, and waterways. They may also be used for basket foundations and scour aprons.

Table TS14K–1 Specified rock sizes for gabions (from CS#64)

Gabion	Predominant rock size (in)	Minimum rock dimension (in)	Maximum rock dimension (in)
12-, 18-, or 36-in basket	4 to 8	4	9
6-, 9-, or 12-in mattress	3 to 6	3	7

All baskets and mattresses are of galvanized wire for corrosion protection. If the baskets are to be installed where abrasion from stream sediments is likely, PVC-coated material should be used. PVC coating adds significantly to the durability and longevity of the gabion installation. This coating provides long-term benefits for a relatively small increase in material costs.

It is important to use good quality rock of the proper size for gabion installation (table TS14K–1). Additional guidance on quality and sizing of rock can be found in ASTM 6711. Many manufacturers of gabions also provide guidance on the design and construction of their products.

Gabions can be delivered to the work site in a roll and in panels and can be partially or fully assembled. Assembly generally must be accomplished at the work site. Important in all aspects of assembly are the sizing, bracing, and stretching of the baskets or mattresses. Assembly and installation procedures are well covered in NRCS National Construction Specification (CS) #64 (USDA NRCS 2005). Details for assembly and placement of double-twisted, wire mesh gabions can also be found in ASTM D7014.

Important considerations in gabion placement are:

- The gabion is stretched and carefully filled with rock by machine or hand placement ensuring alignment, avoiding bulges, and providing a compact mass.
- Machine placement will require some hand work to ensure the desired results.
- The cells in any row shall be filled in stages so that the depth of stone placed in any cell does not exceed the depth of the stone in any adjoining cell by more than 12 inches.
- Along all exposed faces, the outer layer of stone shall be placed and arranged by hand to achieve a neat and uniform appearance (fig. TS14K–11).

The tops of gabions will also require some hand work to make them level and full prior to closing and fastening the basket lids. It is important that the gabion basket or mattress is full and the lids fit tightly. Appropriate tools need to be used in this operation and care taken not to damage the lids by heavy prying.

Various types of fasteners and lacing are used to assemble and secure gabion baskets and mattresses. The manufacturer's recommendations should be followed along with the applicable provisions in CS #64.

Vegetated gabion

In some locations, traditional gabions may be unacceptable from either an aesthetic or ecological perspective. A modification to traditional gabion protection that may satisfy these concerns is the vegetated gabion. A vegetated gabion incorporates topsoil into the void spaces of the gabion. The resulting gabion volume consists of 30 to 40 percent soil that allows root propagation between the stones. The resulting structure is interlocked with stone, wire, and roots (fig. TS14K–12).

Various commercial products, such as the Maccaferri Green Gabion™, provide improved shapes and an organic fiber matting to hold the soil in place while the plants become established. Figure TS14K–13 illustrates the assembly steps of such a gabion.

Grouted riprap

Grouted riprap is a riprap bed where the voids have been filled with concrete. It is often used where the required stone size cannot be obtained or at sites where a significant and damaging debris load is expected. Typical applications include grade protection, bank protection, spillways, inlets to debris basins, and as a repair to conventional riprap structures that have been damaged by high velocity flows. Culvert outfalls and ditch linings have also been constructed with grouted riprap. It has also been used to provide improved recreational access across riprap revetments.

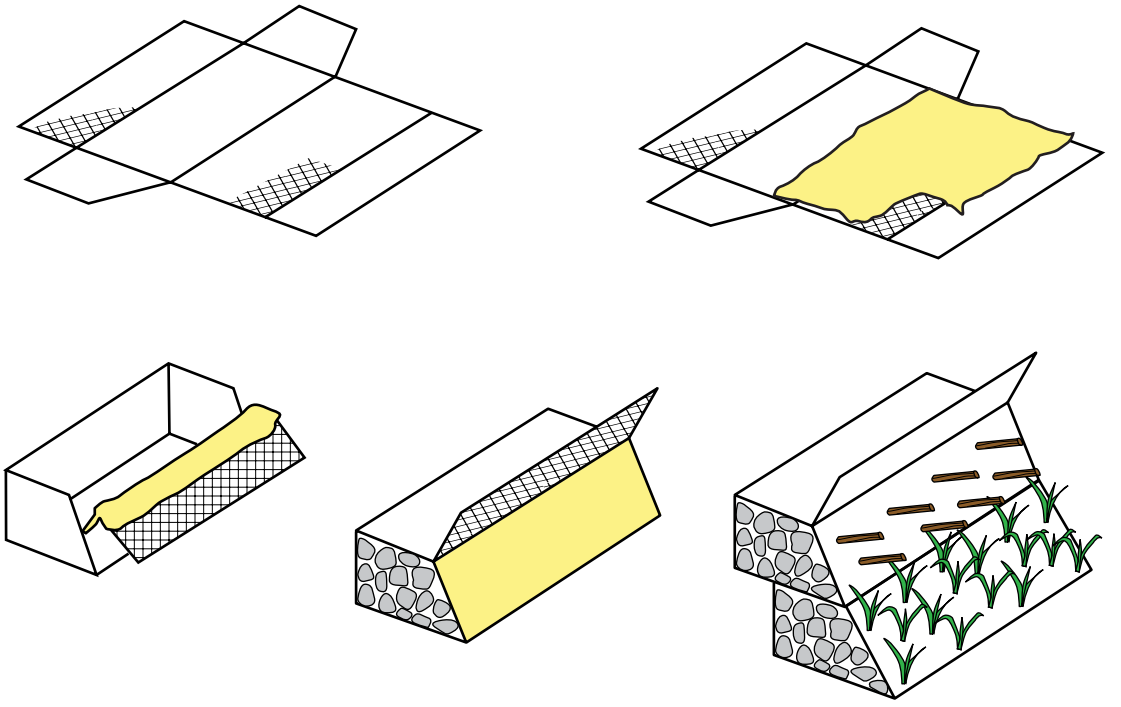
While the stone used for a grouted riprap installation can be smaller than what is required for a loose riprap installation, there is no available guidance that specifies a minimum size. Sizing is usually based on experience with similar projects in the area. The stone used should be as coarse as possible to allow for deep penetration of the grout. A general recommendation is that less than 5 percent of the stone should be less than 2 inches in diameter. Stone quality should be similar to that specified for conventional riprap structures.

The grout strength is typically 2,000 to 2,500 pounds per square inch. The grout must fully penetrate the stone to the subbase. Shoveling the grout over the stone may not fully penetrate the riprap. An immersion or pencil vibrator is often used to ensure that the voids between the stones are filled. The concrete mix should have a slump of 5 to 7 inches to allow for proper penetration. The maximum aggregate in the mix should be three-fourths inch. Typically, the grout is placed up to the top of the stones. However, in some applications,

Figure TS14K–11 Gabions showing a neat, compact, placement of stone with a uniform appearance

Figure TS14K–12 Vegetated gabions under construction

Figure TS14K–13 Assembly sequence of a Green Gabion™ (*Figure Courtesy of Maccaferri Gabions, Inc.*)



up to a third of the stone diameter is left exposed. This may be done for aesthetic reasons or to provide a more durable material to resist abrasion from sediment laden flows.

While the design of all rock structures must consider proper drainage to prevent hydrostatic pressure buildup, it is especially important for a grouted riprap design. Typically, relief holes composed of 3-inch-diameter pipes spaced at 10-foot intervals are set through the grouted structure and into the filtering system. Even well-designed grouted riprap structures will be subject to cracking, so the use of grouted riprap in areas that are subject to freeze-thaw action should be undertaken with caution. Further information on the design and construction of grouted riprap can be found in USACE ETL 1110–2–334 (USACE 1992).

The minimum thickness of the rock and grout is 12 inches. Thicker layers may be needed to prevent uplift of a structure during high flows. While guidance is limited concerning the required thickness, designers have balanced the uplift forces generated at maximum flow velocity against the weight of the cracked block size. In this analysis, the cracked units are assumed to have dimensions equal the thickness of the grouted riprap.

The ecological impacts of grouted riprap should be considered in the design. Since the voids in the riprap are filled, the structure will not provide refuge for small fish and macroinvertebrates. Plant growth through a grouted riprap structure is unlikely, and the thermal loading and lack of shade can contribute to increased stream water temperatures. Finally, grouted riprap is often viewed negatively from an aesthetics perspective, and this impact should be considered.

Habitat enhancement with stone

The designer should consider the habitat value when selecting stone gradations. For example, poorly graded, large stone may have limited habitat value for macroinvertebrates, since the openings are large. However, it may provide refuge for certain fish species.

Another application of habitat enhancement using stone is boulder clusters. These are sized using impinging flow design techniques. Boulder clusters or instream boulders provide structure and create hydraulic cover. Clusters are typically used in runs and glides in triangular-shaped groups of three to five boulders (EMSR–4–01, USACE 2005). The lee of the stones provides resting areas and inchannel refuge for fish during high-flow events. The turbulence generated by flows over and around the boulders diffuses sunlight and creates overhead cover. The tops of the boulders are typically just below the baseflow. They are generally not appropriate for use in sand-bed streams, since downstream scour may cause them to settle into the bed and disappear. Caution should also be exercised for use in braided streams. To avoid having the boulders cause excessive stress on the banks, they should not occupy greater than 10 percent of the channel area at bankfull flow or greater than a third of the width.

Conclusion

Many restoration designs require the use of rock in the stream. Riprap is one of the most effective protection measures at the toe of an eroding or unstable slope. Rock use has distinct advantages in terms of accepted design techniques and established contracting and construction procedures. In addition, many innovative bank stabilization and habitat enhancement projects use stone to perform important functions. Rock does present some drawbacks concerning cost, aesthetics, and ecological and geomorphic impacts. The challenge is to integrate more vegetative and geomorphic solutions without materially increasing the exposure time and risk of failure and meeting the goals of the project. This approach produces a long-term solution that will be complementary to the natural environment and will be more self-sustaining.

Technical Supplement 14L

Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Manufactured concrete blocks are available in a variety of thicknesses and configurations and can be used to stabilize the streambed or streambanks.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14L

Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects

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Technical Supplement 14L

Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects

Purpose

A variety of natural and manufactured materials can provide erosion protection for stream restoration and stabilization projects. One of these products is the articulating concrete block (ACB) revetment system. An ACB revetment system is a matrix of interconnected concrete block units installed to provide an erosion resistant revetment with specific hydraulic characteristics. It is static protection and is applicable in high risk applications where no additional bank or grade movement is allowable. This technical supplement describes the ACBs currently available and some of the benefits of their use. The system consists of concrete blocks, a filter (typically a geotextile), and cables in some products. A summary of testing for hydraulic performance is presented along with a design procedure for open channel flow. Critical installation features are described for typical installations including subgrade preparation, ancillary components (such as drainage layers), filter placement, ACB placement, system termination, anchors, and penetrations.

Introduction

Stream restoration and stabilization may require the use of armoring countermeasures to provide lateral or vertical stability to a stream. Armoring countermeasures include concrete lining and other rigid revetments, rock riprap, gabion baskets, gabion mattresses, or ACB revetment systems. These countermeasures result in a statically stable stream within the armored area. Armoring countermeasures provide permanent erosion protection to underlying soil from the forces of flowing water. Armoring countermeasures may be used when vegetation and other soil bioengineering practices are not suitable or unstable under the stress or duration of the design event or where the consequences of failure are unacceptable. The designer should keep in mind that since its use results in a static section, other stability and ecological issues may become a concern. Typical applications of the ACB revetment system include entire channel cross-sectional protection, toe and lower side slope protection, and grade stabilization structures.

An ACB revetment system consists of a matrix of interconnected concrete block units sufficient for erosion protection. The individual units are connected by geometric interlock, cables, ropes, geotextiles, geogrids, or a combination thereof and, typically, overlay a geotextile for subsoil retention (American Society for Testing and Materials (ASTM) D6684). The filter layer may consist of a geotextile, properly graded granular filter, or both. Proper design of the filter layer is critical to the successful performance of the ACB revetment system. The individual blocks of the system are able to conform to changes in the subgrade, while remaining connected due to the geometric interlock or other system components such as cables.

The intent of this section is to provide an introduction to the applications, materials, hydraulic testing, design, specification, and installation of ACB revetment systems for stream restoration and stabilization projects. This technical supplement does not address stream stability, hydraulic analyses of the stream flow, or geotechnical analyses and slope stability of the stream slopes. ACBs do not provide strength to a slope; therefore, a protected slope must be geotechnically stable prior to placement of the ACB revetment system.

Applications

ACB revetment systems have been used in a variety of applications for streambank stabilization and restoration projects (figs. TS14L–1 through TS14L–6). These applications include:

- armoring the entire cross section
- armoring the toe and lower slope
- armoring the toe and side slope
- streambed grade stabilization
- armoring of pipe/culvert outlets
- scour protection around bridge piers

Figure TS14L–1 Armoring the entire cross section

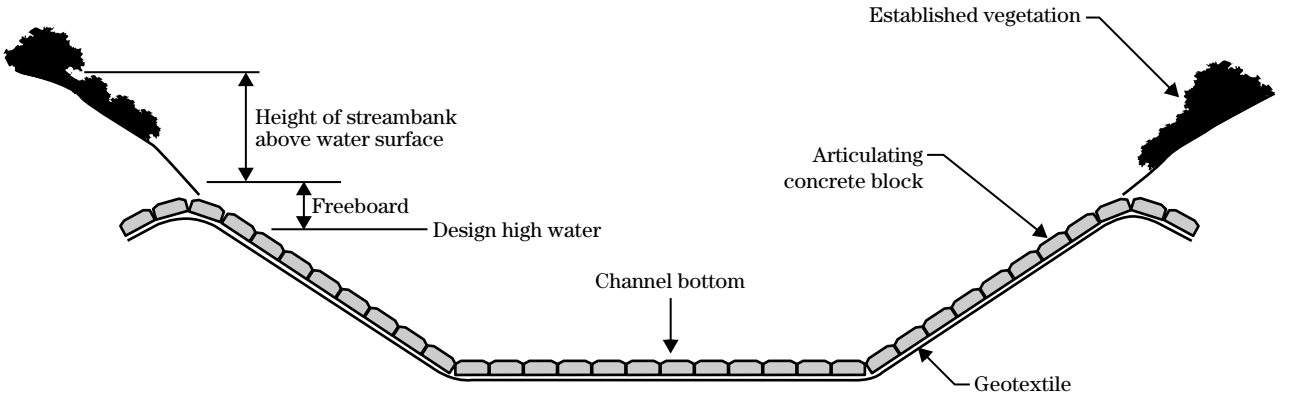
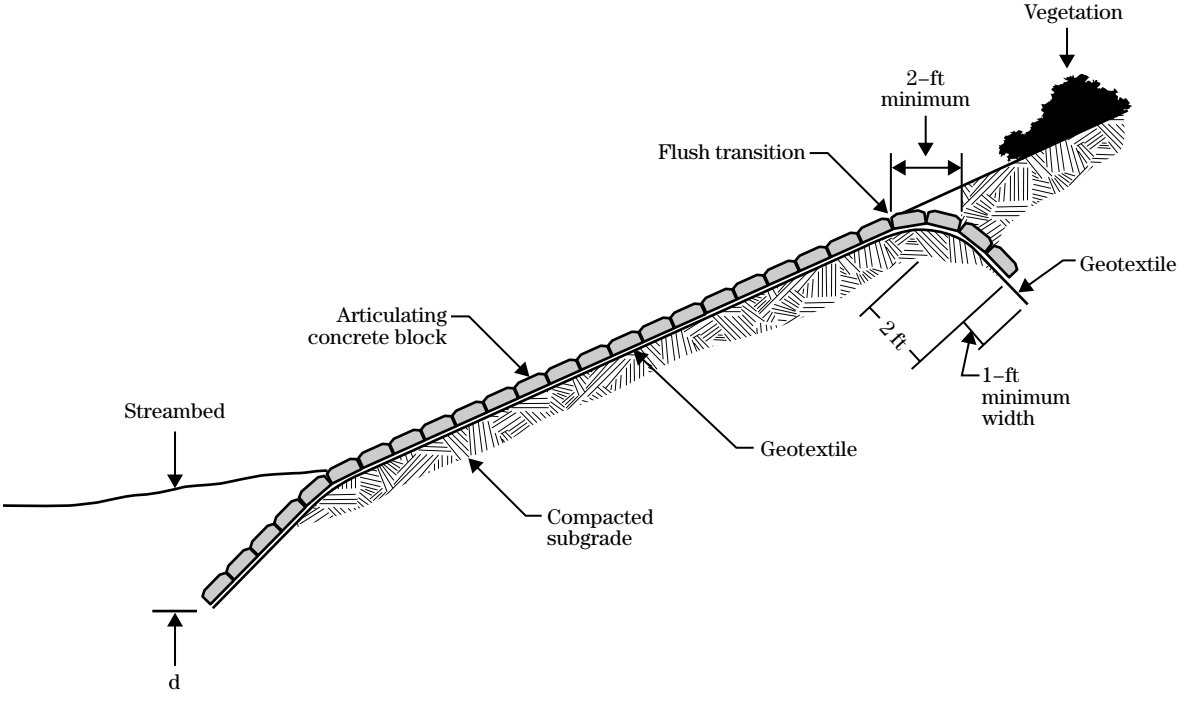
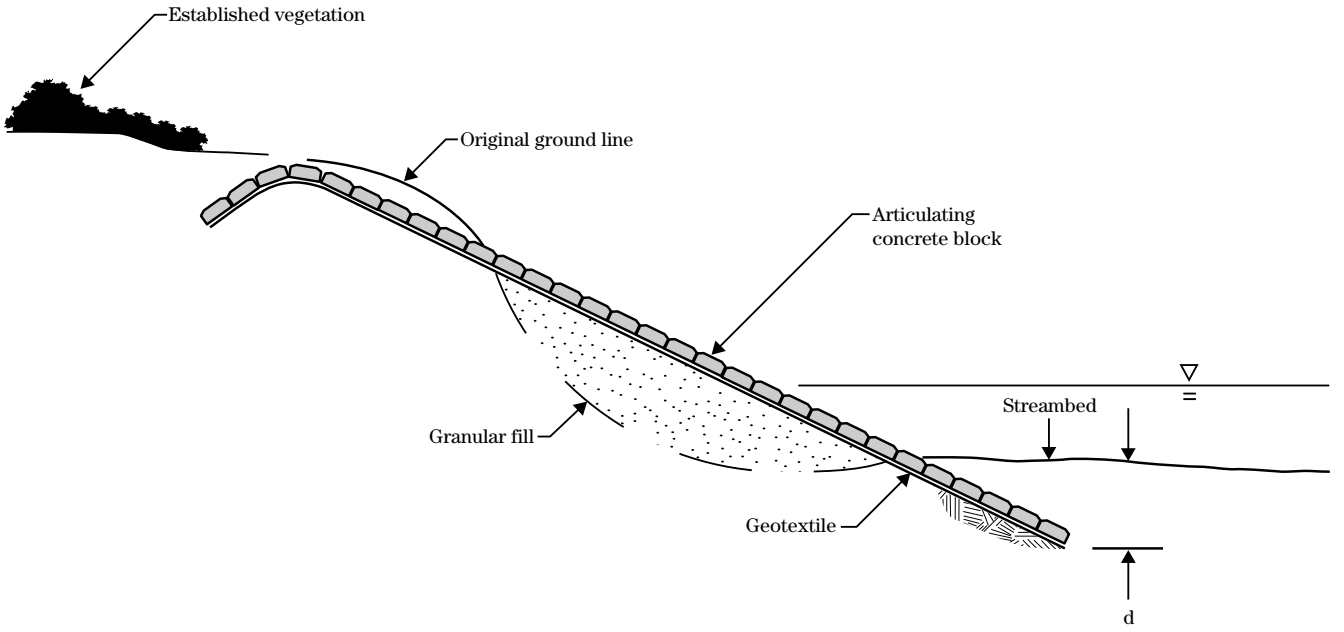


Figure TS14L–2 Armoring the toe and lower slope cross section



d = maximum depth of scour

Figure TS14L–3 Armoring the toe and slide slope cross section



d = maximum depth of scour

Figure TS14L–4 Streambed grade stabilization profile

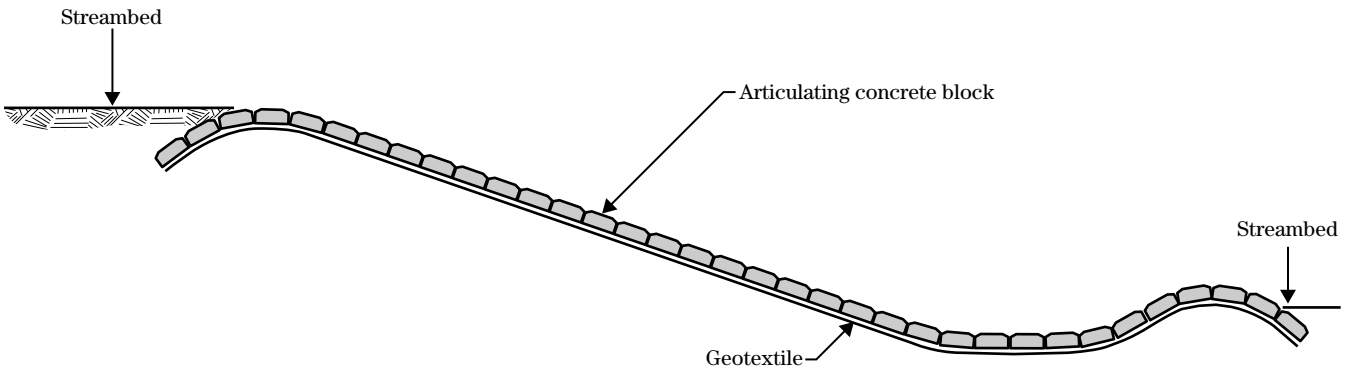


Figure TS14L–5 Armoring of pipe/culvert outlets profile

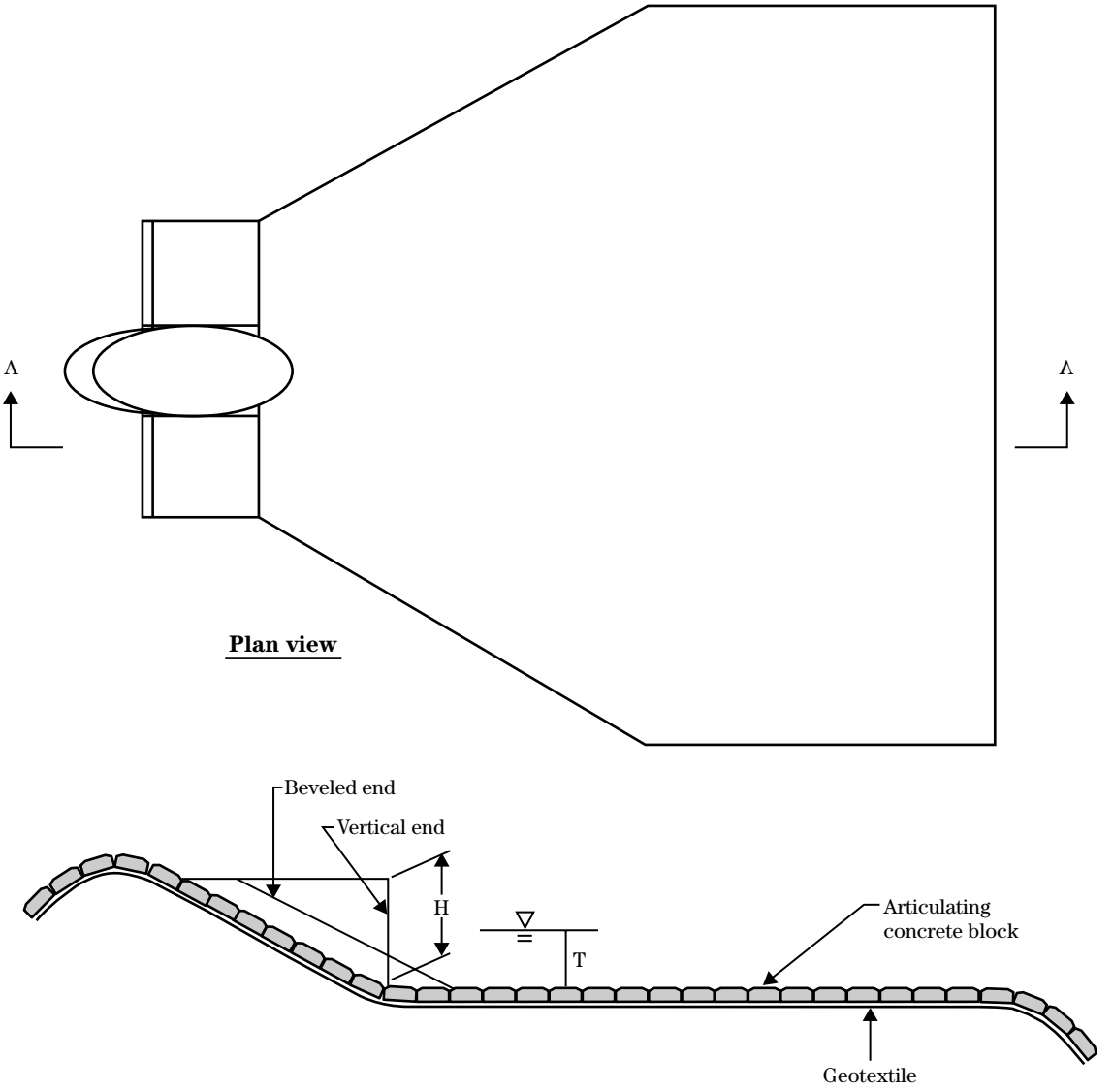
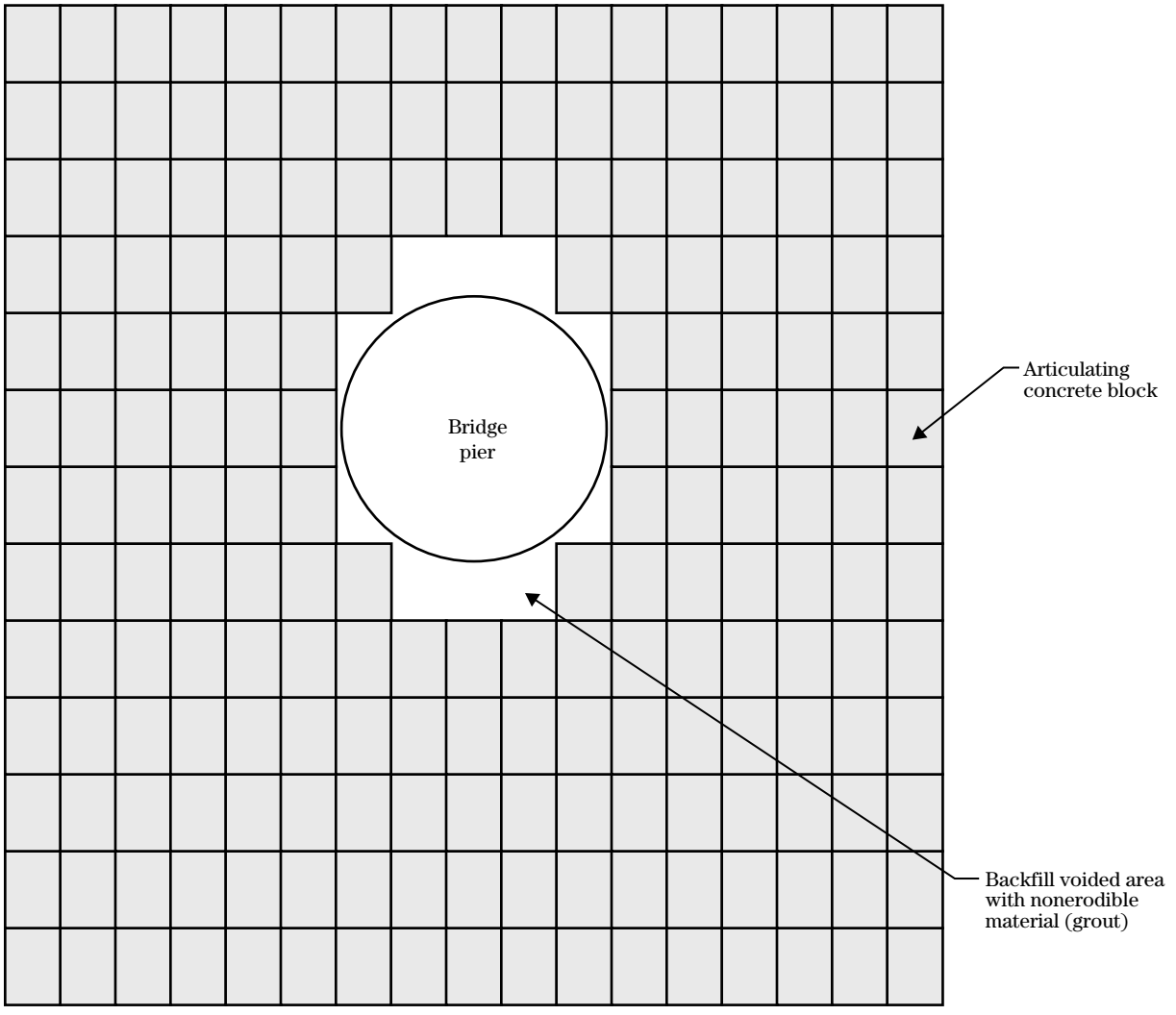


Figure TS14L–6 Scour protection around bridge pier plan



Plan view

Note:
Termination trench required both upstream and downstream

Materials

Blocks

Several proprietary ACB revetment systems are available. The blocks can be made in a variety of shapes and thicknesses. The thickness of available blocks typically ranges from 4 inches to 9 inches. Tapered and wedge-shaped blocks are also available. Figure TS14L–7 shows some of the block shapes available for ACB revetment systems.

The blocks are made of precast concrete. The blocks are cast into interlocking or noninterlocking shapes. The blocks may be cabled into mats or can be noncabled. Blocks to be cabled usually have preformed holes cast in them for placement of the cable, although some systems are manufactured with the blocks cast directly onto the cables. The holes should be smooth to prevent damage to the cable.

The blocks may be open cell or closed cell. Open-cell block systems provide an overall open area ranging from 17 to 23 percent for the system. The open area allows soil to be placed into them or for sediment to fill in the open areas and become vegetated.

Closed-cell block systems provide an open area of approximately 10 percent and allow for some trapped soil and vegetation growth. Although the cable concrete block developed by International Erosion Control Systems is a closed cell, the individual blocks can be spaced to provide an open area of greater than 20 percent.

Connections

Individual blocks that are connected into a mat are often referred to as cabled systems. The cable may consist of ropes, polyester revetment cable, or galvanized or stainless steel cable. An underlying geotextile or geogrid is sometimes used in lieu of cables, and the blocks are attached with adhesive. The individual blocks may be assembled into mats offsite or constructed onsite by hand placement.

The most widely used connections consist of polyester revetment cable and steel cable. Steel cable is typically stainless steel aircraft cable of type 302, 304, or 316 (fig. TS14L–8). Typical steel cable specifications are shown in table TS14L–1.

Polyester cable is typically constructed of high tenacity, low elongating, and continuous filament polyester fibers (fig. TS14L–9). Cable consists of a core construction comprised of parallel fibers contained within an outer jacket or cover. The weight of the parallel core is between 65 percent to 70 percent of the total weight of the cable. Typical polyester cable specifications are shown in table TS14L–2.

Geotextiles

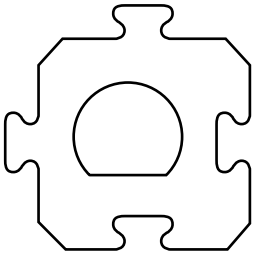
Geotextiles are typically used to retain the soil particles serving as the subgrade for the ACBs (fig. TS14L–10). Geotextiles may be woven or nonwoven and may be composed of multifilament yarns or monofilament yarns. Woven slit film (monofilament or multifilament) geotextiles should not be used as a filter beneath ACBs since the materials are weak, and the opening size and percent open area are unpredictable. Nonwoven geotextiles should be needle-punched and not be heat-bonded or resin-bonded, nonwoven geotextiles. The permeability of heat-bonded and resin-bonded nonwoven geotextiles is too low to allow adequate seepage and dissipation of hydrostatic pressure. Geotextiles are addressed in more detail in NEH654 TS14D. More detailed descriptions of geotextile materials may also be found in Harris County Flood Control District (HCFCD) 2001; American Association of State Highway Transportation Officials (AASHTO) 2000; and U.S. Army Corps of Engineers (USACE) (1995b).

Granular filter

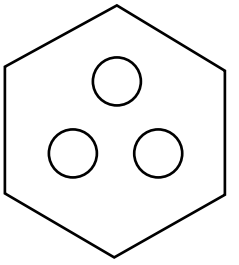
The purpose of the granular filter is to intercept water flowing through the pores of the subgrade soil, allowing for the passage of the water, while retaining the subgrade soil particles. Granular filters consist of sand, gravel, or a sand and gravel mixture and may contain some fine-grained particles.

Fine sand or silt subgrade soils may require the use of a dual granular filter or a combination of a granular filter and a geotextile designed to retain the underlying granular soil. A combination of a granular filter and a geotextile are shown in figures TS14L–10 and TS14L–11.

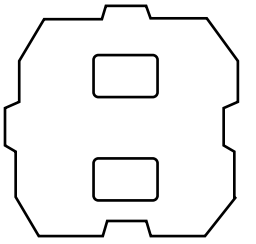
Figure TS14L–7 Examples of ACB revetment systems (*Figures courtesy of HCFCD*)



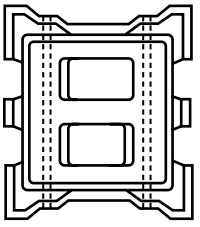
Channellock™



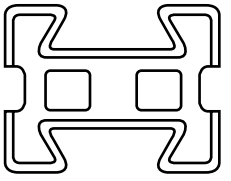
Hex lock™



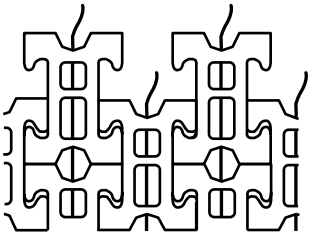
Petraflex™



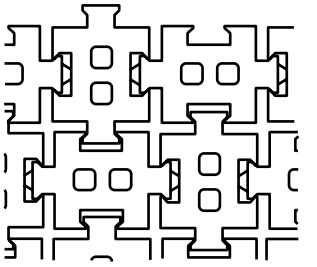
Armorflex™



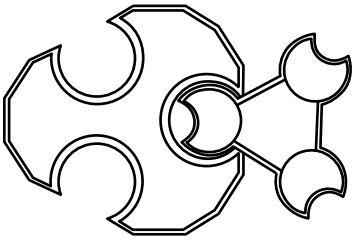
Armorloc™



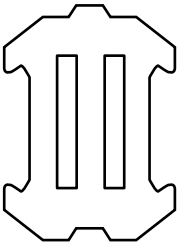
Conlock I™



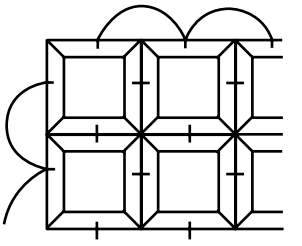
Conlock II™



Trilock™



Geolink™



Cable concrete™

Figure TS14L–8 Steel cables

Figure TS14L–9 Polyester cables

Table TS14L–1 Steel cable specifications

Diameter	Construction	Breaking strength
1/8 in	1 by 19	2,100 lb
5/32 in	1 by 19	3,300 lb
3/16 in	1 by 19	4,700 lb

Table TS14L–2 Polyester cable specifications

Cable diameter (in)	Average strength (lb)	Weight, lb/100 ft Minimum	Weight, lb/100 ft Maximum
1/4	3,700	2.47	2.74
5/16	7,000	3.99	4.42
3/8	10,000	4.75	5.26
1/2	15,000	8.93	9.90

Figure TS14L–10 ACB section with a geotextile filter and combination geotextile and granular filter (*Figure courtesy of HCFCD*)

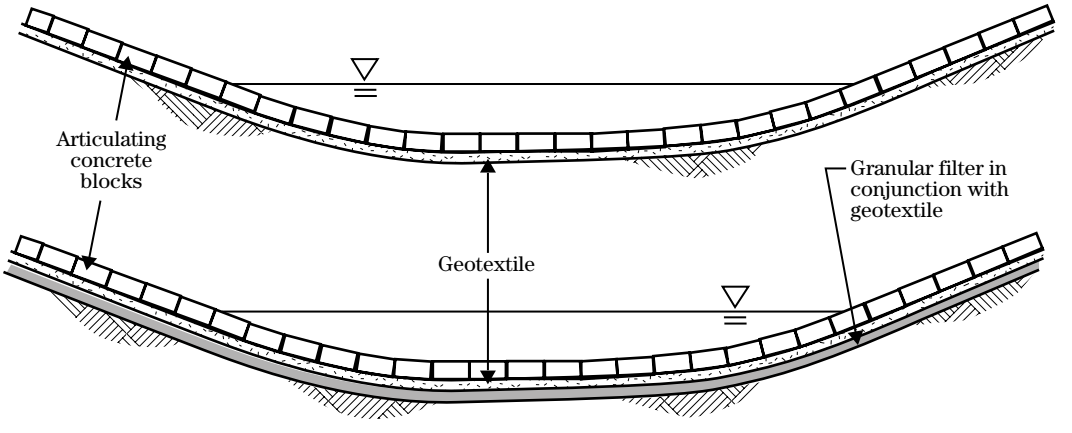


Figure TS14L–11 ACBs in combination with granular and geotextile filter

Performance testing and evaluation

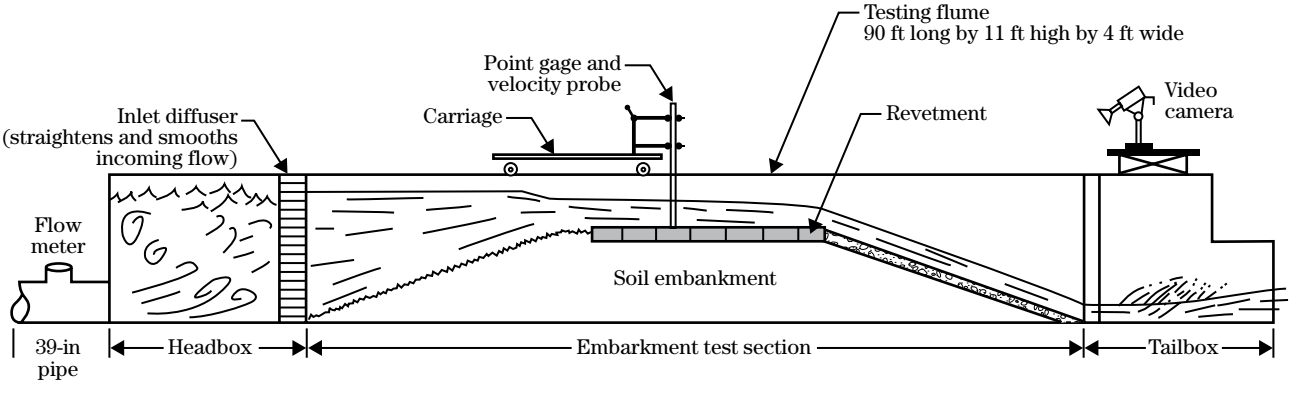
Due to the proprietary nature and unique characteristics of the ACB revetment systems available, a hydraulic stability test should be completed on each family of blocks. The hydraulic stability test should be conducted in accordance with U.S. Department of Transportation Federal Highway Administration (FHWA) RD–89–199 (Clopper 1989). Research conducted throughout the 1980s (Clopper and Chen 1988; Clopper 1989) led to a definition of failure for ACB revetment systems as the local loss of intimate contact between the ACB and the subgrade. The FHWA study (Clopper 1989) identified the following four conditions which may lead to this definition of failure:

- loss of soil beneath the system by gradual erosion beneath the system or washout through the system at joints and open cells
- deformation of the subgrade due to liquefaction and shallow slip failures caused by the ingress of water beneath the system (especially in silty soils on steep slopes)
- loss of block or a group of blocks (uncabled systems) which directly exposes the subgrade to the flow
- flow beneath the ACB causing uplift pressures and separation of the block from the subgrade

Although loss of intimate contact may not lead to total failure of the system, the stability and continued performance of the system has been compromised.

Each ACB revetment system obtains its stability from a unique set of weight, interblock restraint, geometry, and block-to-block articulation. Therefore, laboratory testing of each family of ACB revetment systems is required to determine the critical shear stress. A schematic of a typical laboratory test flume is shown in figure TS14L–12.

Figure TS14L–12 Schematic of a typical laboratory test flume for ACB performance testing (*Figure courtesy of HCFCD*)



The forces causing overturning and restraining moments are illustrated in figure TS14L–13.

Equation TS14L–1 (HCFCD 2001) shows the restraining moments on the left and overturning moments on the right side of the equation:

$$\ell_2 W_{S2} = \ell_1 W_{S1} + \ell_3 (F_D + F'_D) + \ell_4 (F_L + F'_L)$$

(eq. TS14L–1)

The drag force, F'_D, due to protruding blocks (fig. TS14L–14) is a function of the flow velocity and may be expressed by the following equation:

Figure TS14L–13 Forces on an ACB revetment system during performance test (*Figure courtesy of HCFCD*)

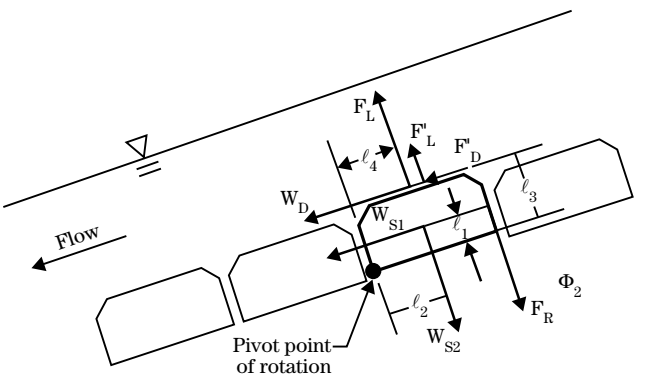
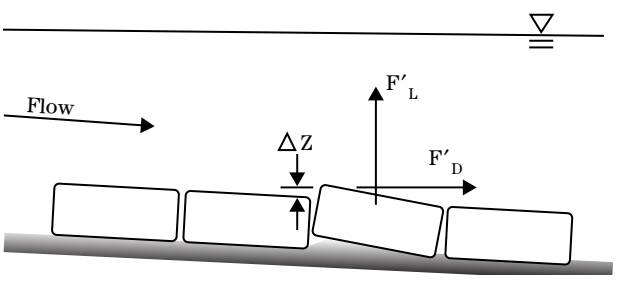


Figure TS14L–14 Forces on a protruding block (*Figure courtesy of HCFCD*)



$$F'_D = \frac{1}{2} \times C_D (\Delta Z) b \rho V^2$$

(eq. TS14L–2)

where:

F_D' = drag force due to block protrusion (lb)
C_D = drag coefficient ($C_D \approx 1.0$)
ΔZ = height of protrusion (ft)
b = block width perpendicular to flow (ft)
ρ = density of water (1.94 slugs/ft^3)
V = velocity (ft/s)

The added lift force (F_L') due to the block protruding above the ACB matrix is assumed equal to the drag force.

The ACB design procedure is based on the critical shear stress for a horizontal surface. Performance testing is typically conducted on bed slopes of 2H:1V or 3H:1V. The following equation (HCFCD 2001) may be used to extrapolate the test results to a horizontal surface:

$$\tau_{C\theta U} = \tau_{C\theta T} \times \left(\frac{\ell_2 \cos\theta_U - \ell_1 \sin\theta_U}{\ell_2 \cos\theta_T - \ell_1 \sin\theta_T} \right)$$

(eq. TS14L–3)

where:

$\tau_{C\theta U}$ = critical shear stress for untested bed slope (lb/ft^2)
$\tau_{C\theta T}$ = critical shear stress for tested bed slope (lb/ft^2)
θ_U = untested bed slope (degrees)
θ_T = tested bed slope (degrees)
ℓ_x = moment arms (ft)

Performance testing is also typically conducted on one block within the same family. An equation has been developed for extrapolating test results from a tested block to an untested block of similar characteristics. The equation should only be used to extrapolate results for a thicker block within the same family as the tested block. This equation is also based on a moment balance approach that neglects interblock restraint. Equation TS14L–4 (Clopper 1991) is suggested for extrapolation of test results from one block to a thicker block within the same family:

$$\tau_{CU} = \tau_{CT} \times \left(\frac{W_{SU} \ell_{2U}}{W_{ST} \ell_{2T}} \times \frac{\ell_{3T} + \ell_{4T}}{\ell_{3U} + \ell_{4U}} \right)$$

(eq. TS14L–4)

where:
τ_{CU} = critical shear stress for untested block (lb/ft^2)
τ_{CT} = critical shear stress for tested block (lb/ft^2)
W_{SU} = submerged weight of untested blocks (lb)
W_{ST} = submerged tested blocks (lb)
ℓ_{xU} = moment arms of untested blocks
ℓ_{xT} = moment arms of tested blocks (ft)

The moment arms used in these two equations should apply to the orientation of the block during testing and are not necessarily the same as those suggested later in the document for design.

Design procedure

The design of ACB revetment systems must be based on hydraulic analyses of the open channel during the design event. The hydraulic analyses should provide the shear stress and velocity associated with the design event. An example calculation is provided at the end of this technical supplement. The cross-sectional average shear stress may be used for most open channel flow applications. For applications such as bends, confluences, flow constrictions, or flow obstructions, a more detailed, area-specific hydraulic analysis should be considered. Site aesthetics and impacts to habitat should also be considered.

Factor of safety

The design engineer must determine the factor of safety to be used for a particular project. The determination should consider the risks associated with the failure of the ACB revetment system, complexity of the hydraulic system, the uncertainties in hydrologic and hydraulic analyses, and uncertainties associated with ACB revetment system installation. Typically, a minimum factor of safety of 1.5 is used for stream revetment project design. A higher factor of safety of 2.0 should be considered for protection around bridge piers, abutments, at channel bends, or other complex hydraulic systems. A systematic procedure to select a project-specific factor of safety is presented in HCFCD (2001).

Blocks

Failure (loss of intimate contact) is typically the result of the overturning of a block or group of blocks about the downstream contact point of the block. The hydraulic stability of a block on a channel side slope is a function of the magnitude and direction of stream velocity and shear stress, the depth of flow, channel side slope, channel bed slope, interblock restraint, block geometric properties, and the weight of the block. The definition of the forces, dimensions, and angles used in the equation for the factor of safety are depicted in figures TS14L–15 and TS14L–16. The factor of safety equations are defined in table TS14L–3 (HCFCD 2001).

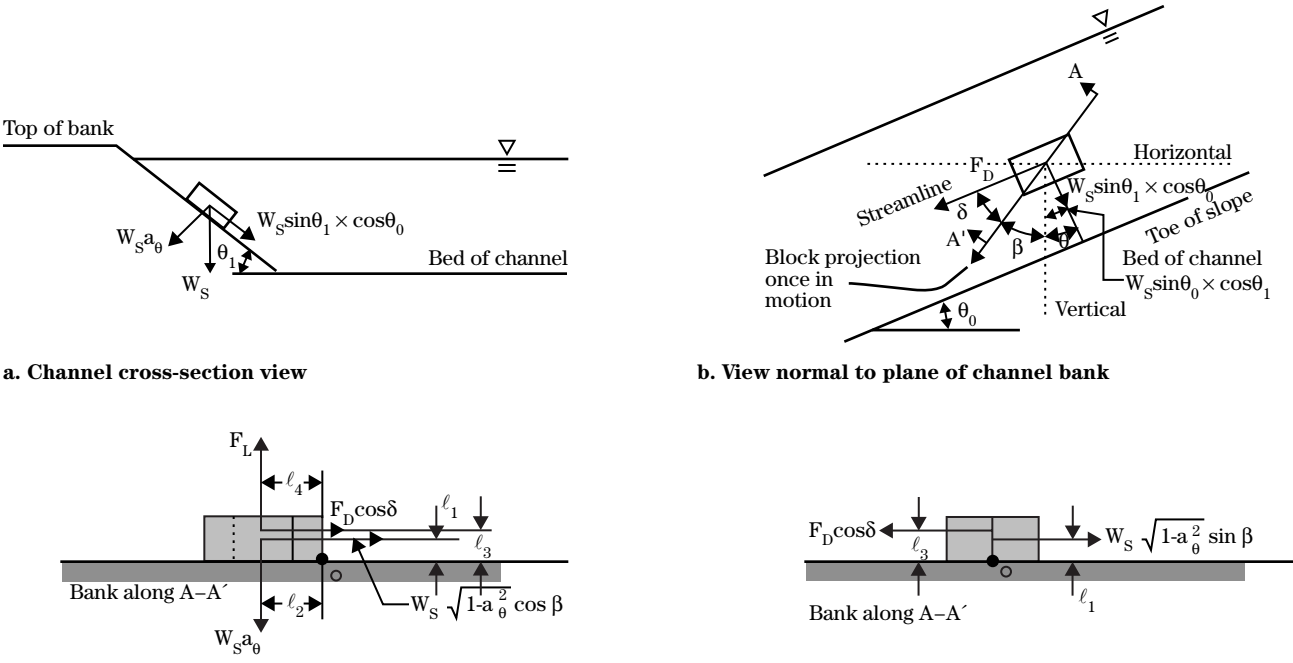
Filter

An appropriate filter design is critical to the successful performance of the ACB revetment system. Design of both a geotextile filter and a granular filter includes determining criteria for filtering and permeability.

References available for design of a geotextile filter include HCFCD (2001); U.S. Department of Agriculture Soil Conservation Service (SCS) (1991); AASHTO (2000), and USACE (1995b). Each of the references includes an analysis of the appropriate geotextile Apparent Opening Size and its permeability. The maximum Apparent Opening Size will allow suitable retention of soil particles, while the minimum geotextile permeability will allow the free flow of water without a buildup of excessive hydrostatic pressure.

Granular filter design criteria are presented in the Natural Resources Conservation Service (NRCS) National Engineering Handbook (NEH), part 633, chapter 26, Gradation design of sand and gravel filters (USDA NRCS 1994). This document provides filter criteria based on the percent finer than the number 200 sieve of the subgrade soil. It also recommends a minimum permeability for any subgrade soil.

Figure TS14L–15 Block on a side slope with design variables (*Figure courtesy of HCFCD*)



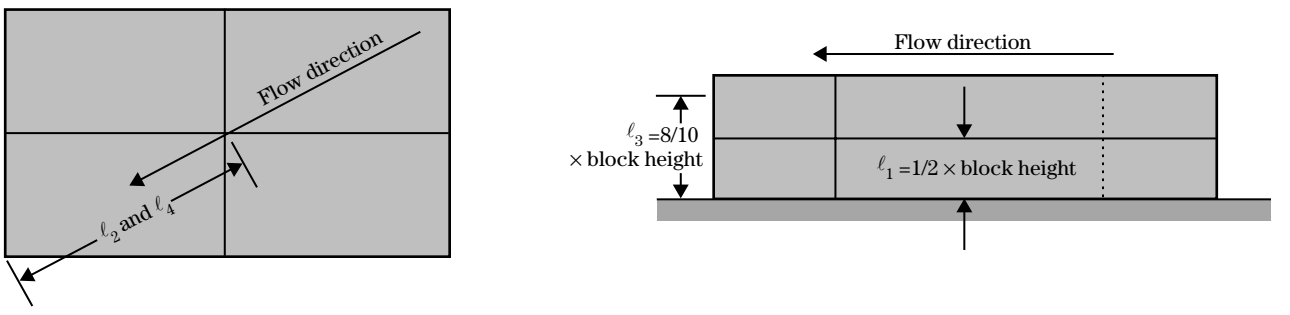
a. Channel cross-section view

b. View normal to plane of channel bank

c. View of section A–A′

d. View normal to section A–A′

Figure TS14L–16 Block moment arms (*Figure courtesy of HCFCD*)



a. Plan view of block design moment arms shown

b. Profile view of block with design moment arms shown

Table TS14L–3 Design equations for ACB revetment systems

$$FS = \frac{\left(\frac{\ell_2}{\ell_1}\right)a_\theta}{\sqrt{1-a_\theta^2}\cos\beta + \eta_1\left(\frac{\ell_2}{\ell_1}\right) + \frac{\left(\ell_3 F'_D \cos\delta + \ell_4 F'_L\right)}{\ell_1 w_S}}$$

$$\delta + \beta + \theta = 90^\circ \text{ or } \frac{\pi}{2} \text{ radians}$$

$$\eta_1 = \left(\frac{\frac{\ell_4}{\ell_3} + \sin\left(\theta_0 + \theta + \beta\right)}{\frac{\ell_4}{\ell_3} + 1}\right)\eta_0$$

$$\beta = \arctan\left(\frac{\cos\left(\theta_0 + \theta\right)}{\left(\frac{\ell_4}{\ell_3} + 1\right)\frac{\sqrt{1-a_\theta^2}}{\eta_0\left(\frac{\ell_2}{\ell_1}\right)} + \sin\left(\theta_0 + \theta\right)}\right)$$

$$\theta = \arctan\left(\frac{\sin\theta_0}{\sin\theta_1} \times \frac{\cos\theta_1}{\cos\theta_0}\right) = \arctan\left(\frac{\tan\theta_o}{\tan\theta_1}\right)$$

$$a_\theta = \sqrt{\cos^2\theta_1 - \sin^2\theta_0}$$

$$\eta_0 = \frac{\tau_{des}}{\tau_c}$$

$$F'_L = F'_D = 0.5 \times (\Delta Z) b \rho V_{des}^2$$

$$W_S = W \times \left(\frac{G_c - 1}{G_c}\right)$$

a_θ = projection of W_s into subgrade beneath block
b = block width (ft)
F'_D = additional drag
F'_L = additional lift force (lb)
P_x = block moment arms (ft)
G_c = specific gravity of concrete (assume 2.1)
FS = calculated factor of safety
V_{des} = design velocity (ft/s)
W = weight of block (lb)
W_s = submerged weight of blocks (lb)
ΔZ = height of block protrusion above ACB matrix (ft)
δ = angle of block projection from downward direction, once in motion
η_0 = angle between drag force and block motion
η_1 = stability number for a sloped surface
θ = angle between side slope projection of W_s and the vertical
θ_0 = channel bed slope (degrees or radians)
θ_1 = channel side slope (degrees or radians)

Note: the equations cannot be solved for:
$\theta_1 \cong$ (division by 0); therefore, a negligible side slope must be entered for the case of $\theta_1 \cong 0$
ρ = mass density of water (1.94 slugs/ft^3)
τ_c = critical shear stress for block on a horizontal surface (lb/ft^2)
τ_{des} = design shear stress (lb/ft^2)

Specifying ACB revetment systems

Blocks

The blocks should meet the physical requirements of ASTM D6684, Standard Specification for Materials and Manufacture of Articulating Concrete Block Revetment Systems. Table TS14L–4 presents the physical requirements in specified in ASTM D6684.

In areas subject to freeze-thaw, the number of freeze/thaw cycles and the corresponding weight loss criterion should be specified. Some specifications require 100 freeze-thaw cycles, with no more than 1 percent weight loss as determined on five block samples. The minimum percent open area should also be specified.

Connections

If a cabled system is desired, the cable specifications recommended in this paper should be considered. If the blocks will be adhered to a geotextile, the geotextile should meet the geotextile specifications described in the following section.

Geotextile

The NRCS has developed national construction and material specifications for geotextiles. These are included in NEH, part 642, Specifications for Construction Contracts. Additional material is covered in NEH654 TS14D. The NRCS specifications are broken into woven and nonwoven geotextiles and into various classes. Class I geotextiles are typically specified for erosion protection systems. The class I material properties included in the NRCS material specifications are shown in tables TS14L–5 and TS14L–6.

Testing

A hydraulic stability test conducted in accordance with FHWA RD–89–199 on the proposed ACB revetment system family should be specified. The streambed slope of the project should be no steeper than the slope used in the hydraulic stability test. If the ACB revetment system is tested with system restraints (such as mechanical anchors) or ancillary components (such as a synthetic or granular drainage medium), these features should also be incorporated into the field installations.

Design

The project-specific design criteria should be specified to allow each ACB revetment system manufacturer to calculate which product should be supplied. The following project conditions should be specified:

- design velocity (ft/s)
- design shear stress (lb/ft^2)
- bed slope (ft/ft)
- side slope (H:V) (ft/ft)
- maximum allowable block-to-block placement tolerance (in)
- minimum required factor of safety

Installation

Detailed specifications are required for the installation of ACB revetment systems. Detailed construction specifications for earthwork (including subgrade preparation) and placement of the geotextile are available from the NRCS, USACE, HCFCD, and other organizations. Specifications for ACB installation are available from the USACE, HCFCD, ACB manufacturers, and other organizations, as well. An ASTM Standard Practice for the installation of ACB revetment systems is under development. General installation considerations are listed.

Subgrade preparation

The ACB revetment system should be placed on undisturbed *in situ* soils or properly compacted fill. The subgrade for ACB placement should be graded smooth to ensure that intimate contact is achieved between the soil surface and the geotextile.

Table TS14L–4 Block physical requirements

Minimum compressive strength lb/in²		Maximum water absorption lb/ft³		Minimum density lb/ft³	
3 unit avg.	Individual unit	3 unit avg.	Individual unit	3 unit avg.	Individual unit
4,000	3,500	9.1	11.7	130	125

Table TS14L–5 NRCS specifications for woven geotextiles

Property	Test method	Class I
Tensile strength (lb) 1/	ASTM D4632	200 minimum in any principal direction
Elongation at failure (%) 1/	ASTM D4632	<50
Puncture (lb) 1/	ASTM D4833	90 minimum
UV (% residual tensile strength)	ASTM D4355 150-hr exposure	70 minimum
Apparent opening size 2/	ASTM D4751	As specified, but no smaller than 0.212 mm (#70) 3/
Percent open area (%)	CWO–02215	4.0 minimum
Permitivity s^{-1}	ASTM D4491	0.10 minimum

1/ Minimum average roll value (weakest principal direction)
2/ Maximum average roll value
3/ U.S. standard sieve size
Note: CWO is a USACE reference

Table TS14L–6 NRCS specifications for nonwoven geotextiles

Property	Test method	Class I
Tensile strength (lb) 1/	ASTM D4632	180
Elongation at failure (%) 1/	ASTM D4632	>50
Puncture (lb) 1/	ASTM D4833	80 minimum
UV (% residual tensile strength)	ASTM D4355 150-hr exposure	70 minimum
Apparent opening size 2/	ASTM D4751	As specified, max. #40 3/
Permittivity s^{-1}	ASTM D4491	0.70 minimum

1/ Minimum average roll value (weakest principal direction)
2/ Maximum average roll value
3/ U.S. standard sieve size

Geotextile placement

The geotextile should be laid flat and smooth, so that it is in intimate contact with the subgrade. The geotextile must be free of tension, folds, and wrinkles. The geotextile should be placed immediately prior to ACB placement.

The joints should overlap a minimum of 18 inches in dry installations and 3 feet in below water installations. The geotextile joints should be shingled so that the upstream or upslope geotextile overlaps the adjacent downstream or downslope geotextile.

When a granular filter is used in combination with a geotextile filter, or the geotextile is placed on a silty sand or fine to medium sand subgrade, the geotextile should encapsulate the granular filter for a minimum length of 1 foot of the subgrade (fig. TS14L–17).

Placement of the ACB

The cellular concrete blocks should be placed on the geotextile or subgrade in such a manner as to produce a smooth planar surface in intimate contact with the geotextile or subgrade. No individual block within the plane of placed cellular concrete blocks should protrude more than the maximum amount of protrusion used in the design and specified for the project. If assembled and placed as large mattresses, the cellular concrete mats are placed by a crane or other approved equipment and attached to a spreader bar or other approved device (fig. TS14L–18), to aid in the lifting and placing of the mats in their proper position.

The equipment used should have adequate capacity to place the mats without bumping, dragging, tearing or otherwise damaging the underlying fabric. The mats are placed side by side or end to end, so that the mats abut each other. Mat seams, or openings between mats, that are greater than the typical separation distance between blocks should be filled with grout. Whether placed by hand (fig. TS14L–19) or in large mattresses, distinct changes in grade that result in a discontinuous revetment surface in the direction of flow should include a grout seam at the grade change location so as to produce a continuous surface.

Termination

The ends of the ACB revetment system should be buried in termination trenches. Termination (or top of slope) trenches, as shown in figure TS14L–20, and side trenches are backfilled and compacted flush with the top of the blocks. The trench may also be backfilled with properly sized riprap, concrete, or other armoring material. The transition from the slope into the trench should be rounded. The integrity of a soil trench backfill must be maintained to ensure a surface that is flush with the top surface of the cellular concrete blocks for its entire service life. Toe trenches are backfilled as shown on the contract drawings. Backfilling and compaction of trenches are completed in a timely fashion. No more than 500 lineal feet of placed cellular concrete blocks, without completed termination or toe trenches, is permitted at any time.

Anchor penetrations

Anchor penetrations through the geotextile should be filled with grout to reduce migration of the subgrade soil through the penetration point.

Filling

The open area of the ACB is filled with topsoil to support vegetative growth (fig. TS14L–21), or gravel material can be used as fill. The fill within the open area should be completed as soon as possible. Topsoil should be overfilled by 1 to 2 inches to allow consolidation of the fill material. A vegetated condition will improve the overall stability of the system by root penetration and anchorage; however, the additional stability benefit provided by vegetation is ignored for the sake of conservatism in the design procedure. Preferred vegetation through the blocks is native grasses. Woody shrubs and trees are discouraged due to the potential for root heaving on blocks. Figures TS14L–22 and TS14L–23 show the same project as in figure TS14L–21 after establishment of vegetation.

Figure TS14L–17 Granular filter encapsulation by a geotextile (*Figure courtesy of HCFCD*)

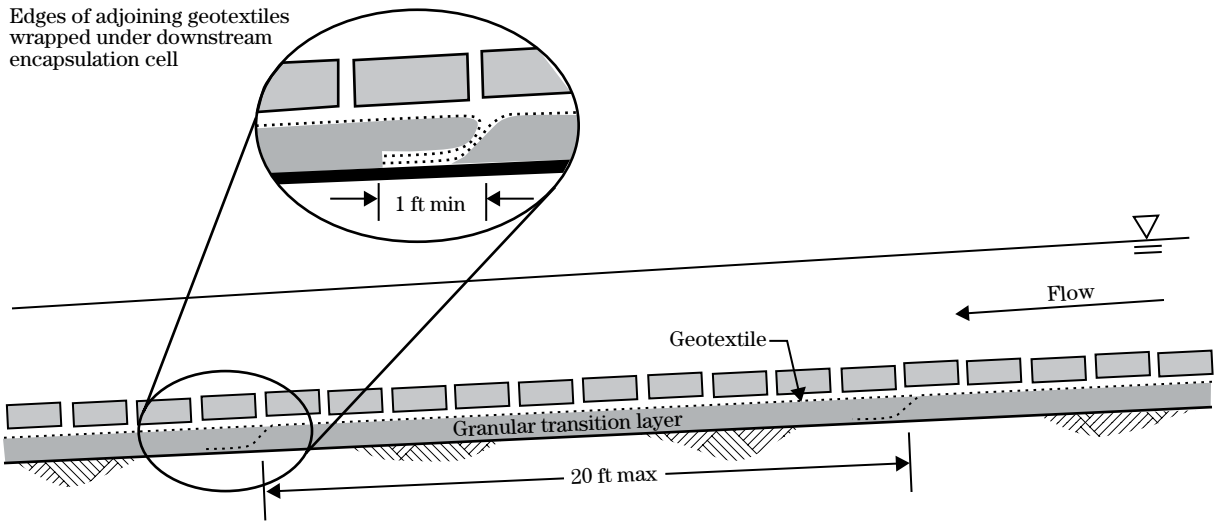


Figure TS14L–18 Spreader bar for placement of cabled mats

Figure TS14L–19 Hand placement of ACB blocks

Figure TS14L–20 ACB termination trench

Figure TS14L–21 Filling ACBs with top soil (*Photo courtesy of Joe Polulech*)

Figure TS14L–22 ACB revetment system 1 year after completion (*Photo courtesy of Joe Polulech*)

Figure TS14L–23 ACB revetment system 2 years after completion (*Photo courtesy of Joe Polulech*)

ACB example calculations

Given: An ACB revetment system is to be installed on the side slopes of a stream channel in the vicinity of a highway bridge. A hydraulic analysis has been conducted, and the following conditions are recommended for the design:

Design velocity:	11 ft/s
Design shear stress:	2 lb/ft^2
Bed slope:	0.03 ft/ft
Side slope:	2H:1V
Allowable block protrusion	1 in
Minimum factor of safety	1.5

The proposed ACB product has the following characteristics:

Weight, W	35 lb
Block width, b	1.1 ft
Block length, l	0.97 ft
Block thickness	4.75 in
Critical shear stress of block on a horizontal surface	15 lb/ft^2
Specific gravity of concrete	2.2

Determine: The factor of safety for the proposed product

Solution:

Step 1 Calculate the moment arms of the proposed block.

$$\ell_1 = \frac{1}{2}\times \text{ block thickness} = \frac{1}{2}\times\frac{4.75}{12} = 0.198 \qquad \text{(eq. TS14L–5)}$$

$$\begin{aligned}\ell_2 &= \ell_4\\ &= \sqrt{(\text{block length})^2 + (\text{block width})^2}\\ &= \sqrt{(0.97)^2 + (1.1)^2}\\ &= 0.733\end{aligned} \qquad \text{(eq. TS14L–6)}$$

$$\begin{aligned}\ell_3 &= 0.8\times\text{block thickness}\\ &= 0.8\times\frac{4.75}{12}\\ &= 0.317\end{aligned} \qquad \text{(eq. TS14L–7)}$$

Step 2 Calculate the submerged unit weight of block.

$$\begin{aligned}W_S &= W\times\left(\frac{G_c - 1}{G_c}\right)\\ &= 35\times\left(\frac{2.2-1}{2.2}\right)\\ &= 19.1 \text{ lb}\end{aligned} \qquad \text{(eq. TS14L–8)}$$

Step 3 Calculate the stability number on a horizontal surface.

$$\begin{aligned}\eta_0 &= \frac{\tau_{des}}{\tau_c}\\ &= \frac{2}{15}\\ &= 0.133\end{aligned} \qquad \text{(eq. TS14L–9)}$$

Step 4 Calculate additional lift and drag forces from block protrusion.

$$\begin{aligned}F'_L &= F'_D\\ &= 0.5\times(\Delta Z)b\rho V_{des}^2\\ &= 0.5\times\left(\frac{0.5}{12}\right)\times 1.1\times 1.94\times 11^2\\ &= 5.4 \text{ lb}\end{aligned} \qquad \text{(eq. TS14L–10)}$$

Step 5 Calculate a_θ.

$$a_\theta = \sqrt{\cos^2\theta_1 - \sin^2\theta_0} \qquad \text{(eq. TS14L–11)}$$

$$\begin{aligned}\theta_0 &= \text{ATAN}(\text{bed slope})\\ &= \text{ATAN}(0.03)\\ &= 1.72^\circ\end{aligned} \qquad \text{(eq. TS14L–12)}$$

$$\begin{aligned}\theta_1 &= \text{ATAN}\left(\frac{1}{\text{channel slope}}\right)\\ &= \text{ATAN}\left(\frac{1}{2}\right)\\ &= 26.57^\circ\end{aligned}$$

$$\begin{aligned}\alpha_\theta &= \sqrt{\cos^2(26.57) - \sin^2(1.72)}\\ &= 0.894\end{aligned}$$

Step 6 Calculate θ.

$$\theta = \arctan\left(\frac{\sin\theta_0}{\sin\theta_1} \times \frac{\cos\theta_1}{\cos\theta_0}\right) \qquad \text{(eq. TS14L–13)}$$

$$= \arctan\left(\frac{\tan\theta_o}{\tan\theta_1}\right)$$

$$= \arctan\left(\frac{\tan 1.72^\circ}{\tan 26.57^\circ}\right)$$

$$= 3.43^\circ$$

Step 7 Calculate β.

$$\beta = \arctan\left(\frac{\cos\left(\theta_0 + \theta\right)}{\left(\frac{\ell_4}{\ell_3} + 1)\right)\frac{\sqrt{1 - a_\theta^2}}{\eta_0\left(\frac{\ell_2}{\ell_1}\right)} + \sin\left(\theta_0 + \theta\right)}\right) \qquad \text{(eq. TS14L–14)}$$

$$\beta = \arctan\left(\frac{\cos\left(1.72^\circ + 3.43^\circ\right)}{\left(\frac{0.733}{0.317} + 1)\right)\frac{\sqrt{1 - 0.894^2}}{0.133\left(\frac{0.733}{0.198}\right)} + \sin\left(1.72^\circ + 3.43^\circ\right)}\right)$$

$$= 17.82^\circ \qquad \text{(eq. TS14L–15)}$$

Step 8 Calculate stability number for a sloped surface η_1.

$$\eta_1 = \left(\frac{\frac{\ell_4}{\ell_3} + \sin\left(\theta_0 + \theta + \beta\right)}{\frac{\ell_4}{\ell_3} + 1}\right)\eta_0$$

$$= \left(\frac{\frac{0.733}{0.317} + \sin\left(1.72^\circ + 3.43^\circ + 17.82^\circ\right)}{\frac{0.733}{0.317} + 1}\right)0.133$$

$$= 0.109 \qquad \text{(eq. TS14L–16)}$$

Step 9 Calculate angle between drag force and block motion, δ.

$$\delta + \beta + \theta = 90^\circ \qquad \text{(eq. TS14L–17)}$$

$$\delta = 90^\circ - \beta - \theta$$

$$\text{So,} \quad = 90^\circ - 17.82^\circ - 3.43^\circ \qquad \text{(eq. TS14L–18)}$$

$$= 68.75^\circ$$

Step 10 Calculate the factor of safety for the proposed block, SF. (See equations for step 10 in box at the bottom of page.)

Solution: FS = 1.63 > 1.5
Factor of safety is acceptable

Step 10 calculations:

$$FS = \frac{\left(\frac{\ell_2}{\ell_1}\right)a_\theta}{\sqrt{1 - a_\theta^2}\cos\beta + \eta_1\left(\frac{\ell_2}{\ell_1}\right) + \frac{\left(\ell_3 F_D' \cos\delta + \ell_4 F_L'\right)}{\ell_1 w_S}} \qquad \text{(eq. TS14L–19)}$$

$$FS = \frac{\left(\frac{0.733}{0.198}\right)0.894}{\sqrt{1 - 0.894^2}\cos 17.82^\circ + 0.109\left(\frac{0.733}{0.198}\right) + \frac{\left(0.317 \times 5.379 \times \cos 68.75^\circ + 0.733 \times 5.379\right)}{0.198 \times 1.9}}$$

If the critical shear stress is determined from an ACB hydraulic test with system restraints (such as mechanical anchors) or ancillary components (such as a synthetic or granular drainage medium), the restraints or components should be incorporated into the installation.

Conclusion

ACB revetment systems provide a viable product for armoring countermeasures to be used in stream restoration and stabilization, particularly in open channels that have high velocities and shear stresses and in applications where the operational boundaries are fixed or limited and no further erosion can be tolerated. An ACB revetment system is also useful in arresting lateral stream migration and local vertical instability. Its use has distinct advantages, not only in terms of accepted design techniques, but also in established contracting and construction procedures.

The blocks must be tested in accordance with the procedures identified in this technical supplement and the associated references. Design should follow the design procedures as shown here. ACBs should be considered as a system and include all the restraints and components in the hydraulic stability testing. The use of a properly designed geotextile or granular filter is critical to the successful performance of the ACB revetment system. As with all armoring countermeasures, proper subgrade preparation, placement of geotextile or granular filter, and block installation are also essential to the proper functioning and performance of the system during the design event.

The decision to use an ACB revetment system for stabilization must include considerations for costs, performance requirements, maintenance, aesthetic characteristics, ecological habitat and functions, upstream and downstream effects, and the dynamics of fluvial geomorphology of the system.

As described, some ACB systems provide the flexibility of including grass in topsoil-filled block openings to provide additional erosion control. Since the use of woody vegetation is discouraged because of its potential damage to the block installation and maintenance costs, the prospect of reestablishing a fully functioning riparian zone is minimal. Where connection of people back to the stream is an important consideration, however, ACBs can provide a foundation for grassed greenways to be established along stabilized channels.

Technical Supplement 14M

Vegetated Rock Walls

Issued August 2007

Cover photo: Rock walls combine economy of design with physical stabilization and ecological function.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14M

Vegetated Rock Walls

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Technical Supplement 14M

Vegetated Rock Walls

Purpose

A vegetated rock wall is in the category of mixed-construction bioengineering. Both structural, mechanical, and vegetative elements work together to prevent surface erosion and shallow mass movement by stabilizing and protecting the toe of steep slopes. These types of treatments can provide edge habitat and reduce the need to grade the banks. These walls differ from conventional retaining structures because they are placed against relatively undisturbed earth and are not designed to resist large earth pressures. They are most applicable in high energy streams with narrow riparian corridors.

Introduction

Vegetated rock walls are considered toe walls that are normally 3 to 5 feet high, with 2 to 3 feet below grade for its footing. Rock used to construct the wall should normally range from 8 inches to 3 feet in diameter. Smaller stones may be used to chink or fill the gaps between the larger stone. Usually the stones are dry stacked.

Rectangular shaped rock is often used because it can be stacked better than rounded stone. Larger stones should be used for the base. The foundation for the walls should be firm, undisturbed or well-tamped soil. The wall should be constructed with a 6V:1H external batter angle (fig. TS14M–1).

Figure TS14M–1 Dry stacked stone wall

A sloping bench can be provided behind the wall to serve as a transition slope on which vegetation can be planted. Well tamped backfill should be placed behind the wall and in the spaces between the rocks as they are placed. Live branch cuttings can be placed in the interstices of the rock wall as it is constructed. The butt ends of the branches should extend into the backfill behind the wall. A cross section of a vegetated rock wall is shown in figure TS14M–2.

Construction expertise levels required

The construction of vegetated rock walls is more labor-skill intensive than energy-capital intensive. Well-supervised, skilled labor can be substituted for higher cost, energy-intensive materials and receive excellent results.

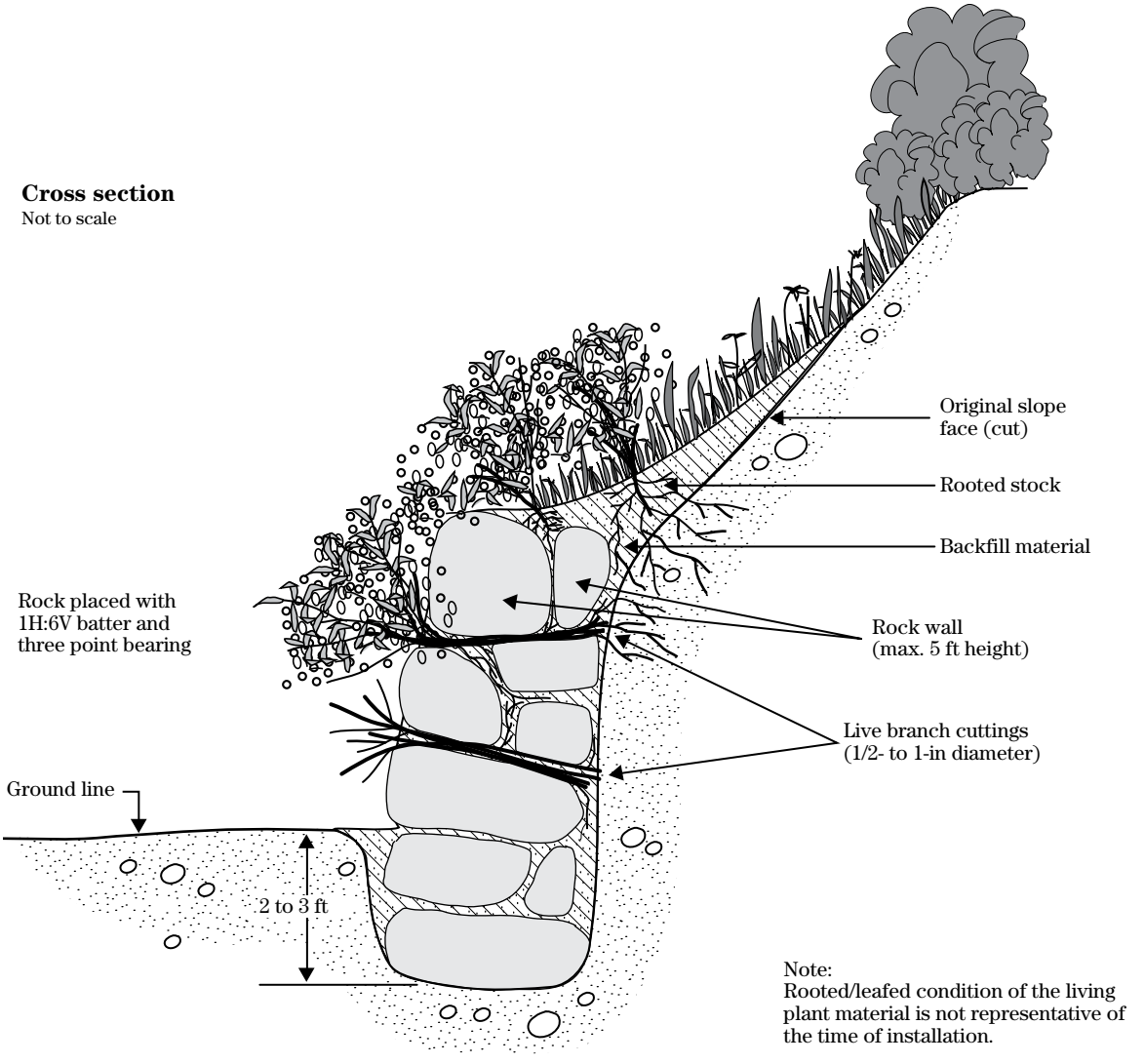
Applications and effectiveness

This system is appropriate at the base of a slope where a low wall may be required to stabilize the toe of the slope and reduce its steepness. It is useful where space is limited and natural rock is available.

Analyzing existing slope conditions

Vegetated rock walls are not intended to resist large lateral earth pressures. Their purpose is to stabilize and protect the toe of steep slopes. These walls will not solve the problem of slope instability that is based on the degree of inclination of the slope, the presence of ground water seepage, or the presence of deep-seated, lower strength soils. A complete visual reconnaissance should be made of any slope that is being considered for repair, using toe-wall construction. If a condition other than toe erosion/scour has taken place on a particular slope, such as slope erosion or slope failure, those conditions must be addressed, as well as the toe erosion/scour. In addition to the vegetated rock wall, a combination of scaling, contour-wattling, and brush-layering could be required to stabilize the

Figure TS14M–2 Vegetated rock wall details



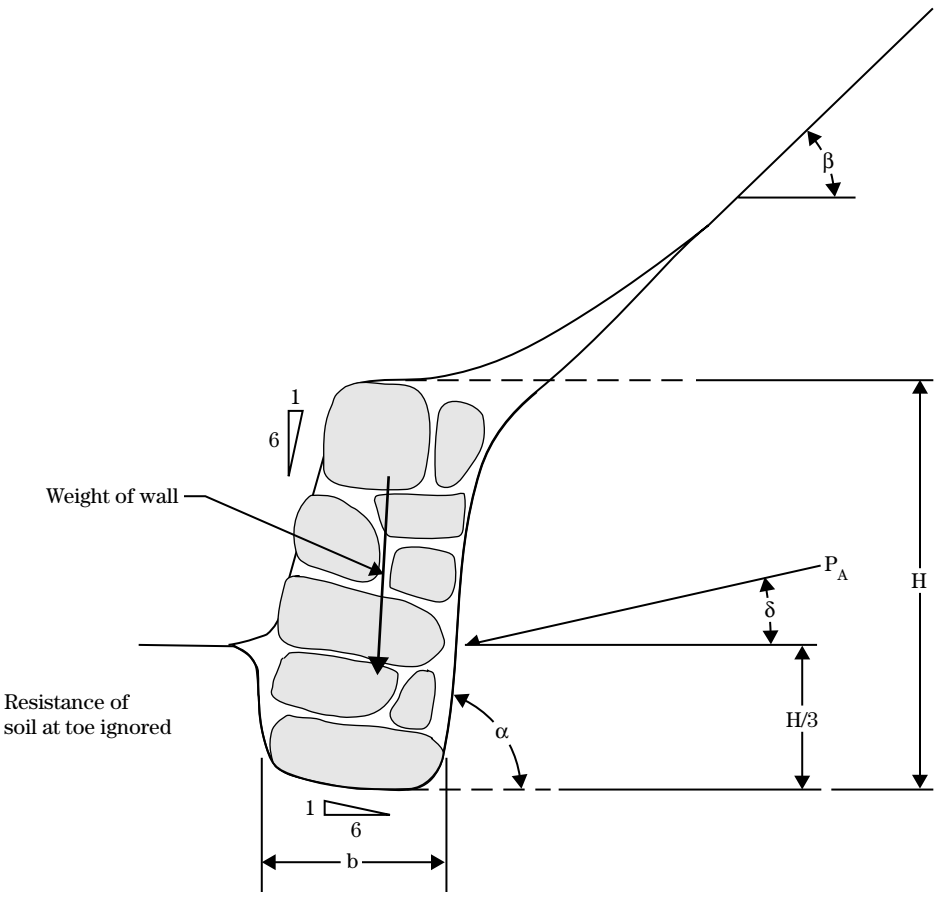
slope (increasing the factor of safety for slope stability). A visual reconnaissance should be followed up by a mechanical analysis of the slope to address its global stability related to slope failure.

Design procedure for establishing width and height

The vegetated rock wall should be analyzed as a massive gravity wall, which requires computations to determine the external, as well as internal, stability of the wall. Internal and external stability calculations of the vegetated rock walls should be performed to determine the stability of the wall against overturning, sliding along the base, sliding along the bedding planes between rock layers, tension stresses at rock layers and the foundation, and bearing capacity failure. These computations include determining the weight of the wall and the lateral force exerted by the soil retained behind the wall. It is generally assumed that hydrostatic pressures will not build up behind the wall. This is a fairly safe assumption, since the walls are generally permeable enough to allow water to drain.

The forces acting on the rock wall are shown in figure TS14M–3. The lateral force acting on the wall can be calculated using Coulomb's equation for lateral earth pressure coefficient, K_A (Coulomb 1776) (eq. TS14M–1):

Figure TS14M–3 Forces acting on vegetated rock wall



$$K_A = \frac{\sin^2(\alpha+\phi)\cos\delta}{\sin\alpha\sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

(eq. TS14M–1)

where:

β = angle of inclination of slope
α = batter angle of wall
ϕ = angle of internal friction of slope soil
δ = angle of wall friction, usually taken as 2/3 of ϕ

Some values for K_A have been tabulated in table TS14M–1. The force is then calculated by equation TS14M–2:

$$P_A = \frac{1}{2}\gamma H^2 K_A \qquad \text{(TS14M–2)}$$

where:

P_A = active earth force per unit length of wall
γ = unit weight of retained soil
H = height of the wall

The resultant of the combined weight of the wall and the lateral earth force must pass through the middle third of the base as shown in figure TS14M–4. The factor of safety against overturning should be greater than or equal to 2.0:

$$\bar{x} = \frac{\sum M_R - \sum M_O}{\text{Wall weight}} \qquad \text{(TS14M–3)}$$

with $\frac{b}{3} \le \bar{x} \le \frac{2b}{3}$

$$\frac{\sum MR}{\sum MO} \ge 2.0 \qquad \text{(TS14M–4)}$$

where:

ΣM_O = overturning moment due to horizontal component of lateral earth force about the toe of the wall
ΣM_R = resisting moment due to the weight of the wall
$\bar{x}$ = distance from toe to point of application of resultant base
b = width of wall

Sliding at the base-to-foundation interface and between the rock layers of the wall should be investigated. It can be assumed that the friction angle between the rock layers is at least equal to the friction angle of the soil. If this is not the case, the sliding stability of the wall must be checked based on the smaller value of the rock-to-rock interface. A factor of safety against sliding should be greater than or equal to 1.5.

$$\frac{\Sigma F_v \tan\phi}{\Sigma F_H} \ge 1.5 \qquad \text{(TS14M–5)}$$

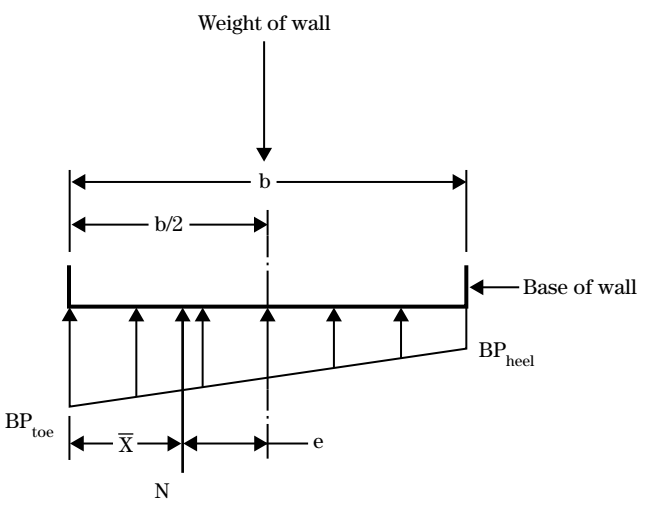
where:

F_v = weight of the wall (vertical component of lateral earth force ignored)
F_H = summation of horizontal forces against the wall

Table TS14M–1 Coefficient of active earth pressure as a function of internal friction angle and inclination of slope

β =			0 (deg)	10 (deg)	20 (deg)	30 (deg)
ϕ = 20	α = 90	δ = 13.33	0.43	0.51	0.88	–
ϕ = 30	α = 90	δ = 20	0.28	0.32	0.39	0.75
ϕ = 40	α = 90	δ = 26.7	0.18	0.20	0.23	0.28

Figure TS14M–4 Bearing pressure distribution at base of wall



Additional resistance to sliding of the wall along its base is provided by embedding the base below the ground line. However, this is ignored due to the fact that the soil in this area could be removed, and this additional resistance would be eliminated.

The bearing pressure at the base of the wall, due to overturning forces and the weight of the wall, should be less than the allowable value for the soil on which the wall bears. Building codes frequently provide allowable bearing pressures for different types of soils, which can be used for the design of vegetated rock walls. The distribution of the bearing stress beneath the wall, which varies from a maximum at the toe to a minimum at the heel of the wall, can be computed using equation TS14M–6:

$$BP = \frac{\sum F_v}{b}\left(1 \pm \frac{6e}{b}\right) \qquad \text{(TS14M–6)}$$

where:
b = width of wall at base

$$e = \frac{b}{2} - \bar{x} \qquad \text{(TS14M–7)}$$

Use the plus (+) sign in equation TS14M–6 to calculate the base pressure at the toe of the wall and the negative (–) sign to calculate the pressure at the heel. The pressure at the toe should be less than the allowable bearing pressure. The pressure at the heel should be less than the allowable bearing pressure, and greater than or equal to zero. A negative pressure at the heel would indicate that the wall is lifting away from the soil at the heel. It could also mean that the wall is lifting at joints between layers of rocks—a potentially dangerous situation. A sudden failure of the wall could happen, since these walls are not reinforced. The width of the wall would need to be increased or the height reduced, if equation TS14M–6 indicates a negative stress or exceeds the allowable bearing pressure. A tension crack that opens will continue to get wider until compressive stresses balance out the tensile stresses.

Finally, it is important that the base of the wall extend below the lowest potential scour elevation. This important issue is not addressed in this technical supplement, but more information on calculating ultimate scour depths is provided in NEH654 TS14B.

Construction guidelines

Figure TS14M–2 shows a typical profile of a vegetated rock wall. Live cuttings should have a diameter of 1/2 to 1 inch and be long enough to reach beyond the rock structure into the fill or undisturbed soil behind the wall. Suggested construction guidelines are:

- Starting at the lowest point of the slope, remove loose soil until a stable base is reached. This usually occurs 2 to 3 feet below ground elevation, but ultimate scour depth should be checked. Excavate the back of the stable foundation (closest to the slope) slightly deeper than the front to add stability to the structure.
- Excavate the minimum amount from the existing slope to provide a suitable recess for the wall.
- Provide a well-drained base in locations subject to deep frost penetration.
- The vegetated rock wall shall be constructed so that the external wall face has a 6V:1H batter. The rocks should have a slight rearward pitch.
- The rocks should be placed with at least a three-point bearing on the foundation material or underlying rock course. The rock-to-rock contact is maximized.
- The rock should be rectangular or nearly so at the rock-to-rock contact. If not perfectly flat, the thicker end should be placed towards the front of the wall.
- The rock should be placed so that the center of gravity is as low as possible, with the long axis and bedding planes slanting inward toward the slope. As the rocks are placed, fill is laid behind and around the rocks and tamped thoroughly.
- As the wall is built, the layers must be placed in an overlapping pattern, closely adjacent and in a continuous manner to minimize gaps.
- When a rock wall is constructed adjacent to an impervious surface, place a drainage system at the back of the foundation and the outside toe of the wall to provide an appropriate drainage outlet.

- Overall height of the rock wall, including the footing, should not exceed 5 feet.
- A wall can be constructed with a sloping bench behind it to provide a base on which live branch cuttings can be placed during construction.
- Live branch cuttings of shrub-type species of adventitiously rooting material can be placed in the interstices of the rock wall during or after construction. The basal ends of the branches must extend into the backfill or undisturbed soil behind the wall. The live branch cuttings should be oriented perpendicular to the slope contour, with growing tips protruding slightly from the finished rock wall face (fig. TS14M–2).

Conclusion

Vegetated rock walls can provide aesthetic bank protection in confined channels. Their application tends to be in urban and suburban streams. The spaces between the stones can provide edge habitat, and the overhanging vegetation can reduce thermal loading of the streamflow.

Technical Supplement 14N

Fish Passage and Screening Design

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Restoring fish migration may require culverts and other water conveyances to be redesigned or replaced.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14N

Fish Passage and Screening Design

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Technical Supplement 14N

Fish Passage and Screening Designs

Introduction

Fish passage and screen facility design is often a significant component in stream restoration and water resource management. A wide variety of issues often arise regarding passage and screening design, depending on the project region and species of interest. This technical supplement provides an overview of fish passage and screening design approaches that incorporate biological considerations into the hydraulics of commonly used structures; guidance on site assessment and operations and maintenance (O&M) of fishways and screens; an overview of assessment and design approaches for fishways and screens, ladders, culverts, tidegates, and floodgates; and finally, an example design is presented.

Passage barrier and screening overview

Fish migration is a natural, usually seasonal life cycle function, as fish move in large numbers from one habitat type to another to spawn, feed, grow, or seek refuge from predators. Federal and state regulations often require mitigation for passage barriers and water intakes or diversions that entrain fish. As a result, the timing, duration, and frequency of fish migrations must be accounted for when planning and implementing water resource projects within a watershed.

Passage barriers are primarily a problem for fish trying to move upstream in an estuary, river, or stream. Both natural and manmade barriers occur within river and stream systems. Natural physical barriers include features such as waterfalls, cascades, and large rapids. Common manmade physical barriers include dams, diversions, culverts, weirs, and grade control and sill structures. Chemical and biological barriers also exist in many rivers across the United States, including water quality (temperature) and predation from nonnative species. Virtually all manmade barriers impede fish passage, limit natural migration patterns, regulate population dynamics, and fragment diverse habitats.

Physical fish barriers are classified by water velocity, water depth, and barrier height. The magnitude of a fish passage impediment can generally be classified as:

- partial—impassable to some species or certain age classes all or most of the time
- temporary—impassable during some times to all or most species and/or age classes (during low-flow conditions)
- complete—impassable to all fish at all times

Some situations present difficult conditions under which passage for all fish species and size classes cannot be provided 100 percent of the time. However, typical design modifications for fish passage barrier mitigation include the following:

- culvert removal, modification, or replacement
- channel modification
- structural fish passage features (concrete or metal ladders and chutes)
- natural-type fish passage design (rock riffles, rock aprons, step-pool rock ladders)
- dam or barrier removal or modification

In addition to fish barriers, fish entrainment into water diversions or pump intake structures also affects natural migration patterns—primarily for downstream movement, but sometimes for upstream movement. Typically, fish screens are used to prevent adult and juvenile fish entrainment or attraction into manmade diversion structures or other features (power or sewage treatment plant outfalls). Typical types of surface and subsurface diversion structures requiring fish screens include municipal and irrigation water intakes, irrigation diversions, and pump stations. Protecting fish from entrainment at these structures may be achieved through the use of the following features:

- physical barriers and screens
- behavioral guidance to direct swimming direction
- capture and release systems

This technical supplement provides general design guidance for three types of fish passage features: concrete fishways, step-pool rock ladders, and roughened channels (engineered channel, Denil, and Alaskan Steeppass). Additionally, approaches are described for modifying or replacing existing culverts to improve or provide fish passage. Considerations for tide gates and floodgates are also presented. Finally, screen design guidance addressing active and passive screening approaches for gravity and pumped diversions is presented.

Federal and state authority and regulations

Federal and state regulations require fish passage and protection from fish entrainment under several authorities that require water resource project developers to mitigate for impacts to fisheries resources. For example, under the Endangered Species Act (ESA) it is illegal to knowingly take a fish that is listed as threatened or endangered. The ESA and other Federal and state laws may require that a design provides passage upstream of barriers and prevents entrainment into diversion structures or pump intakes. The following sections provide an overview of passage and screening design criteria often promulgated in Federal and state regulations. Designers should contact local authorities and experts to determine if species, season, or region-specific passage and screening criteria are emphasized in the project area.

Several authorities and regulations require the implementation of fish passage and screening projects. In many parts of the United States, fish passage and screening projects are undertaken to protect ESA-listed species and state species of concern and enhance their habitat. The National Oceanic and Atmospheric Administration National Marine Fisheries Service (NOAA Fisheries Service), U.S. Fish and Wildlife Service (USFWS), and state game and fish agencies are excellent contacts for legal requirements and technical criteria. Generally, projects that are authorized, funded, or carried out by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) are subject to ESA Section 7 consultation unless they clearly have no effect on listed resources. When the project is likely to affect listed fish (even if the effects are completely beneficial), the project designer must comply with technical criteria from NOAA Fisheries Service or USFWS when designing fishways or screening facilities to expedite the consultation process and increase the likelihood of project success. However, in all projects, NRCS personnel should strive to design fishways and screens that protect all aquatic resources and provide private landowners with workable solutions. More information on permits, process, and regulatory requirements is provided in NEH654.13.

Biological design considerations

The design of a fish passage or screening project begins with identifying the current or historical distribution and migratory patterns of fish species in the project area. Target species can be those listed as threatened or endangered under the ESA and/or state species of concern, other native species, or aggressive nonnative species tagged for potential exclusion (species to be isolated at a sorting facility in a fish passage). The project design should be based on the physical limitations of the weakest species requiring passage and accommodating the smallest size within that species, wherever feasible, based on stream conditions (Washington Department of Fish and Wildlife (WDFW) 2000a). These physical limitations are usually based on biological data and characteristics and are then used to develop the design criteria for fish passage and screening structures (Bates 1992).

The following planning sequence and biological characteristics are often evaluated when developing design criteria for a fish passage or screen project:

Step 1 Identify the target species for fish passage or screening.

Step 2 Determine the migratory timing and life history stage at migration.

Step 3 Determine the physical limitations on fish passage (swimming speed, jumping ability).

Step 4 Identify the environmental attractors and stressors (flow volumes, flow velocity, water temperature, seasonal timing).

Step 5 Identify any relevant behavioral characteristics of the target species that could affect fish passage (water temperature preferences and avoidances).

Many fish species must migrate to satisfy their habitat requirements for foraging, resting, rearing, and spawning. Additionally, many resident freshwater species commonly move several miles within freshwater systems on a daily or monthly basis for feeding or sheltering purposes. Migrating or moving fish are vulnerable to injury and mortality if normal movement patterns are blocked or impeded by constructed barriers. They are also more susceptible to injury as they try to negotiate manmade barriers. If fish passage is impeded

during spawning migrations, impacts on population can be severe and include decreased egg size and abundance, decreased redd excavation success, and outright mortality (Rainey 1991).

Migration type

Migratory life history strategies vary widely and include bidirectional migrations between marine and freshwater environments (diadromy), or solely within freshwater environments (potamodromy). Most major migrations occur for reproduction (spawning) purposes (anadromous and catadromous fishes), although large-scale movements also occur seasonally as fish exploit food resources along inland rivers, estuaries, and coastlines. Fish migration categories and strategies are briefly described in tables TS14N–1 and TS14N–2.

Table TS14N–1 Examples of diadromous life histories and species

Category	Description/life history strategy	Species
Anadromous	Species that incubate and hatch in freshwater, migrate to saltwater as juveniles to grow, and return to freshwater as adults to spawn	Alewife; striped bass; Atlantic, coho, pink, chum, sockeye, and Chinook salmon; steelhead, cutthroat, and bull trout; dolly varden; sturgeon; American shad; perch; Atlantic herring; sea and Pacific lamprey
Catadromous	Species that hatch in saltwater, migrate to freshwater as juveniles to grow, and return to saltwater to spawn	American eel
Amphidromous	Species that move between fresh and saltwater during some part of life cycle, but not for breeding	Sawfish, gobies, other tropical fishes

Table TS14N–2 Examples of potamodromous life histories and species

Category	Description/life history strategy	Species
Adfluvial	Species that hatch in rivers or streams, migrate to lakes as juveniles to grow and return to rivers or streams to spawn	Bull trout, cutthroat trout, kokanee, smelt, suckers
Fluvial	Species that live in the flowing waters of rivers or streams, but migrate between rivers and tributaries for breeding, feeding, or sheltering	Bull trout, cutthroat trout, brown trout, rainbow trout, Arctic grayling, sturgeon, paddlefish, pike

Migration schedule

Migration timing is tied to species life stage (spawning), time of day (foraging or predatory migrations), changes in water temperature, increases in flow rates, or changes in flow direction (tides). In addition, migrations can occur hourly, daily, monthly, and seasonally in fish populations across the United States and Caribbean area. Providing uninterrupted passage or continuous operational screening is often very difficult, and it is likely that any given project will have short periods of inoperation or down time. However, project designers should develop fish passage and screening approaches that provide the best level of protection for sensitive life histories and important migratory periods and that have the greatest effect on population health and sustainability.

Since migration timing and frequency of movement vary among species and watersheds, knowledge of the specific behavior of the target species is necessary for development of fish passage and screening criteria. Different species or age classes may migrate at different times of the year; multiple hydrologic analyses may be needed to determine the controlling hydraulic requirements at any particular site. Movements may occur both upstream and downstream.

Generally, anadromous adult salmon and steelhead spawning migrations occur during a distinct season (fall, winter, spring, or summer). Juvenile salmon rearing in freshwater migrate first in the spring as fry and later in the summer and early fall as fingerlings or parr, searching out different habitats as they grow (WDFW 1999). The largest movement of anadromous salmonids occurs in the spring, as juveniles transition (smolt) from their freshwater rearing areas into the productive ocean environment where they will grow to adulthood. Anadromous fish on the Atlantic coast primarily make spawning runs in the spring, and adults reproduce in estuarine or freshwater stream habitats. Conversely, catadromous American eels of the eastern coast of the United States live in freshwater streams and lakes for up to 5 years before they journey to the Sargasso Sea of the Atlantic Ocean near Bermuda to spawn. Juvenile eels spend about a year in the ocean before returning to freshwater where they grow to adulthood.

Freshwater migrations occur for spawning and foraging purposes. A study of warm-water fish in Arkansas shows bidirectional movement in streams and is not influenced by season (Warren and Pardew 1998). Redhorses, carpsuckers, catfish, muskellunge, walleye, and northern pike migrate along the Fox River in Illinois virtually year-round for foraging purposes, but only between May and July for spawning (Illinois Department of Natural Resources 2000). Young (1994) found that brown trout in south-central Wyoming moved more than 60 miles during the spawning season between mainstem rivers and adjoining tributaries. Studies by Young (1996) and Colyer et al. (2005) suggest that salmonids often undertake lengthy daily and seasonal migrations to exploit feeding areas, seek refuge or resting cover, and colonize new habitats. In addition to longitudinal (main channel) migrations, movements may occur laterally between the main channel and side channels, emergent wetlands, or backwaters. For example, some species such as northern pike spawn in side channels, oxbows, and flood plain ponds adjacent to large river systems.

Physical characteristics and capabilities

In any given stream system, juveniles and/or adults may be present during different times of the year and most likely have different swimming abilities and passage requirements. Consequently, design of fish passages and screens should incorporate available information on the specific physical capabilities of target species. These physical characteristics and capabilities vary depending on the species and life stages present, but will likely include fish body type and size, swimming ability, impact resistance, and leaping ability.

Generally, physical characteristics and swimming capabilities become the biological basis for engineering design criteria in a fish passage or screening project. For example, fish passage features designed for salmonids consider the swimming capabilities of migrating adult fish headed for spawning areas. Swim speed (burst and sustained) and distance, minimum swim depths, maximum jump/drop height, and pool approach depths are critical in providing upstream navigation for spawners. For fish screens, downstream migration of juvenile salmonids focuses more on body size, sweeping velocities, orientation to flow, and cross-sectional streamflow patterns to prevent unwanted entrainment and impingement on the structure.

Warm-water and cold-water species

The physical form and capabilities of a given species are products of evolutionary and behavioral adaptations to its physical and biological environment. The most basic distinction between fish species is their adaptation to water temperatures, and swimming performance can be directly affected by thermal conditions. Most fish are ectotherms, meaning that their body temperature is mainly regulated by their external environment. Consequently, water temperature is one of the most important physical factors affecting the behavior, physiology, and distribution of fish (Great Lakes Information Network 2004). Fish are often classified as either cold-water or warm-water species. Cold-water fish such as trout and salmon generally require temperatures below 70 degrees Fahrenheit, while warm-water species like bass and catfish thrive in temperatures primarily above 70 degrees Fahrenheit.

Body type and size

Body shapes and size of fish and aquatic organisms at maturity are often adapted to the flow regimes and general physical attributes of their respective habitats (Food and Agriculture Organization of the United Nations 2002). For example, fish in fast-flowing reaches of higher gradient streams often have torpedo-shaped bodies that offer lower flow resistance (steelhead, rainbow trout). Conversely, high-backed fish (carp and razorback suckers) colonize rivers with more gentle currents or deeper average depths (fig. TS14N–1 (Schua and Schua 1970)). Body size at a given age is especially important in screen design. Some fish species are very small shortly after hatching and are more susceptible to entrainment into surface diversions or pumping stations. Likewise, the adults of many species of fish never grow to more than 4 to 6 inches and are similarly in danger of being entrained into pumps or canals. Small-bodied or weak-swimming fish are susceptible to being impinged on fish screens where they will eventually die or fall victim to predators.

Swimming and leaping capabilities

The swimming speeds and jumping capabilities of a fish are adaptations to stream morphology, flow characteristics, and migratory life history. The swimming and jumping characteristics of a fish are defined as:

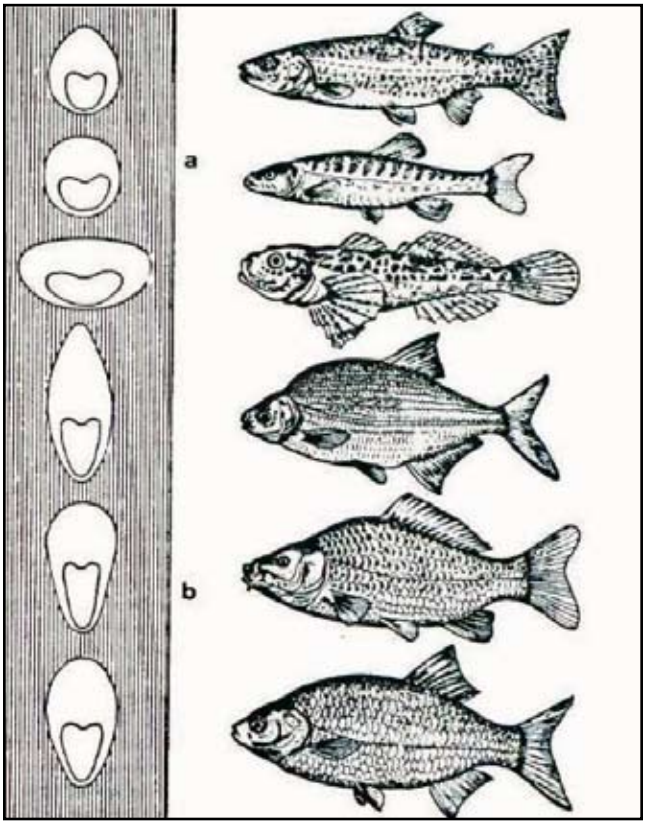
Burst (darting) speed—highest swimming speeds; endurance less than 20 seconds; ends in extreme fatigue

Sustained speed—low swimming speeds; maintained for extended time periods with little to no fatigue

Cruising speed—intermediate swimming speeds; endurance 20 seconds to 200 minutes; ends in fatigue

Jumping height—a function of swimming speed and water depth, jumping height is the maximum height obtained by a specific species and age of fish. Older and larger fish have greater maximum jumping heights, although some species have no jumping abilities at any age.

Figure TS14N–1 Fish body types



The swimming speeds and maximum jumping heights have been researched in controlled settings and documented for many fish species, with particular emphasis on salmonids. Table TS14N–3 lists the known maximum swimming speeds and maximum jumping heights for adult salmonid species (Bjornn and Reiser 1991).

Salmonids are strong swimmers and leapers (Tillinger and Stein 1996) (table TS14N–4 ((modified from Bell 1990)) especially in comparison to warm-water species and other migratory species. However, although salmon and steelhead are famous for their swimming and leaping abilities, their physical prowess steadily weakens as they swim further and further into freshwater habitats. Many fish species cannot or will not jump over obstructions; shad and herring can be blocked by a structure only 1 foot high (USFWS 2004). Likewise, although chum and pink salmon are powerful swimmers, their leaping abilities are somewhat limited, and few individuals will attempt to negotiate vertical leaps much greater than 1 foot (Orsborn 1985). All of these factors should be considered when designing fishways, road crossings, or roughened channels.

Fish size and stage of development also affect swimming capabilities. Juvenile and smaller fish do not swim as strongly as healthy adults of the same species (table TS14N–5 (modified from Bell 1990)), so slower velocities should be considered in the design of fish passage and screening projects (Tillinger and Stein 1996). Projects in settings with a variety of fish species of differing body sizes and swimming capabilities can pose especially challenging design requirements. However, if the fishway or screen passes or protects the smallest or weakest swimming fish, it is likely that other fish seeking passage at the same time or when streamflow is higher will find adequate passage conditions or be protected from entrainment into diversions or pumps.

Behavioral responses

Understanding the behavioral response of a species to stimuli enables the development of fish attractors and detractors for fish passage and screening projects. Attractors and detractors may take the form of shade, light, fishway water velocity, relative volume of fishway attraction flow to streamflow, temperature, sound, and shoreline or overhead movements. Fishway composition can be a very important factor determining success or failure. For example, some fish (shad) are hesitant to swim through a submerged orifice, instead preferring flow that is directed through a vertical slot or over a weir. Excessive turbulence at a fishway entrance may confuse or restrict target species, and the orientation of a fish ladder's entrance to the adjacent stream channel is of the utmost importance. Many fish move up a river system by capitalizing on lower velocities along the bankline boundary layer. Conversely, juvenile emigrants are usually found moving downstream in the fastest flowing portion of the channel, within 1 foot of the surface.

Table TS14N–3 Example of maximum swimming speeds and maximum jumping heights for selected adult salmonids

Salmonid species	Sustained speed		Cruising speed		Burst speed		Maximum jump height	
	ft/s	m/s	ft/s	m/s	ft/s	m/s	ft	m
Steelhead	4.6	1.40	13.7	4.18	26.5	8.08	11.2	3.4
Chinook	3.4	1.04	10.8	3.29	22.4	6.83	7.8	2.4
Coho	3.4	1.04	10.6	3.23	21.5	6.55	7.2	2.2
Cutthroat	2.0	0.61	6.4	1.95	13.5	4.11	2.8	0.9
Chum	1.6	0.49	5.2	1.58	10.6	3.23	1.7	0.5
Sockeye	3.2	0.98	10.2	3.11	20.6	6.28	6.9	2.1

Table TS14N–4 Relative swimming speeds of average-sized adult fish

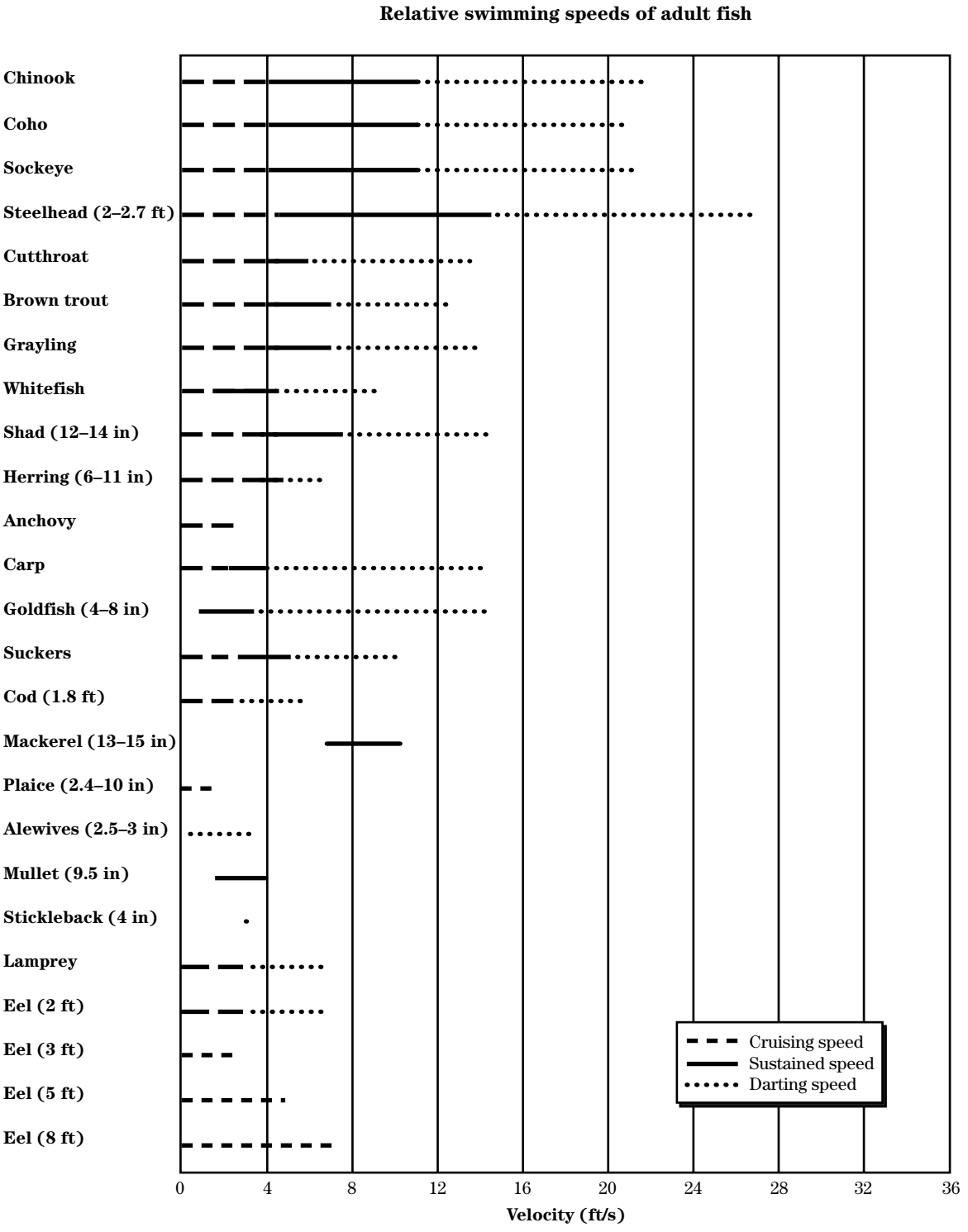
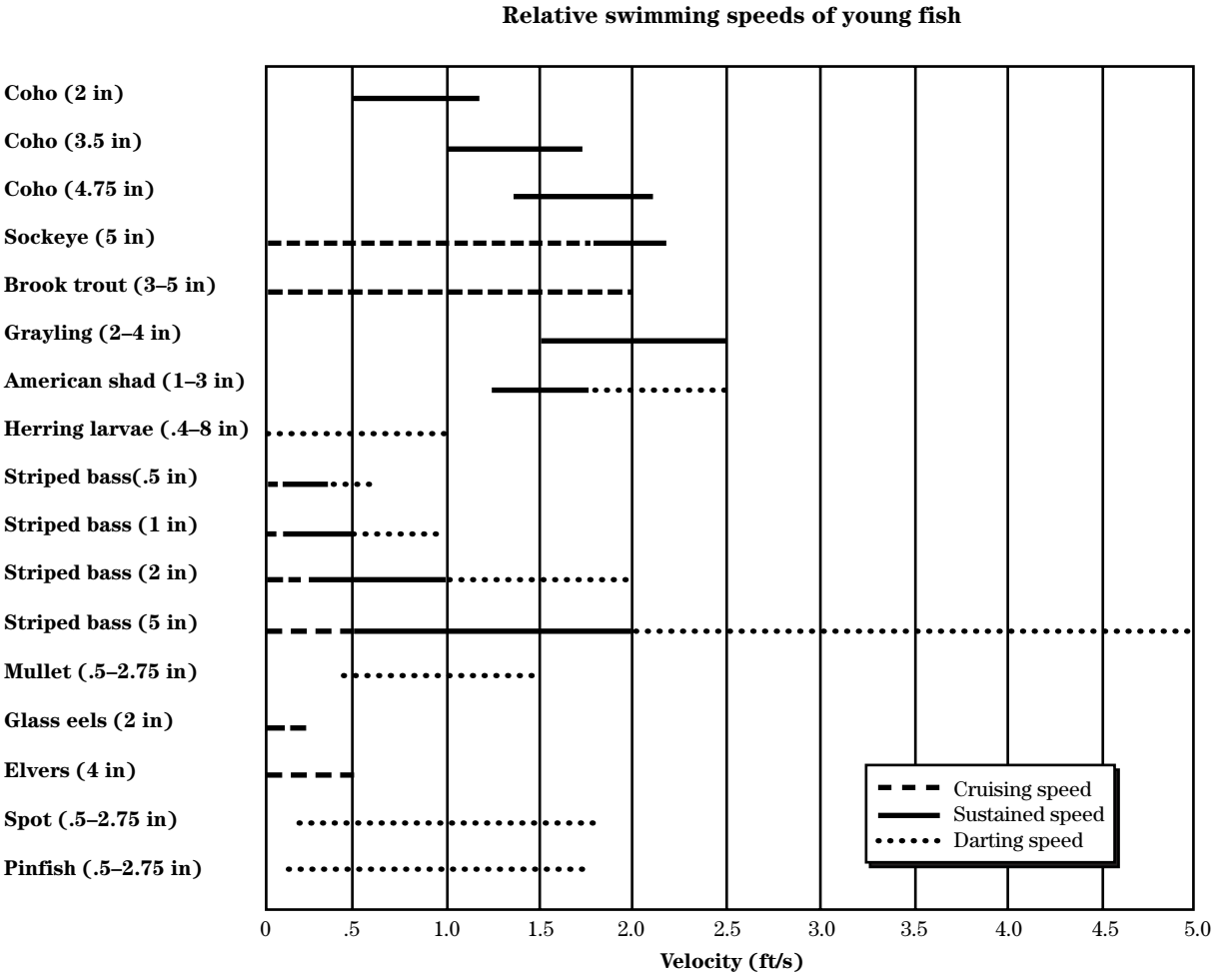


Table TS14N–5 Relative swimming speeds of young fish



Ambient environmental conditions also greatly affect the migratory habits of fish. For many species, spawning migrations may be triggered by changes in water temperature. For example, Lower Columbia River white sturgeon spawn when water temperatures are between 48 degrees and 63 degrees Fahrenheit (Wydoski and Whitney 2003) and may be delayed or prevented when water temperatures are unsuitable (Fresh et al. 1999). Light can be used as an artificial guidance stimulus, repelling fish at higher intensities and attracting them at lower intensities (Bell 1990).

Life cycle histories and physical characteristics information

Further information on various aspects of life cycle history and physical characteristics for a variety of fish may be found at the following Web sites:

http://www.nwrc.usgs.gov/publications/specindex.htm

http://www.fishbase.org

Incorporating biology into design

Much is known about the physical capabilities and behavioral tendencies of many fish species. The design of fishways and screens should incorporate these physical characteristics and capabilities of targeted species. Swimming and leaping information for many fish species is not available. Designers should use recorded data from similar species with comparable swimming and behavioral characteristics.

Fish passage and screening facilities should not impose artificial conditions that exceed the natural locomotive abilities of fish or adversely affect their behavioral response to a given stimulus. The following section provides a few useful rules of thumb regarding biological requirements and capabilities, hydraulics, and fish passage and screen design. Additional qualitative and quantitative criteria are described later in this technical supplement for fishways and screens.

Velocity

Velocities within a fish passage structure should be less than the sustained swimming capability for each species in long uniform sections and less than burst swimming ability over short distances (Katopodis 1991). Fish that are forced to swim through a structure with bursts or sustained cruising speeds will suffer stress from fatigue. If adult or juvenile migratory fish are unduly fatigued by a fishway, their ability to survive and complete life history requirements may be significantly diminished. Resting alcoves or cover must be provided if velocities within a fish passage structure exceed the swimming capabilities of the target species for long distances. Velocity breaks and shadows using boulders or large wood can be used to provide resting areas in roughened channels or fishways that are designed to mimic natural stream conditions. Species' velocity criteria would then be applied to flow areas between constructed resting areas.

For adult salmonid passage through a culvert, NOAA Fisheries Service (2000) recommends average calculated velocities of 6.0 feet per second for distances of less than 60 feet, 4.0 feet per second for distances between 100 and 200 feet, and 2.0 feet per second for distances greater than 300 feet. Recent studies found that warm-water fish passage through culverts less than 30 feet in length was reduced substantially at velocities over 1.3 feet per second (Warren and Pardew 1998). Conversely, salmonids are expected to sustain this velocity for more than 300 feet. Knowledge of the swimming abilities of target species is a vital element of the design process.

Fish screen designs must account for approach velocities in the forebay of the structure and sweep velocities along the face of the screen. Approach velocity is velocity perpendicular to the screen that may trap or impinge a fish against a screen. Physical contact with a screen face causes various injuries, and studies of fish biomechanics have been used to set hydraulic criteria for approach velocities (Pearce and Lee 1991). Sweep velocity is the velocity parallel to the face of the screen that sweeps fish along its face and into a bypass that will take them back to a river or other water body. For juvenile salmonids, NOAA Fisheries Service (2000) and WDFW (2000a) recommend an approach velocity of 0.4 feet per second in rivers and streams, and 0.33 feet per second in lakes and reservoirs. Sweeping ve-

locity should always be greater than approach velocity, regardless of screen location in a river, lake, or other body of water.

Depth

Minimum low-flow depths within fishways should be maintained to accommodate fish size, swimming abilities, and behavioral responses. For pool style fishways or channel-spanning structures, WDFW (2000a) recommends a maximum head differential of 12 inches for most adult salmonids, 6 inches for juvenile salmonids, and 3 inches for grayling. These depths are difficult to attain in many culvert crossings on small headwater streams at baseflow, so culvert size, shape, composition, and installation techniques become important factors that regulate passage.

Minimum operating depth at screening facilities depends on the type of screen and site hydraulics. However, a good rule of thumb to protect juvenile or small-bodied fish is to provide a minimum of 2.5 square feet of submerged screen for every cubic foot per second of flow diverted through it.

Jump height and pool length

The jumping heights of target species must be identified when designing a fish passage structure using stepped pools or weirs. These typically include a maximum vertical height, and the jump pool length and depth needed to allow the fish to generate enough speed to clear the barrier. In addition, pool spacing and configuration must satisfy resting requirements of all target species. The WDFW (2003) has developed the following recommendations for salmonid passage structures based on the species' swimming and leaping capabilities:

- *Entrance jump* (maximum vertical height) into a fish passage project should be no greater than 1 foot for salmon and steelhead adults and 6 inches for adult trout, kokanee salmon, and steelhead juveniles. These jump heights should also be considered as maxima when a series of jumps and pools are required.
- *Jump pool* (where entrance jumps are planned) must be at least 1.5 times the jump height or at least 2 feet deep to account for resting requirements of salmonid species.

Traditionally, fish passage projects have been designed based primarily on the capabilities of jumping species (salmonids) and only recently have nonjumping fish been considered (Peake et al. 1997). Chute ladders or roughened channels (rapid/pools or riffle/pools) without discrete drops can provide adequate fish passage for nonjumping target species.

Behavioral attractors

Many fish passage structures use high velocity attraction flow at or near their entrances. This practice is based on behaviors observed in salmonids. Migratory salmon and steelhead tend to assume upstream migration paths by "cueing-in" on higher velocity currents. A fishway entrance can be designed as a constriction to increase velocities compared to surrounding flow conditions, guiding fish into the structure based on their natural behaviors in finding upstream migration paths. When gravity flow through a passage structure decreases, auxiliary pumps may be required to supply high velocity attraction flow near fishway entrances. Alternative behavioral attractors, including entrance size, light, and acoustics are being explored in many areas of the Pacific Northwest. For example, recent studies show that salmonid species will select smaller, well-lit entrances over larger, darker ones (Oregon Department of Fish and Wildlife 2004).

Predation

Predation is a common problem at many fish passage and fish screen facilities. Avian predation most often occurs along screening structures, where birds can either land or wade near the fish screen, or dive underwater and prey on disoriented fish at the bypass pipe exit. Birds are also known to prey on groups of fish stacked up at the entrance to a fishway. Piscine or mammalian (seals and river otters) predation occurs wherever fish are in pools near the entrance to a fishway, along the face of screens, or bypass outlets downstream of a fish screen. Fish behavioral characteristics must be incorporated into screen and fish passage designs so that pooling and holding areas for predators are not adjacent to critical areas such

as fishway entrances or bypass outlets. For example, fishway entrances or bypass pipes can be located in areas where site morphology and hydraulics discourage target species to rest due to higher velocities, inadequate cover, or unsuitable depth. Avian predation can be reduced by providing overhead cover or vegetation above the entrance to a fishway or outlet from a fish screen. Designers should take great care to minimize and mitigate avian, piscine (predatory fish), and mammalian predation in any sector of a fish screen or passage project where the target species is likely to congregate for any period of time. Care should also be taken to avoid excessive fatigue or disorientation of target species as they transit a passage or screen project because physical impairment can lead to higher predation rates.

Several resources are available to designers regarding fish passage and screen history, design, and research. Examples of fish passage projects and design criteria can be found at the following Web sites:

http://wdfw.wa.gov/hab/engineer/habeng.htm

http://www.fws.gov/fisheries/fwma/fishpassage/

http://www.nwfsc.noaa.gov/research/divisions/fed/fishpassage.cfm

http://swr.nmfs.noaa.gov/hcd/expert.htm

Examples of fish screen projects and design criteria can be found at the following Web sites:

http://wdfw.wa.gov/hab/engineer/habeng.htm#dwnstrm

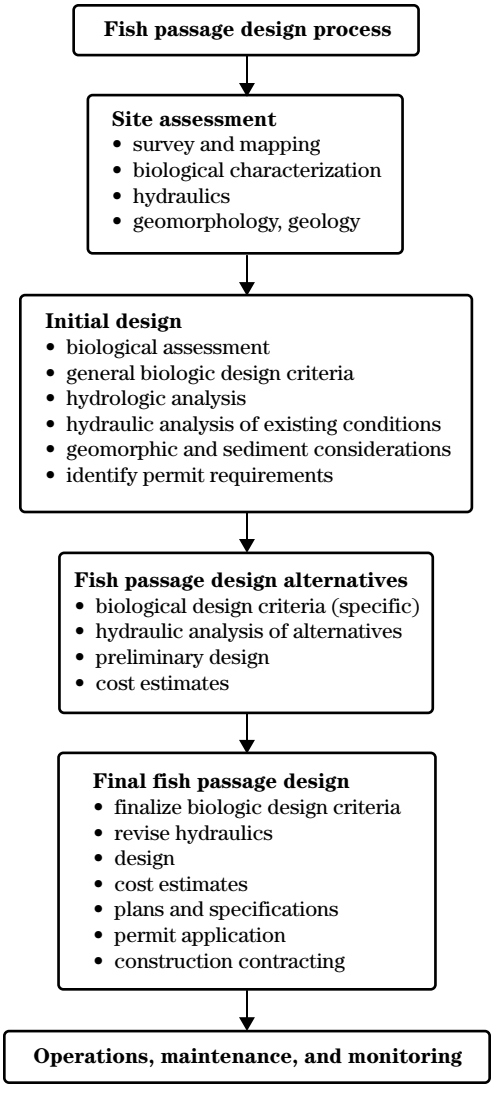
http://swr.nmfs.noaa.gov/hcd/fishscrn.htm

http://swr.nmfs.noaa.gov/hcd/pumpcrit.pdf

Fish passage and screening design

As described in previous sections, the fish passage and screening design process often begins by collecting all available information for the species of interest pertaining to migration patterns, life history requirements, and swimming and leaping capabilities. An assessment of physical conditions and site suitability usually occurs concurrent with an evaluation of relevant biological factors for the target species. Fish passage design typically includes a site assessment including site survey, geologic and geomorphic characterization, hydrologic and hydraulic analyses, and structural design (fig. TS14N–2).

Figure TS14N–2 Fish passage design process



Site assessment

Site assessments are addressed in detail in NEH654.03. This section focuses on site assessments that are more specific to fish passage issues. The site assessment should include topographic and hydrographic surveys of the passage barrier and stream channel upstream and downstream of the barrier. Whenever possible, collect historic photos of the site, and interview nearby residents for their perspective on the area. Accurate contour and infrastructure (dams, diversions) as-builts are essential for developing plans. Geomorphic assessment is necessary to characterize stream behavior and substrate conditions. In addition, depending on site conditions and proposed structures, geologic and geotechnical consultation may be necessary to characterize the soils, foundation, and river alluvium composition. Fish habitat characterization should also be performed to evaluate migration patterns, holding pool areas, and environmental conditions that will affect fish migration and use of the fish passage feature. The information developed in the site assessment is the foundation for developing topographic, geomorphic, edaphic, and biological criteria in final engineering designs.

Hydrologic analysis overview

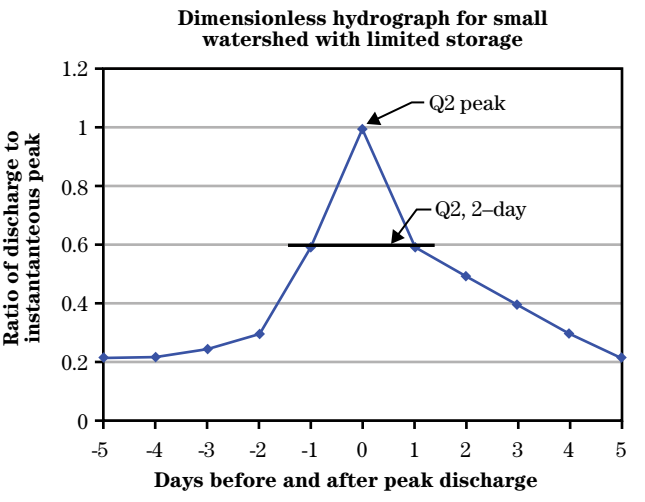
The first step in the engineering design is the hydrologic analysis. Typically, designs require defining the range of high and low discharges the fish passage facility will operate within. Hydrologic analysis must consider the period of interest when migration occurs and when statistical analyses for streams with gage data typically include flood frequency and flow-duration investigations. Hydrologic information for ungaged streams may be based on regional regression equations, correlation analyses to similar, adjacent gaged streams, or runoff modeling. However, synthesizing streamflow data should only be undertaken in smaller watersheds of about 50 square miles or less.

Hydrologic analyses are used to describe streamflow timing, magnitude, frequency, and duration during the migration period of interest. Ultimately, this information will identify the operating conditions under which the fish passage or screening facility will function.

Regional guidelines from local fish and wildlife agencies provide suggestions regarding fish passage and typically identify design discharge analysis methods. For instance, NOAA Fisheries Service (2000) recommends that for streams where streamflow data are available, the high fish passage design flow for adult salmonids should be the 1 percent annual exceedance flow (This is not the 100-year storm.). For adult passage at low flows, NOAA Fisheries Service recommends using the 50 percent annual exceedance flow or 3 cubic feet per second (whichever is greater), and for juveniles, the 95 percent annual exceedance flow or 1 cubic foot per second (whichever is greater). Similarly, a design flow guideline used for fish passage projects in Alaska identifies the 2-year, 2-day duration flood using log-Pearson Type III for high-flow passage design criteria. Figure TS14N–3 shows a conceptual unit hydrograph for the 2-year, 2-day duration flood analysis method (Alaska Department of Transportation and Alaska Department of Fish and Game 2001).

Fish migration upstream may be limited during peak flow events, although migration patterns vary across species. Many fish migrate during spring or winter runoff events, sometimes following high-flow freshets or influxes of freshwater that affect water quality (salinity, turbidity, temperature). In the Pacific Northwest, winter steelhead and spring chinook spawning migrations overlap flood seasons, while coho and sockeye migrate at much lower flows in the fall. It is therefore important to understand both the flood and baseflow characteristics, if migration for the species of interest occurs during these periods.

Figure TS14N–3 2-year, 2-day duration design discharge



Hydrologic analysis for gaged streams

Hydrologic analysis techniques for characterizing flow during a specific period of interest or season usually involves flow-duration analysis of gage station data. Flood frequency recurrence analysis is typically performed using the guidelines in U.S. Water Resources Council (WRC), 1981, Guidelines for Determining Flood Flow Frequency, Bulletin #17B. Developing a flood frequency curve provides the designer with an estimate of flood magnitude and recurrence intervals for use in determining the size, configuration, and orientation of a fish passage facility. Computing flow duration is essential in determining the performance of a passage or screening structure across its operational range of flows. Flow-duration analysis is often performed by using daily average flow (or other periods such as 3-day, 5-day, or weekly) during the period of interest. A more detailed description of flow duration analyses is provided in NEH654.05.

Hydrologic analysis for ungaged streams

Often, gages are not sufficiently close to a project site or located within the same river system. Several methods are available to the designer for determining the magnitude and recurrence interval of seasonal high flows in ungaged watersheds. These include regional regression equations, discharge correlation to adjacent gaged streams, or development of hydrologic rainfall-runoff models.

The U.S. Geological Survey (USGS) has regional regression equations for estimating flood events based on watershed area, annual precipitation, and regional variables. Regression equations have been developed for many states and can usually be obtained from state USGS offices. Typically, the regression curves are in the form identified in equation TS14N–1.

$$Q_x = aA^bP^c \qquad \text{(eq. TS14N–1)}$$

where:

Q_x = x-year peak flood discharge (10-yr flood)
a = regression constant related to basin parameters
A = watershed area
b = regression exponent related to basin parameters
P = annual precipitation
c = regression exponent related to rainfall characteristics and annual recurrence event

The designer can also use transfer techniques to estimate flow characteristics at a project location in an ungaged stream, using the results of an analysis of streamflow data at an adjacent, gaged location. In addition, a variety of mathematical and computer hydrologic modeling systems (HEC–HMS, WinTR–20, and ArcHydro) are available to aid the designer. Depending on the hydrologic model, either single event peak flow or continuous multiple event modeling can be performed. The use of regional regression, transfer techniques, and hydrologic modeling are described in NEH654.05.

Hydraulic analyses overview

Hydraulic analyses are performed to evaluate flow conditions through a fish passage or screening structure. Typically, hydraulic design is an iterative process that balances available water and flow rates with site conditions and limitations, biological design criteria, and evaluation of a variety of potential hydraulic flow control structures. The following is a general overview in the approach for performing hydraulic analyses of a fish passage feature. Further description of fish screens is provided at the end of this section.

The first step in a hydraulic analysis is to characterize streamflow and morphology. Important data elements that are necessary to characterize the project site include flow patterns, velocity and depth, fish migration paths and holding pool locations, identification of potential sediment scour and deposition zones, and forebay and tailwater conditions. This information is essential in aiding a designer in selecting the appropriate location and design configuration of the fish passage facility. Field measurements and surveys are needed, particularly to determine low-flow characteristics, site geometry, and local topography.

Once stream conditions are characterized, potential fish passage design alternatives can be developed and evaluated. Fish entrances, ladders, and exits typically use flow control structures such as weirs, gates, and orifices. Two of the most critical pieces of hydraulic information in the design of a fish passage facility are flow circulation patterns above, below, and adjacent to the fishway site and water surface elevations

across the range of operating flows identified in the hydrologic analysis. The following section describes models available for hydraulic analysis of stream conditions and basic equations used for design of hydraulic design of weirs, gates and orifices. A more detailed description of hydraulic analyses is provided in NEH654.06.

Hydraulic models

The current standard for evaluating stream hydraulics is to develop a computer hydraulic model. Several models (such as HEC–RAS) are available for predicting water surface elevations, forebay and tailwater conditions, flow and diversion characteristics, and site velocities and depths (NEH654.06). Hydraulic analysis and design is an iterative process, balancing the various criteria and design requirements of the project. Therefore, the designer should perform separate calculations of composite flow profiles due to the complex nature of the hydraulic structures associated with fish passage facilities.

Evaluating existing hydraulic conditions will provide the designer with forebay and tailwater curves used in setting the preliminary invert elevations for the fish passage entrance and exit areas. Both tailwater and forebay rating curves are required for a wide range of flows (if available) for fishway design. The difference between upstream and downstream water surface elevations at the entrance and exit is the total change in head that the feature must be designed for. Structural head is a major determinant in how much flow will likely be diverted into the fishway. Completing stream hydraulics analyses and determining the range of operational flows for a passage facility begin the design of the actual fishway.

Concrete fishways and ladders

Fishways and ladders provide migrating fish with upstream passage around or through fish passage barriers. The general function of a fish passage facility is to attract fish into the structure and step them up the gradient created by the barrier to a point upstream, where they exit the ladder into the river and resume migration. The following section contains criteria, equations, and schematics related to designing concrete fishways and ladders (fig. TS14N–4).

General overview

Fishways and ladders are constructed in many different configurations from a range of materials. Common variations include:

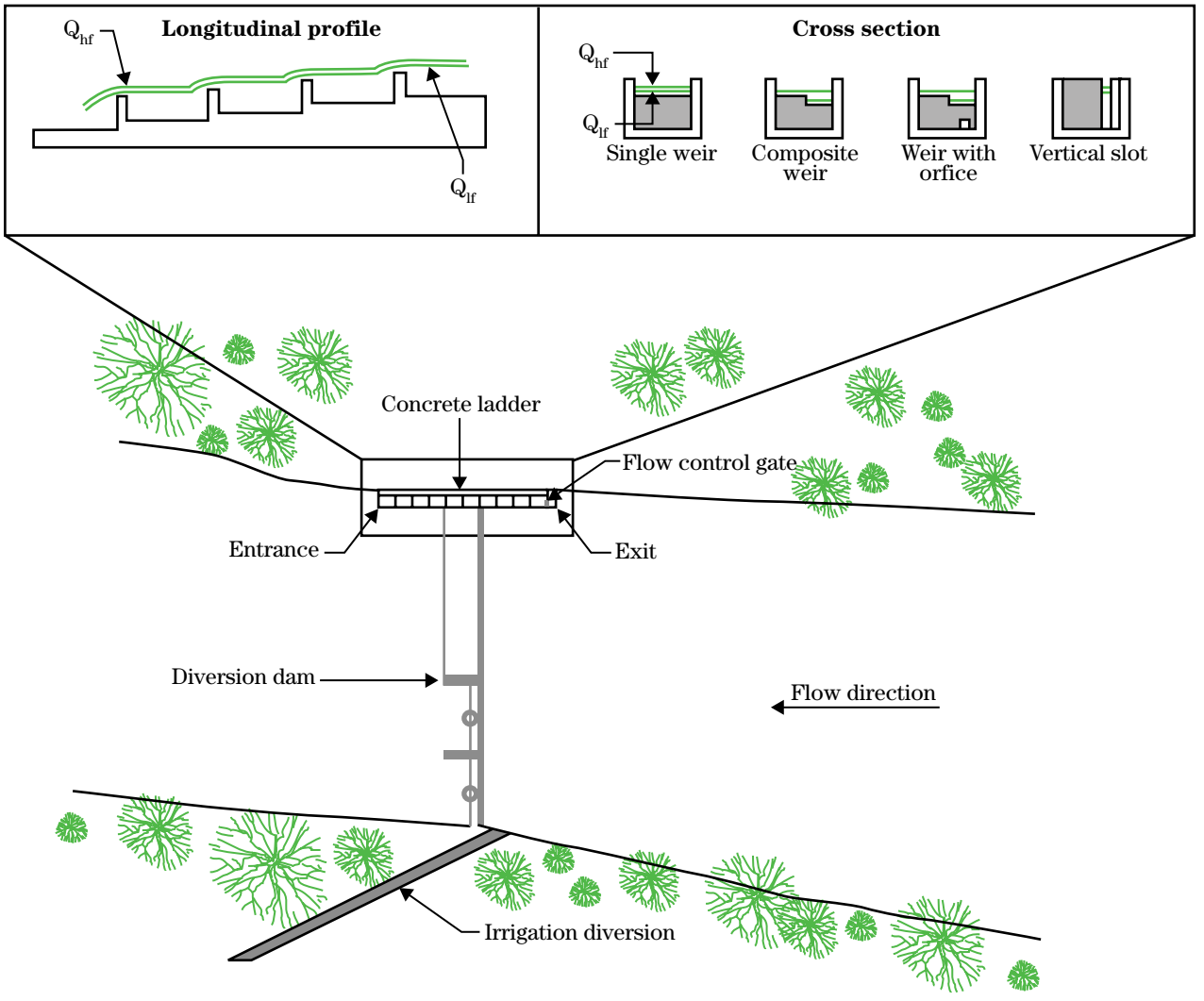
- excavated, earthen channels artificially roughened with large rocks
- seminatural channels equipped with stair-stepped resting pools held in place with rocks, logs, or stoplogs
- concrete and/or metal structures that slow water velocity enough to provide upstream passage

These structures are designed to function across a range of flows and are often built at fish passage barriers with excessive drops or velocities. Many fishways and ladders in common use today are pool-forming structures.

Pool-forming fishways are usually constructed with concrete, metal, or dimensional lumber and can be designed to take all, or part, of the total streamflow. Partial-flow fishways are more difficult to design than full-flow fishways constructed across the entire channel. To divert only a portion of the flow, a water control structure must be included at the top (the fishway exit from a fish's viewpoint), that provides a permanent, relatively maintenance free water supply into the fishway. Pool and weir or orifice fishways are often designed with stoplogs or gates to allow adjustments to pool depth according to streamflow. Although fishways are usually more difficult to maintain proper movement of bed load and debris, they can be installed on gradients up to 10 percent.

Pool-forming fishways function similarly to natural step-pools formed by logs, rocks, or bedrock outcrops along natural stream reaches. Flows down a relatively steep channel can be governed by weirs, slots, or other restrictions that hold back part of the flow and create resting pools. Since fish are supplied resting pools along the fishway, structural length is generally not a concern.

Figure TS14N–4 Plan view of a generalized concrete ladder fish passage facility



Two common pool-forming fishways are pool and weir/orifice. Commonly referred to as fish ladders, the resting pool depth in these structures is set by the height of channel-spanning weirs or headwalls (fig. TS14N–5 modified from Orsborn (1985)). Water flows over the top of a weir (pool and weir), or through a submerged orifice (pool and orifice), depending on flow rate. These structures are designed for fish that are able to jump over obstacles (pool and weir), or for nonleaping fishes, through submerged orifices at low flows. Water generally flows directly from pool to pool (rather than in a zigzag direction) to minimize energy expenditures on migrating fish. Pool and weir/orifice fishways can take many forms, but are generally useful at gradients up to 10 percent.

For juvenile and small-bodied adults, pools should be spaced no further than 15 feet with a drop of no more than 9 inches across pools. Pool spacing can be increased to 20 feet and head differential to 12 inches for adult fish.

Another type of fish ladder is a vertical slot. These structures are usually a rectangular channel made of concrete or metal in which a series of regularly spaced metal or concrete panels are installed perpendicular to the flow (fig. TS14N–6 modified from Orsborn (1985)). Each panel has a narrow slot from top to bottom and is designed to work with low velocities. Water spills from chamber to chamber through vertical slots, and pools are formed as the flowing water is backed up at each slot opening. Pool depth and velocity in each chamber are determined by slot width and the quantity of water flowing down the fishway. Although vertical slot fishways can be designed to pass a wide variety of fish species over a significant flow range, they are less passable for fish that tend to follow or cling to walls or jump over weirs. The pools of a vertical slot fishway are hydraulically complex and do not supply resting areas as tranquil as a pool and weir/orifice ladder. Consequently, these structures must be set at a low gradient to pass weak-swimming fish, although they will pass strong swimming fish at relative steep slopes. The vertical slot fish ladder transports bed material efficiently, but is susceptible to debris blockages at each of the vertical slots.

Weirs, orifices and gates (flow controls)

Weirs, orifices, and gates are found throughout fish passage design features. The following section describes general equations and resources for evaluating flow through weirs, orifices, and gates. Specific biological considerations for the main components of the fish passage feature including the entrance, ladder, and exit follow in the next section.

General weir flow and orifice flow equations are typically in the form of equations TS14N–2 and TS14N–3 and are illustrated in figures TS14N–7 and TS14N–8, respectively. Many references provide additional information and ranges of discharge coefficients for the many types of weirs, orifices, gates, and flow conditions. Although orifice and gate equations are derived from the same general equation, the current fish passage design practice is to include orifices with weirs. A few of the references listed below provide theory and calculations for weir and orifice flow. In addition, example solutions to weir and orifice flow equations are found at the end of this technical supplement.

- U.S. Department of Interior, Bureau of Reclamation Water Measurement Manual, 2001

  http://www.usbr.gov/pmts/hydraulics_lab/pubs/wmm/wmm.html

- U.S. Department of Interior, Bureau of Reclamation Design of Small Canal Structures, 1978

  http://www.usbr.gov/pmts/hydraulics_lab/pubs/manuals/SmallCanals.pdf

- International Institute for Land Reclamation and Improvement, Discharge Measurement Structures, 1978

- ISCO Open Channel Flow Measurement Handbook, 1989 (Grant and Dawson 1989)

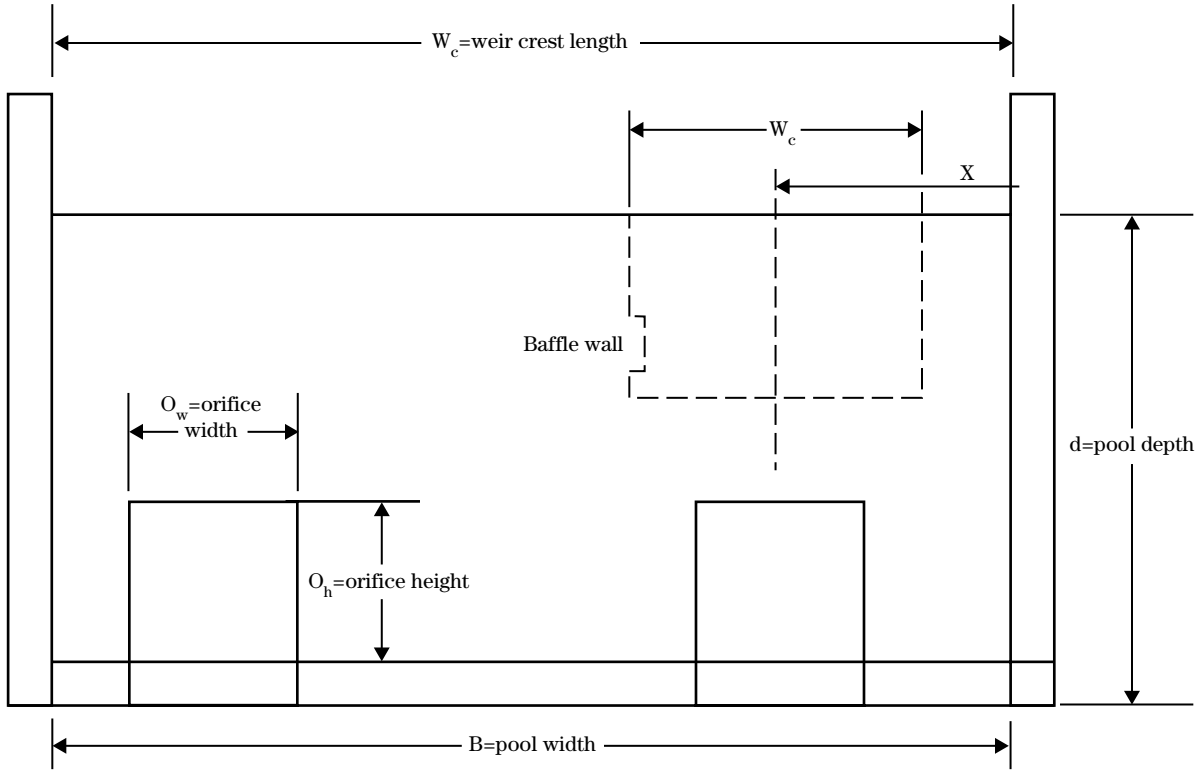
Equation TS14N–2 is the general form of a weir equation and can be used to estimate discharge, given water surface elevation and weir height, or back-calculate water surface elevations by rearranging the equation to solve for head on the weir (fig. TS14N–7).

$$Q = C_d L H^{1.5} \qquad \text{(eq. TS14N–2)}$$

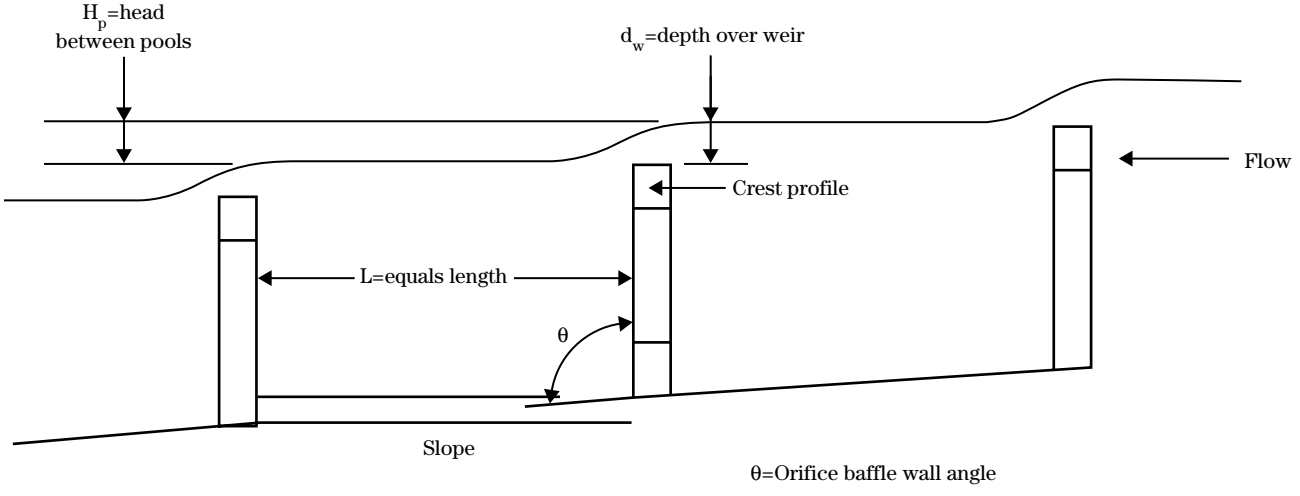
where:

Q = flow rate (ft^3/s)
C_d = coefficient of discharge
L = weir length (ft)
H = head above the weir crest (not including velocity head) (ft)

Figure TS14N–5 Cross section and profile views of a pool and weir/orifice fishway



Cross section



Profile

Once the discharge or water surface elevations are determined, a back check should be performed, using equation TS14N–3, to ensure that velocity over the weir does not exceed the burst swimming speed of the target fish.

$$V_{weir} = \frac{Q}{L0.67H} \qquad \text{(TS14N–3)}$$

where:
V = velocity (ft/s)

Equation TS14N–4 is the general form of an orifice equation and can be used to estimate discharge, given water surface elevation and orifice dimensions (fig. TS14N–8).

$$Q = kA_o\left(2g\Delta h\right)^{0.5} \qquad \text{(eq. TS14N–4)}$$

where:
Q = flow rate (ft^3/s)
k = flow coefficient (function of opening size and shape)
A_o = area of opening (ft^2)
g = acceleration due to gravity (ft/s^2)
Δh = head differential (ft)

Again, once the discharge or water surface elevations are determined, a back check should be performed, using equation TS14N–5, to ensure that velocity over the weir does not exceed burst swimming speed of the target fish.

$$V = \frac{Q}{A_o} \qquad \text{(eq. TS14N–5)}$$

Figure TS14N–6 Plan and end views of a vertical slot fishway

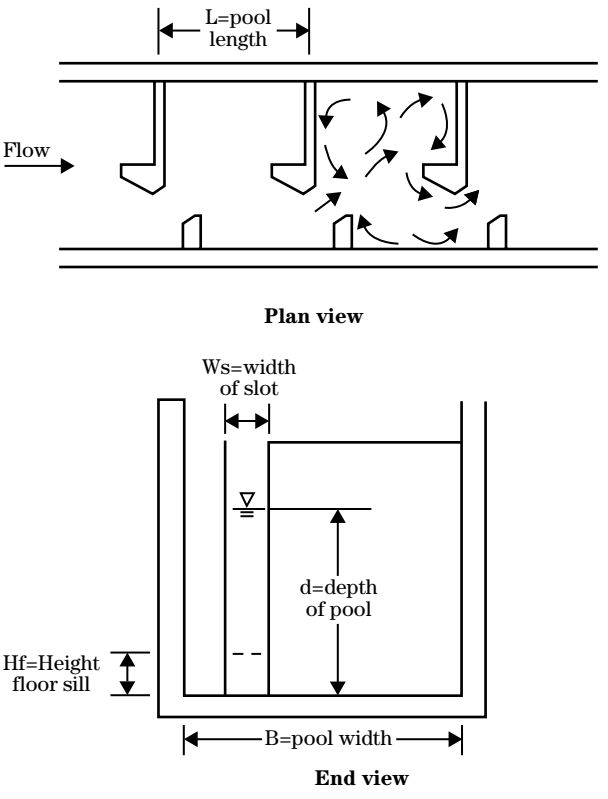


Figure TS14N–7 Weir schematic

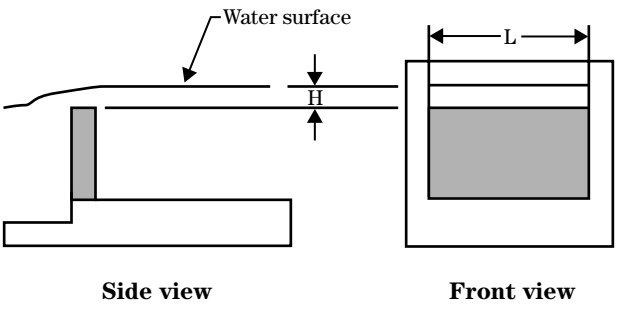
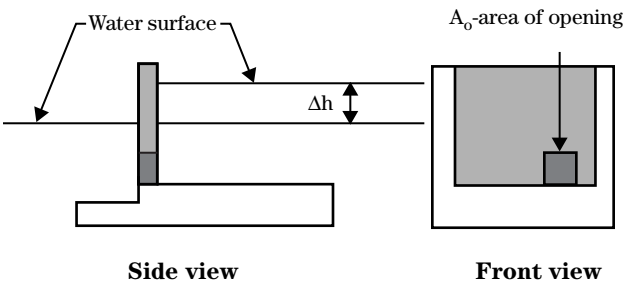


Figure TS14N–8 Orifice (gate) schematic



Concrete ladder pools

A final step in designing a concrete ladder system is to evaluate pool size. Pool volume should provide adequate capacity and depth to dissipate hydraulic energy, maintain stable flow, provide room for fish to accelerate and jump, and space to meet fish run capacity. Hydraulic capacity provides adequate energy dissipation so that stable, plunging flow occurs through the pool. If the pool is undersized, flow instabilities can occur in the form of surges, water fluctuations, or heavy turbulence. Pool size and shape should be configured so that fish have adequate room to accelerate and burst through openings or leap over weirs. Finally, fish capacity may be a consideration where fish runs are large enough to potentially overload the system. An overloaded fish ladder forces fish to hold in a queue until the structure can be passed. Overloaded fishways can cause significant adverse delays and should be minimized as part of the design process.

The hydraulic analysis for determining pool design configuration involves a detailed assessment of hydraulic jump characteristics. Ideally, the downstream weir is established at a height and length from the upstream weir, so that the hydraulic drop has plunging flow conditions with a fully submerged jump and no streaming flow conditions (fig. TS14N–9 (Bates 1992)).

Chow (1959) provides a method for evaluating hydraulic jump characteristics of a vertical drop (fig. TS14N–10). The general approach is to first evaluate the unsubmerged jump condition (eqs. TS14N–6 through TS14N–13), and then set the downstream weir at a height and length that forces a submerged hydraulic jump. For fish passage design, the jump is submerged by establishing the downstream weir height above the sequent depth (y_2). Downstream weir location is then set a distance beyond the drop and hydraulic jump lengths (L_d+L_j) to develop plunging flow.

Figure TS14N–9 Plunging and streaming flow

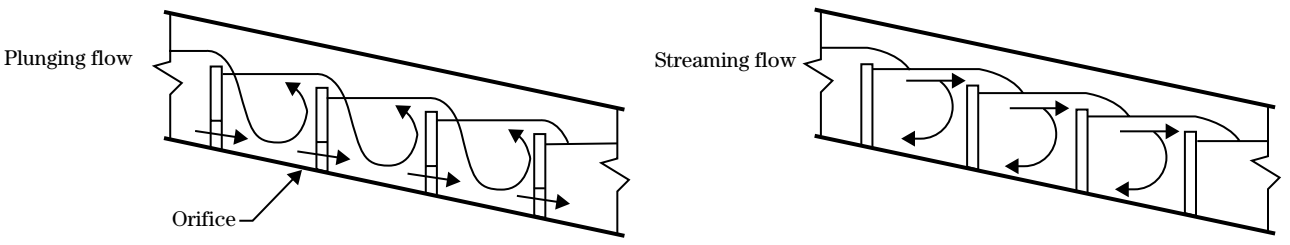
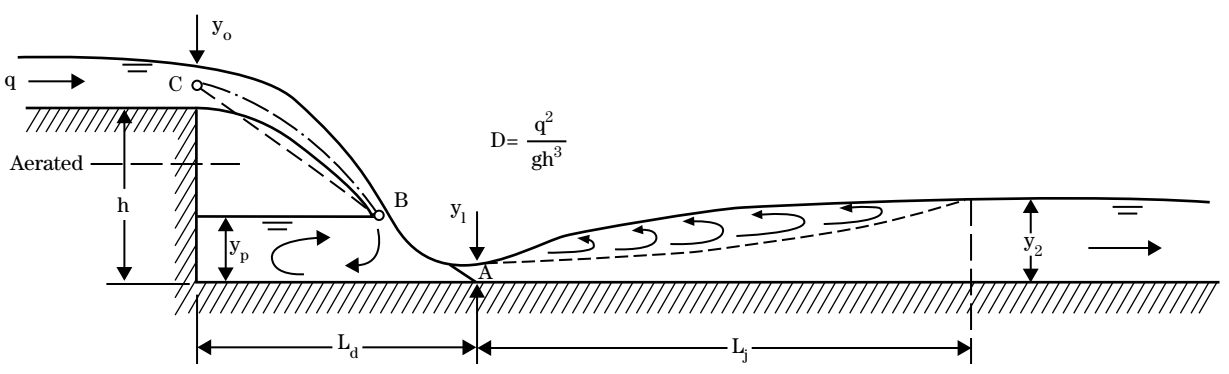


Figure TS14N–10 D number parameters for evaluating hydraulic jump geometry of a vertical drop spillway



The drop number (D) is determined using equation TS14N–6.

$$D = \frac{q^2}{gh^3} \qquad \text{(eq. TS14N–6)}$$

where:
D = drop number, dimensionless
q = unit discharge (ft^2/s)
g = gravitational constant (ft/s^2)
h = drop height (ft)

Once the drop number is determined, equations TS14N–7, TS14N–8, and TS14N–9 are used to evaluate initial jump height (y_1), final jump height (y_2), and drop length (L_d).

$$y_1 = 0.54hD^{0.425} \qquad \text{(eq. TS14N–7)}$$

$$y_2 = 1.66hD^{0.27} \qquad \text{(eq. TS14N–8)}$$

$$L_d = 4.3hD^{0.27} \qquad \text{(eq. TS14N–9)}$$

The final step is to determine the jump length (L_j) using equations TS14N–10 through TS14N–13 (Krochin 1961).

$$\frac{y_2}{y_1} = 0.5\left(\sqrt{1+8Fr_1^2} - 1\right) \qquad \text{(eq. TS14N–10)}$$

$$Fr_1 = \frac{V_1}{\sqrt{gy_1}} \qquad \text{(eq. TS14N–11)}$$

$$V_1 = \frac{Q}{Ly_1} \qquad \text{(eq. TS14N–12)}$$

$$L_j = \beta y_2 \qquad \text{(eq. TS14N–13)}$$

where:
Fr = Froude number, dimensionless
V_1 = velocity at the start of the jump (ft/s)
Q = discharge (ft^3/s)
L = weir length (ft)
β = jump length coefficient

The jump height coefficient can be determined through empirical values shown in table TS14N–6 (Department of Interior Bureau of Reclamation 1984).

The next step in design is checking to ensure that the weir system is not washed out with streaming type flows and that the plunging flow condition exists (fig. TS14N–9) (Bates 1992; Rajaratnum, Katopodis, and Lodewyk 1988). Rajaratnum developed techniques for evaluating plunging and streaming flows using the following scaling equations. For plunging flows, the dimensionless discharge (Q_*) is approximately 0.61. Equation TS14N–14 is solved for the dimensionless discharge of plunging flow.

$$Q_* = \frac{Q}{\sqrt{gL^2h^3}} \qquad \text{(eq. TS14N–14)}$$

where:
Q_* = dimensionless discharge
Q = discharge (ft^3/s)
g = gravitational acceleration (ft/s^2)
L = length of weir (ft)
h = head across weir (ft)

Another approach to determine weir flow characteristics is to check the streaming flow condition. For streaming flows, equation TS14N–15 is used to determine the dimensionless discharge. If the streaming flow condition exists, modifications to weir and pool configurations are recommended to force plunging flow conditions and provide satisfactory fish passage hydraulics.

$$Q_* = \frac{Q}{\sqrt{gSL^2h^3}} \qquad \text{(eq. TS14N–15)}$$

where:
S = slope (ft/ft)

The dimensionless discharge is equal to,

$$Q_* = 0.5\frac{L_{pool}}{h} \qquad \text{(eq. TS14N–16)}$$

where:
L_{pool} = length of pool (ft)
h = depth of streaming flow, similar to head above weir

Table TS14N–6 Jump height coefficient

Channel slope	Fr ≥ 4	4 ≥ Fr ≥ 3	3 ≥ Fr ≥ 2	2 ≥ Fr ≥ 1
0.00	6.15	5.54	4.99	4.49
0.05	5.20	4.68	4.21	3.79
0.10	4.40	3.96	3.56	3.21
0.15	3.85	3.46	3.12	2.81
0.20	3.40	3.06	2.75	2.48
0.25	3.00	2.70	2.43	2.19

A final design check is to evaluate pool volume based on energy dissipation criteria for the target species. Excessive turbulence and bubble formation can physically fatigue, injure, or disorient fish transiting a passage structure. Bates (1992) and WDFW (2003) suggest using the energy dissipation factor (EDF), (eq. TS14N–17), to estimate forces acting on fish in a ladder pool. EDF values greater than 4 foot-pounds per cubic feet per second for salmon and steelhead and 3 foot-pounds per cubic feet per second for shad (Larinier 1990) indicate adverse hydraulics in a ladder pool. Equation TS14N–17 can be applied to evaluate energy dissipation in pools less than 10 feet long, with an average width (for the calculation only) limited by a 4:1 side expansion from the weir opening, and pool depth at least 3h and sufficiently deep to submerge any hydraulic jump (Chow 1959).

$$EDF = \frac{\gamma Q h}{V} \qquad \text{(eq. TS14N–17)}$$

Energy dissipation factor (ft-lb/ft^3/s)

where:

V = volume of the pool (ft^3)
γ = unit weight of water (62.4 lb/ft^3)
Q = discharge, ft^3/s, through openings or over weirs
h = head (ft)

Entrance and attractors

Fish passage entrance design is a critical element of any fishway. The primary design goal is to site and configure the entrance so that it attracts fish into the passage channel by mimicking the hydraulics and morphology of natural analogs (waterfalls, cascades, log overpours). Traditionally, migrating fish seek and swim towards or alongside stream lines of higher velocity. Consequently, a fishway entrance must consider natural migration patterns along the river, as well as turbulence, velocity patterns, and dead spots that distract fish from entrance attraction flows. Designers should also account for holding patterns and migration routes, such as along the bankline of the river, and place the fish passage entrance proximate to these features. Field observations should include mapping flow patterns and velocity vectors to help identify and prioritize entrance locations. Dam spillways and penstocks significantly influence hydraulic velocity fields and affect the performance of the fish passage entrance. In these settings, it is critical that the fishway entrance focuses flow into a jet of higher velocity water that cleanly penetrates the tailwater and attracts fish (Bates 1992).

Fish passage entrances can be overflow weirs, orifices, or vertical slots. Ultimately, fishway entrance design is a balance between attraction velocity and maximum head for the fish to swim against, while also accounting for behavioral and migration patterns. Additional fish attractors, including auxiliary flows and pumped jets of water, are included in some fish passage features. However, these hydraulic features can also distract fish from entering the passage facility at the right location. Designers should be aware of both attractors and distractions near fish passage entrances and ensure that all distractions are eliminated from the entrance area. The following are useful criteria for fishway entrance location and hydraulics:

- The fishway entrance should be at the upstream-most point of fish passage adjacent to a barrier. Do not place fish passage facility entrances in turbulent areas.
- Provide adequate trashracks.
- Align low-flow entrances perpendicular to tailwater flow.
- Align high-flow entrances 30 degrees downstream off perpendicular to tailwater flow to help with flow penetration.
- Attraction velocities should be from 4 to 8 feet per second, preferably closer to 8 feet per second.
- Cross velocities should not exceed 2 feet per second.
- Auxiliary water velocities should be between 0.5 and 1.0 feet per second when pumped into the entrance chamber of a fishway.
- Approach flow should be parallel to the axis of the fishway entrance or at least no greater than 30 percent to the axis of the main current.
- Design and build fishway entrances to provide access across changing water surface elevations such as the tailrace of a hydropower facility or the low- and high-flow elevations of a natural stream.

Exit

The primary design considerations for the fish passage exit are headwater and flow diversion control, maintenance of diversion design discharges during fluctuating headwater conditions, protection from debris, and alignment with migration pathways to ensure that fish find their way upstream from the passage barrier. It is also important to locate the exit far enough upstream from the crest of the dam so that fatigued or disoriented fish do not fall back downstream. Typically, fish need to acclimatize and orient themselves to the river after traveling through the fish passage facility.

Oftentimes, the fishway exit is also used to divert and regulate streamflow into the passage structure. Gates, stoplogs, tilting weirs, and other combinations of slots and orifices are often used as flow control devices. The following list provides useful considerations for fishway exit conditions:

- Place exits away from spillways, powerhouse intakes, or other hydraulic structures that pose risk of harm to target species.
- Place exits in areas of positive flow to avoid stagnant, low-quality water.
- Design and build exits with adequate trashracks.
- Include adequate structural freeboard into a fishway exit to protect it from flood damage.
- Build the fishway exit so that it can be dewatered for maintenance and inspection.
- The length of the exit channel upstream should be a minimum of two standard ladder pools.

Rock fishways

The use of rock weirs and step-pools as fish passage features is a viable option in stream systems with large cobble to boulder channel beds. Use of rock emulates natural step-pool sequences, cascades, riffles, rock aprons, and log sills that fish naturally migrate past. They are typically more visually appealing than concrete and, in some cases, may be more cost effective (fig. TS14N–11).

Rock ladders have the same general features (entrance, ladder, and exit area) as concrete ladders, and many of the equations that are applied to designing concrete ladders can also be used to evaluate rock ladder performance. However, additional analyses are required to account for increased energy losses and turbulence induced from the uneven shape and placement of boulder and cobble materials. Hydraulic models should be developed to evaluate water surface profiles through rock ladders, especially when no structures are included to control flow rates into the fish passage channel, entrance, and exit areas. Flow control structures can be incorporated into the design to limit the amount of flow diverted into the fish passage channel. If not, the rock fishways must be designed to withstand a range of flows throughout the year, rather than diversions made only during fish migration seasons. A flow control structure may be necessary to protect the rock ladder from flood flows and provide adequate head to diversion facilities at low-flow conditions.

Much of the information available for designing step-pool features is related to studies performed on boulder and rock grade control weirs. These types of designs can be adapted to meet biological design criteria for fish passage. A rock ladder has three main components: a boulder, rock or cobble weir; scour or plunge pool; and tailwater area (fig. TS14N–12).

Boulder and rock weirs

Special design and analysis considerations are required when evaluating flow conditions over rock-weirs, boulder sills, and along step-pool sequences. Figure TS14N–12 is a schematic of the general elements associated with a boulder weir-step-pool. Compared to standard weirs, rock weirs significantly influence turbulence, resistance, energy losses, and water surface elevations. Although empirical equations for standard smooth-crested weirs are good as a first approximation, several modifications are required to more accurately evaluate flow hydraulics in boulder weirs.

Evaluating a boulder weir incorporates hydraulic theory associated with weir length modifications and flow contractions. Weir length is determined by measuring and adding together incremental distances between the boulders (fig. TS14N–13) and adjusting for each of

Figure TS14N–11 Plan view of a generalized roughened rock channel/rock ladder fishway

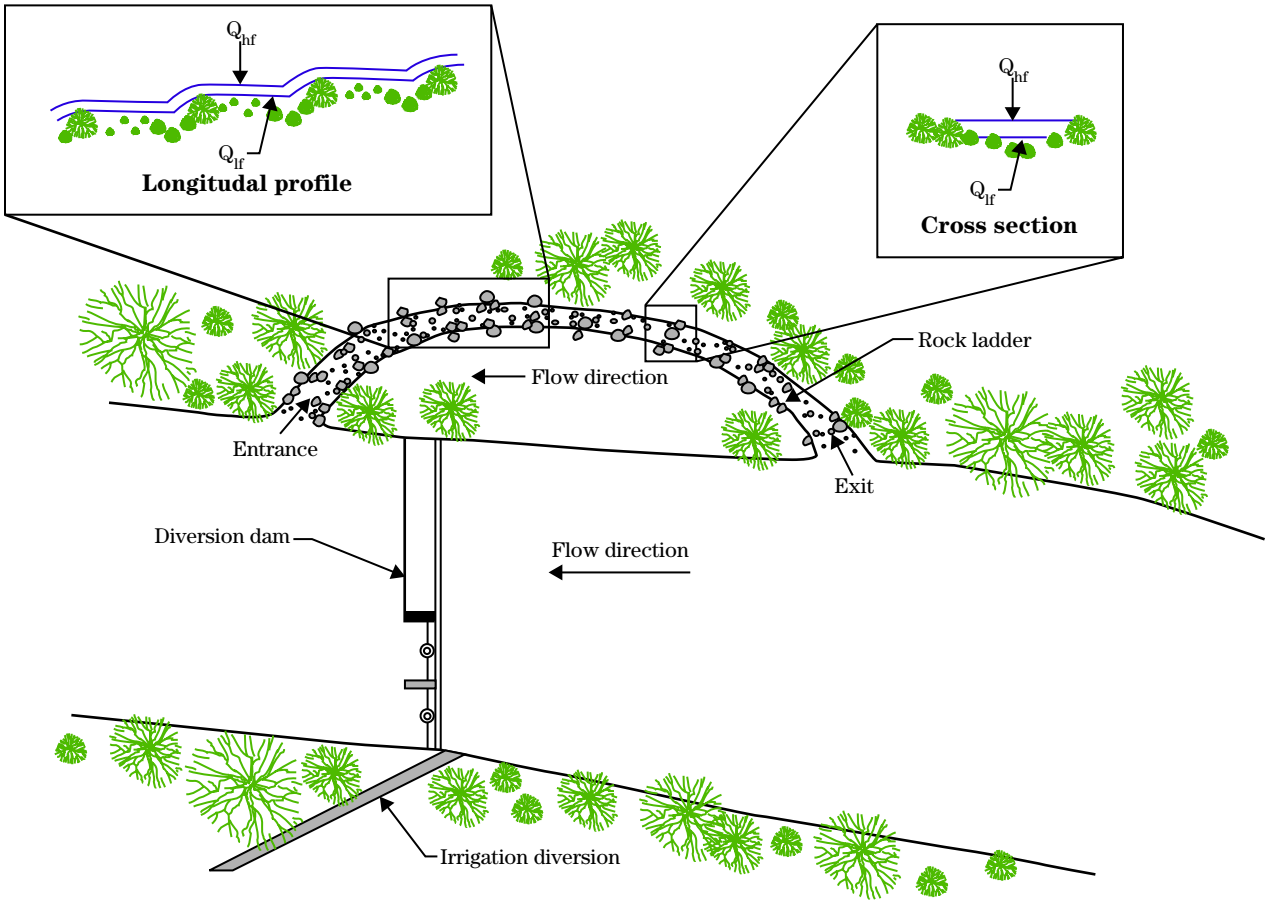
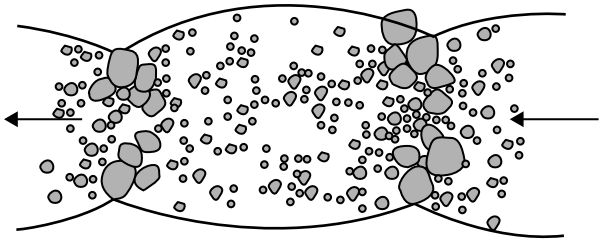
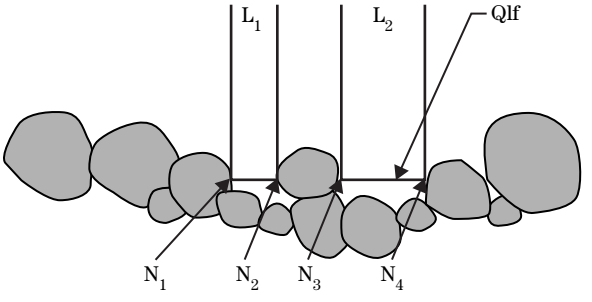


Figure TS14N–12 Boulder weir (plan view)



Boulder weir-step pool
Plan view

Figure TS14N–13 Boulder weir cross section



the side wall contractions. The basic weir equation is then modified using equation TS14N–18 (Chow 1959).

$$Q = C_d \left(\sum_{i}^{N} \left(L_i - 0.1NH \right) \right) H^{1.5} \quad \text{(eq. TS14N–18)}$$

where:
L_i = incremental widths (ft)
N = number of contraction sides

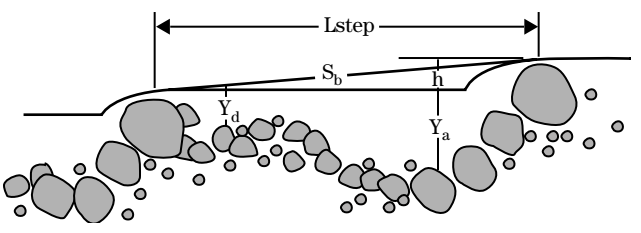
Boulder pools

The next step in evaluating boulder and rock weirs is to determine scour depth below the drop (fig. TS14N–14). Scour depth determination should be done for the highest design discharge expected at the site location. As an initial estimate, scour depth below the bed of the channel should be equal to the drop height from the water surface to the bed surface along the tailwater area. Several scour equations are available, although the most appropriate are plunge scour functions for vertical drop structures. Equation TS14N–19 (Jager 1939 in Simons and Senturk 1992) is derived from empirical analyses of scour downstream from grade control structures.

$$Y_s = 6\Delta h^{0.25} q^{0.5} \left(\frac{Y_d}{D_{90}} \right)^{\frac{1}{3}} \quad \text{(eq. TS14N–19)}$$

where:
Y_s = depth of scour (m)
Y_d = downstream depth of flow (m)
q = unit discharge (m^3/m-s)
D_{90} = sediment diameter with 90 percent of material finer (mm)
Δh = difference in head between upstream water surface and downstream water surface (m)

Figure TS14N–14 Boulder step-pool profile



Step-pool length is the final geometric element evaluated in designing a rock pool feature. Pool length and volume in boulder weirs is directly related to the EDF described in the previous section (eq. TS14N–17) and has a similar effect on fish passage success. Pool length equations from the previous section should be used as a first estimate. A second method to estimate pool length involves examining natural step-pool geometry and spacing in the same river system. The goal in designing a step-pool feature is to allow the fully turbulent flow jet to dissipate. Comiti (2003) reports a range of step-pool lengths based on head, channel slope, and scour depth listed in equations TS14N–20 and TS14N–21. Ratios in natural systems for pool drop to scour depth typically range between 1.0 and 2.0 for slopes greater 15 percent. However, as the slope flattens (less than 15%), step lengths to scour hole depth ratios typically begin to approach 3.0.

For slopes between

$$0.05 < \frac{h}{L} < 0.50 \quad \text{(eq. TS14N–20)}$$

The drop to scour ratio is

$$1.0 < 1 + \frac{Y_s}{h} < 3.0 \quad \text{(eq. TS14N–21)}$$

Rock sizing

The final design element for step-pool rock ladders is substrate sizing. Overall, the rock along the ladder must be designed to withstand the entire range of flow conditions. Designers should identify a safety range based on the accuracy of the design hydrology, hydraulics, and other site conditions and apply this range to subsequent rock size estimates. The primary design elements requiring rock sizing are the weir structure, plunge pool scour apron, and tailwater area.

Typically, rock weirs are comprised of boulders with interstitial cobbles. The boulder and rock features must be sized to withstand the highest expected flow event and provide openings and passage paths for fish during the migration period. Forces acting on the boulder and cobble rock on the weir crest include drag, lift, weight, and frictional resistance. Structural rock remains stable and in place, as long as weight and frictional resistance are greater than drag and lift forces. Equation TS14N–22 can be used to estimate minimum rock diameter on a boulder weir crest for fully turbulent flow over a rough horizontal surface,

with completely submerged rock and similar submergence along both faces of a weir. The major difficulties in using equation TS14N–22 are estimating the friction slope and verifying the assumption that similar submergence occurs on all sides of the rock. Friction slope is difficult to determine over a drop, but an estimate can be made using equation TS14N–9 for drop length. A general rule of thumb is that the rock size should be greater than the drop height. Another general criterion is that the final step-pool at the downstream end of a sequence should have a buried armor layer along the entire length of the step that is similarly sized to the weir and toe protection material.

$$D_{min} = \frac{\left(18\left(0.67H\right)S_f\right)}{\left(G_s - 1\right)} \qquad \text{(eq. TS14N–22)}$$

where:
D_{min} = minimum boulder median diameter (ft)
H = depth of flow over weir (ft)
S_f = friction slope (ft/ft)
G_s = specific gravity of rock (~ 2.65)

Rock size required along a weir crest can be determined using equation TS14N–23 when velocity acting on the weir structure is known (a function of crest height and the drop into a scour hole area). In addition, velocity (V_1) can be compared to the rock sizes shown in table TS14N–7 for guidance (Fischenich 2000). Designers are encouraged to perform more thorough calculations that refine friction slope and energy losses across the boulder weir and evaluate other factors affecting rock stability (countersinking rock to resist hydraulic forces).

Table TS14N–7 Incipient motion thresholds for rock sizes

Class name	Median diameter (in)	Critical velocity (ft/s)
Large boulder	>40	19
Medium boulder	>20	14
Small boulder	>10	10
Large cobble	>5	7
Small cobble	>2.5	5
Very coarse gravel	>1.25	3

Sizing rock for toe protection along the scour hole across the downstream face of a weir is largely done according to previously described methods. Rock diameter can be estimated using equation TS14N–23 (Department of Interior Bureau of Reclamation 1984; USACE 1994f) and table TS14N–7. In areas subject to toe scour, the resultant vector should be used with horizontal and vertical velocity components. In a step-pool sequence with a flat tailwater bed slope, the horizontal velocity (V_{weir}) can be used. If the channel bed between the scour hole and next weir drop is not flat, the resultant vector should be used to size the bed material in the sloped tailwater area.

As a first approximation, the D_{50} can be assumed to be one to two times the size of the drop height of the structure. The velocity (V_1) can be approximated using equations TS14N–24 and TS14N–25 and equation TS14N–3 to estimate the weir velocity (V_{weir}) in the horizontal plane. Equation TS14N–26 is used to specify the size gradation of riprap and stone for weir and scour hole protection areas.

$$D_{50} = \left(\frac{V_1}{2.57}\right)^2 \qquad \text{(eq. TS14N–23)}$$

where:
D_{50} = median diameter (in)
V_1 = velocity (ft/s)

$$V_1 = \sqrt{V_{weir}^{\ 2} + V_y^{\ 2}} \qquad \text{(eq. TS14N–24)}$$

$$V_y = \sqrt{2g\Delta h} \qquad \text{(eq. TS14N–25)}$$

$$1.7 < \frac{D_{85}}{D_{15}} < 2.7 \qquad \text{(eq. TS14N–26)}$$

Exposed bed material in the tailwater area of each weir pool should be sized using weir velocity (V_x) and equation TS14N–3 solved for the horizontal plane. This element of the boulder pool sequence provides protection for the upstream approach of the next downstream weir.

Rock sizing calculations are addressed in more detail in NEH654 TS14C, and grade stabilization structures are described in NEH654 TS14G.

Roughened channel fishways

Roughened channel fishways function similarly to steep, boulder-strewn, cascading stream reaches. These chutes or flumes contain roughness elements to break up streamflow and reduce water velocity. Roughened channels, unlike pool-forming fishways, do not create deep pools where fish can stop to rest before jumping or swimming to the next step. Instead, target species must use prolonged or sustained swimming speed to transit the entire length of the structure, with minimal or no resting. This same concept makes a baffled culvert more passable than a corrugated metal pipe which, in turn, is more passable than any smooth pipe (see description of culverts in next section). Roughened channels must be designed with careful consideration for the swimming capabilities of target species, and overall fishway length should be kept to the minimum possible for prevailing site conditions.

Engineered channel

An engineered channel is a roughened waterway that is an excavated earthen channel or a natural stream channel lined with a series of boulders that are properly sized and placed for site streamflow and gradient. Roughness elements, commonly boulders or concrete blocks, are anchored in place where streamflows are high or gradients are steep. At slopes up to about 5 percent, roughness elements can be embedded into a cobble and gravel streambed; for slopes between five and 9 percent, they must be anchored into a concrete channel subgrade. Strategic placement and anchoring of rocks or concrete blocks into modified natural stream channels can significantly improve passage conditions. Maximum engineered channel length depends on the swimming abilities of the target fish. A boulder and rock weir fishway (or fish ladder) is required if site conditions dictate a long engineered channel that likely exceeds the known swimming abilities of the target species.

Engineered, steepened channels are designed to survive very high flows, are easily maintained, and cost less to build than concrete fishways. Guidance for designing these fishways can be found in previous chapters. There are no standard empirical methods to predict passage using this informal method, so they cannot be built with gradients as steep as roughened-channel fishways. However, the geometry of natural analogs in the same stream system (cascades or bedrock chutes) can provide designers with insight into structural limitations at a given project site.

Engineered channels have been successfully used in some Midwestern states to pass warm-water fishes such as redhorse, walleye, northern pike, and various minnow and sucker species. Commonly referred to as rock ramps, these structures have been in use for several years to provide fish passage at low head dams (fig. TS14N–15). Rock ramp fishways can also provide additional protection against undermining from toe scour caused by water spilling over the face of a dam.

Denil

Made from wood, steel, and/or concrete, a Denil fishway is a rectangular channel fitted with a series of symmetrical, closely spaced baffles that redirect flowing water and allow fish to swim around or over a barrier (fig. TS14N–16). The figure on right is modified from Powers et al. (1985). Baffles placed on the floor or walls of the relatively steep, (10 to 25% slope) rectangular flume reduce mean flow velocities to ranges negotiated by migratory fish. Denil ladders generally do not have resting areas, although pools can be included in the design to provide resting areas or velocity reductions. Further, switchbacks can be added to minimize the footprint of the structure. When small-bodied or weaker swimming fish are targeted for passage, Denil ladders can be built at a shallower slope with smaller baffles, or closer baffle spacing, to minimize physiological exertion.

Many different Denil fishway designs are presently in use. The most common, the plane baffle or standard Denil fish ladder, is composed of baffles angled up from the floor at 45 degrees and spaced between 2 and 4 feet. Standard Denil ladders are commonly applied at slopes between 15 and 20 percent. Another frequently seen approach uses herringbone-patterned baffles made of thin steel attached only to the bottom of the flume; the two sides of the channel remain smooth. Although the width of this design is generally not limited, the maximum applicable slope is about 15 percent.

All Denil ladder applications are susceptible to damage from debris, as well as debris accumulation. They

Figure TS14N–15 (a) North Dam on the Red River of the North (ND) before construction; (b) after construction of an engineered channel fishway

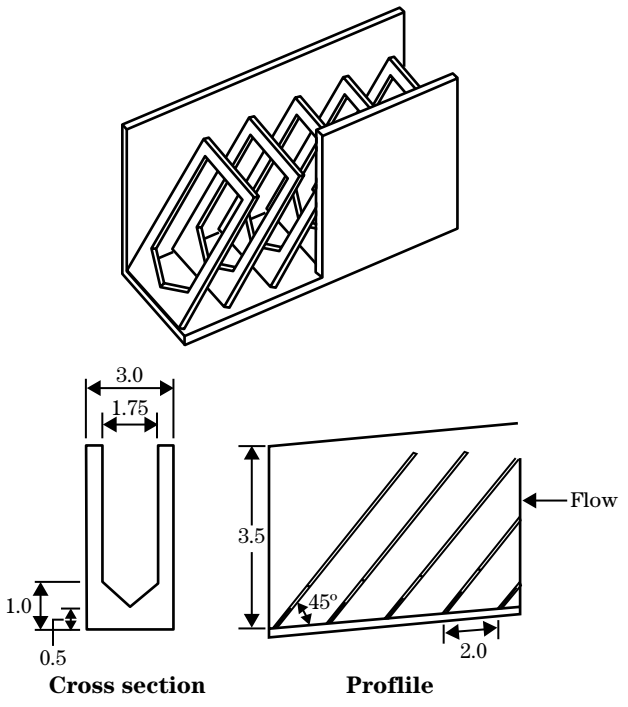
(a)

(b)

Figure TS14N–16 (a) Site photo; (b) schematic of a common Denil fishway

(a)

(b)



are generally most applicable in settings where water surface elevation fluctuations are 1 foot or less. Denil fishways only provide adequate fish passage conditions under a narrow range of flow. Consequently, adequate flow control at the upstream opening is essential for successful operation and fish passage.

Alaska Steeppass

Alaska Steeppass fishways, a variation of the Denil ladder, are prefabricated, modular, and usually constructed of a lightweight material like aluminum (fig. TS14N–17). The figure on right is modified from Powers et al. (1985). These factors make the Alaska Steeppass relatively economical to build, install, and use, especially for temporary applications or in remote locations. The Alaska Steeppass has a more complicated baffling system than a Denil fishway, but this design controls water more efficiently and allows installation and operation at slopes up to 35 percent. In addition, internal baffle design permits the Alaska Steeppass to successfully operate at lower flow rates than a Denil ladder. However, generally smaller inlets and complicated baffles also make steeppasses more susceptible to debris problems than common Denil ladders. Flow control is also critical for these structures, and headwater range generally cannot fluctuate more than about 1.5 feet without creating passage difficulties.

Baffle design in an Alaska Steeppass can be adjusted to fit the passage needs of target species. Although the floor fin angle (Ø) is generally 45 degrees, side fin angle (θ) adjustments from 45 to 90 degrees are commonly used to reduce air entrainment and structural turbulence to improve passage conditions.

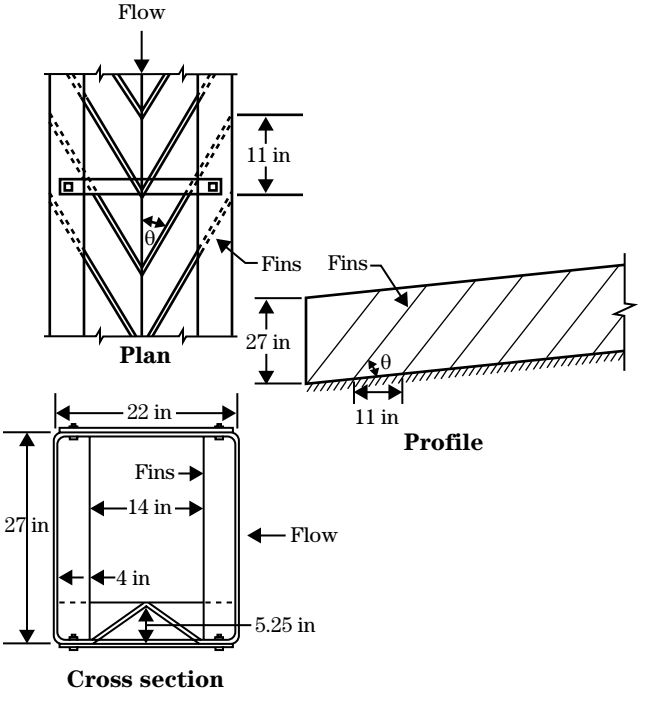
Culvert modification and design

Culverts may be the most common artificial barriers to upstream fish passage. Although usually associated with road crossings, they are also found under railroad grades, pipeline crossings, irrigation canals, buildings,

Figure TS14N–17 (a) Site photo; (b) schematic of an Alaska Steeppass fishway

(a)

(b)



and parking lots. Culverts are usually round, rectangular, elliptical, flat-bottomed, or bottomless, and are often made from steel, concrete, or plastic (PVC, ABS). Their interiors can be relatively smooth, but are often roughened by streambed substrate and/or corrugations. Culverts create fish barriers in one or more of the following ways:

- high velocities or sudden velocity changes at the inlet or outlet or inside the culvert barrel
- inadequate flow depth in the culvert barrel during critical migration periods
- excessive length without adequate resting areas
- significant drop at the culvert outlet
- debris accumulation at the culvert inlet, outlet, or inside its barrel
- excessive turbulence inside the culvert or at its outlet or inlet

This section on modifying or installing culverts to provide fish passage relies heavily on results of fish passage studies, field applications, and information published by a team of engineers and biologists from the Washington State Department of Fish and Wildlife. Their work, Design of Road Culverts for Fish Passage (WDFW 2003), is commonly cited in fish passage regulations and studies across the United States. The following sections provide a general overview of options for designing, installing, or modifying new or existing culverts for fish passage.

Modifications to existing culverts

Generally, an unblocked culvert with an outfall greater than 0.8 foot, diameter 50 percent of bankfull channel width or less, and slope greater than 1 percent should be considered as at least a partial barrier to migratory fish. Short of replacement, culverts can be modified in a number of ways to improve fish passage. For example, perched culverts are usually undersized and relatively steep, and over time, the channel bed often drops, leaving the outlet lip many inches or feet above the water surface elevation of the downstream pool (fig. TS14N–18). If site conditions allow, fish passage at a perched culvert can be improved by raising the culvert outlet pool water surface elevation with a channel-spanning structure or series of structures (fig. TS14N–19).

Figure TS14N–18 A significantly perched culvert (*Photo courtesy of Dick Quinn, USFWS, Newton, MA*)

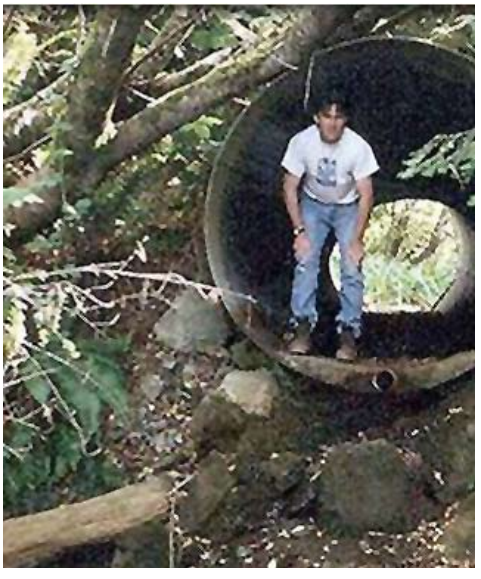


Figure TS14N–19 Series of channel spanning weirs used to step up water surface and raise outlet pool to culvert lip

Excessive velocity and shallow-flow depths also impede fish passage in existing culverts—this condition is especially common in concrete box culverts. Retrofitting a culvert with baffles may improve fish passage across a range of flows, but only where site conditions allow. Placing baffles within a culvert may reduce capacity of the culvert by an unacceptable amount. Baffles are a series of features that increase hydraulic roughness inside the barrel of a culvert (fig. TS14N–20 (modified from WDFW (2003); variables shown are defined in their appendix D)). Unlike hydraulic control structures (weirs), which independently reduce velocities, baffles work together to reduce the average cross-sectional velocity throughout the length of a culvert.

Installing baffles into a culvert should only be considered as a temporary solution to improve fish passage. Adding baffles reduces hydraulic capacity, generally increases the risk of failure from flooding conditions, and makes culverts more prone to capture debris and bed load. Baffled culverts require maintenance, so the barrel diameter should allow at least 5 feet of headroom for crews to safely work inside. Proper and frequent maintenance of culvert baffles is essential to ensure that a modification made in the name of improving fish passage does not result in poorer conditions than existed prior to the retrofit.

Installing baffles alters the hydraulics of a culvert and requires a good knowledge of the flow characteristics of the subject stream. Baffles installed near the inlet of a culvert should be placed at least one culvert diameter downstream at a height that will ensure subcritical flow at high discharges. Baffle systems like those shown in figure TS14N–20 should only be installed in culverts with slopes no greater than 3.5 percent. Corner baffles are generally used in culverts with slopes between 1.0 and 2.5 percent and are intended to provide wall roughness, while minimizing debris blockage potential (fig. TS14N–21). Notch baffles can be applied in culverts with slopes between 2.5 and 3.5 percent, but are designed to function as weirs at slopes greater than 3.5 percent.

Culvert replacement or installation

Replacing existing culverts or installing new road crossings can challenge the engineer and fisheries biologist: a hydraulically efficient culvert often poses a barrier to fish passage because of the inherent hydraulic differences between supercritical and subcritical flow. Standard culvert hydraulic and structural analyses apply. Fish passage requires more data to be considered. However, the three replacement and installation options described provide approaches that often balance resource constraints and needs in an economical manner. Still, culverts may not always provide adequate fish passage, and other more invasive

Figure TS14N–20 Common baffle styles for round and box culverts

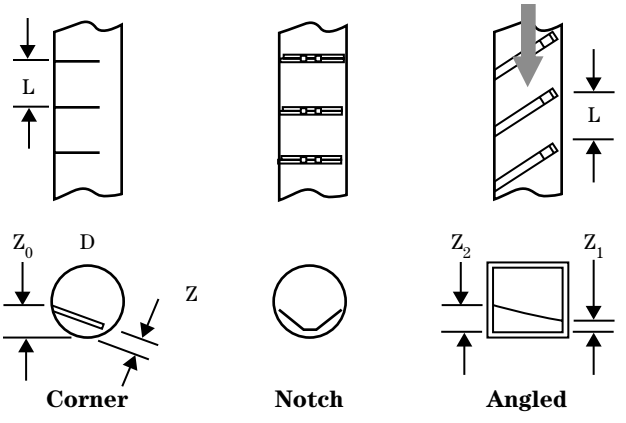


Figure TS14N–21 Round, corrugated metal culvert retrofitted with corner baffles to improve fish passage

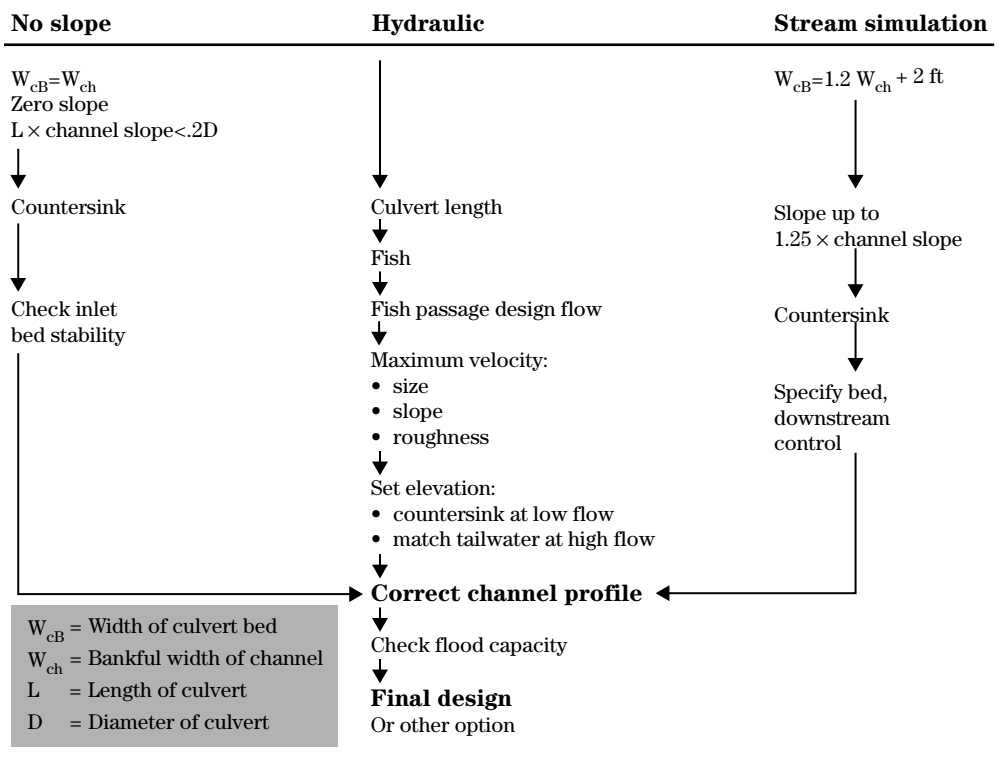
and expensive options such as rerouting a waterway or building a bridge may require consideration. Road abandonment can be an option if a culvert barrier to fish passage is along a poorly maintained and/or unused road.

The no-slope, stream simulation, and hydraulic design approaches to culvert replacement and installation are a mixture of standard methods and new advances in fish-friendly culvert design. The no-slope and stream simulation options are favored over the hydraulic design approach, but project and site-specific conditions will affect which method and outcome is selected. An overview flowchart of culvert criteria and a general design process is presented in figure TS14N–22 (modified from WDFW (2003)).

No-slope option
The no-slope design approach is founded in the assumption that a sufficiently large culvert installed at grade will allow the natural movement of bed load and formation of a stable bed inside the culvert barrel. Maintaining sediment transport continuity and the preservation of a natural channel bed inside the culvert usually provides excellent fish passage conditions across a range of flows. A no-slope culvert is defined by the following characteristics:

- width equal to or greater than the average bankfull channel width where the culvert meets the channel bed
- relatively flat gradient
- downstream invert is countersunk into the channel bed by a minimum of 20 percent of the culvert diameter (or rise, for noncircular culverts)
- upstream invert is countersunk into the channel bed by a maximum of 40 percent of the culvert diameter (or rise)

Figure TS14N–22 General flowchart of the culvert design process



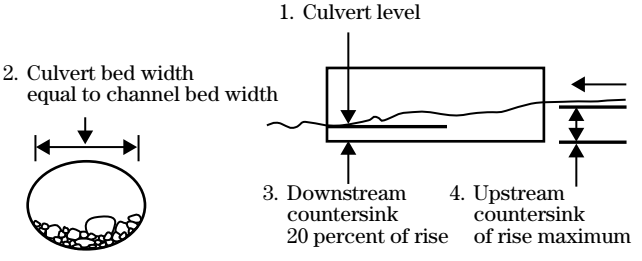
- upstream and downstream geomorphic instability (headcuts) is addressed
- adequate flood capacity exists

No-slope culverts are usually constructed where channel gradient is 3 percent or less, and culvert lengths are short to moderately long. This approach requires minimal engineering and surveying, although the following information is mandatory:

- average bankfull width of the undisturbed channel (new installations and replacements)
- average channel slope (measured along the thalweg for 20 channel widths upstream and downstream of the site, especially if a perched culvert is being replaced)
- elevation of the natural channel bed at the outlet of an existing culvert (for replacements only)
- evaluation of headcut potential immediately upstream of the crossing (for installations and replacements)
- measures to protect culvert fill during floods (riprap abutments or concrete wingwalls)

A no-slope culvert can be almost any shape; however, the streambed at the site must be relatively flat because the culvert itself will be laid level with at least 20 percent of the culvert height countersunk at the outlet and with no more than 40 percent embedded at the inlet (fig. TS14N–23 (modified from WDFW (2003)). For circular pipes, height is the diameter; for noncircular (box, pipe arch, elliptical, or bottomless) culverts, it is the rise.

Figure TS14N–23 No-slope option culvert schematic



Therefore limited to S × L less than or equal to 0.2D

The diameter (circular pipe) or span (noncircular pipe) must be a minimum of 1.25 times the average bankfull width. The average bankfull channel should be derived from three width measurements taken in naturally straight channel reaches, within 20 channel widths upstream and downstream of the crossing or nearest hydraulic control. If an existing culvert is being replaced, it is important that all stream measurements (slope, width) are collected in reaches isolated from any hydraulic or geomorphic influences attributable to the culvert or other unique channel constrictions. Often, impassable culverts cause the channel to become wider at the inlet or outlet or to become incised at the outlet. Further, geomorphic changes can occur many bankfull channel widths upstream or downstream from a culvert until the system attains an equilibrium state or encounters a stable hydraulic control (boulder debris flow, bedrock outcrop, channel-spanning dam).

For a given span, box culvert height can be variable, but a pipe arch or elliptical culvert has only one height. Also, for a given height, a box culvert has greater hydraulic capacity than either a pipe arch or a round pipe. Pipe arches may have the least amount of flow capacity and should be used only for severe restrictions on fill height. A corrugated steel pipe is the most commonly used culvert for the no-slope option because they are less expensive and easier to install than a box culvert and have more capacity than a pipe arch or an elliptical culvert. Finally, under the no-slope option, the acceptable culvert length becomes shorter as the channel becomes steeper, especially for pipe arches (table TS14N–8 (developed by Mark Schuller, NRCS WA)).

Most common round and elliptical culverts are made from corrugated steel. Larger culverts with wider and deeper corrugations produce more flow resistance (friction) and result in slower average water velocities through the culvert. Slower velocities provide better fish passage conditions and tend to balance sediment transport by retaining bed materials within the barrel. Therefore, for fish passage purposes, bigger, countersunk culverts are always better. As a rule of thumb, WDFW (2003) suggests that 36 inches should be the minimum diameter for culverts in fish-bearing streams less than 30 inches wide.

Single, large, noncircular culverts are preferred over multiple smaller pipes at fill-limited sites because they provide better fish passage, minimize debris accumulations, and are less apt to fail during flood events. A good rule of thumb for any culvert replacement or installation is to design the road crossing so that the outlet velocity is no more than 25 percent greater than what would have occurred at any given flow without a culvert in place. In addition, erosion and deposition upstream of the culvert can be minimized by designing for less than 1 foot of head loss during a 10-year flood event. This guideline minimizes backwater effects upslope of the culvert inlet and decreases downstream scour caused by head buildup at the inlet and resultant high velocity outflow at the culvert outlet (fire-hose effect).

Oversizing a culvert for high flows will improve debris passage and allow for easier maintenance inside the barrel. Care should be taken to armor the upstream and downstream abutments of a newly installed or replaced culvert. Finally, additional overflow culverts or hardened dips in the road prism may be necessary in watersheds that experience significantly high flows from episodic climatologic events (rain on melting snow or hurricane-driven rainfall).

Stream simulation option

As the name implies, the stream simulation approach is used to create or maintain natural stream processes within the barrel of a culvert. Stream simulation is based on the assumption that, if fish can easily swim through a natural channel, they should be able to swim through a manmade channel that simulates the natural channel. Generally, stream simulation culverts are best applied under the following circumstances:

Table TS14N–8 Maximum lengths for no-slope culverts

$$\text{Maximum culvert length (ft)} = 0.2H / \text{channel slope} / 12 \text{ in}$$

where:
CW = channel bed width (bankfull: ordinary high water: active channel width) (in)
H = (1.25)(CW) = diameter of round pipe or rise of pipe arch (in)
RP = round pipe diameter (in)
PA = pipe arch rise (in)
Note: Shaded areas are culverts at least 40 feet long (typical two-lane road)

Channel and culvert widths			Culvert length per channel slope (corrugated metal)							
			Round pipe				Pipe arch			
CW	RP	PA	1%	2%	3%	4%	1%	2%	3%	4%
24	30	35×24	50	25	17	13	40	20	13	10
36	45	46×36	75	38	25	19	60	30	20	15
48	60	60×46	100	50	33	25	77	38	26	19
60	75	81×59	125	63	42	31	98	49	33	25
72	90	95×67	150	75	50	38	112	56	37	28
84	105	112×75	175	88	58	44	125	63	42	31
96	120	128×83	200	100	67	50	138	69	46	35
108	135	137×87	225	113	75	56	145	73	48	36
120	150	142×91	250	125	83	63	152	76	51	38

- complex installations in moderate to high gradient channels
- longer culverts in narrow stream valleys
- culvert bed slopes that will be no more than 125 percent of the upstream channel slope
- locations where passage is required for all species (including those for which no swimming or leaping performance data exist)
- locations where ecological connectivity is of high importance or where terrestrial animals are forced to cross the road surface

Applying the stream simulation approach requires a working knowledge of the stability (both vertical and horizontal) of a prospective work site. The target stream channel must be stable within a range that can be accommodated by the planned culvert. Channels suitable for stream simulation culverts must be in equilibrium, meaning that the quantity and size of sediment delivered to the reach is roughly equivalent to the quantity and size transported out. The target stream channel must be stable within a range that can be accommodated by the planned culvert, and knowledge of vertical channel stability is essential. If the downstream channel is likely to degrade, the new culvert must be countersunk deep enough to accommodate any base level changes. Additionally, downstream grade controls are necessary to ensure further degradation will not lead to a perched culvert. Conversely, if the reach is susceptible to aggradation, the culvert must be sized to accommodate any bed-material buildup until competent streamflows occur to transport accreted sediments. If the degree of aggradation or degradation is unknown, additional baseline data collection or alternative crossings (bridges or large bottomless arches) should be strongly considered.

Stream simulation culverts are sized wider than the active channel and filled with a mix of bed material that will promote natural sediment transport dynamics through the road crossing (fig. TS14N–24). Stream simulation culverts are most often applied at slopes between 3 percent and 6 percent, although installations have occurred in gradients up to 8 percent (WDFW 2003). This method requires the largest culverts of all approaches described (minimum of 6 ft wide) and involves either placing a bottomless arch (precast concrete, structural steel plate) over the entire width of the channel or countersinking an oversized round culvert or flat-bottomed pipe (pipe arch, precast concrete). The most basic stream simulation culvert is a bottomless arch placed over an undisturbed natural channel, allowing the streambed to remain intact and decreasing chances of geomorphic instability.

Round, corrugated metal or concrete box culverts are preferred over pipe arches. A round pipe with a diameter roughly equal to a given pipe arch span affords a greater fill depth for the same bed and crown elevations, thus providing a vertical erosion buffer before the pipe bottom is exposed. Costs are very similar, but assembly and installation of a round pipe is easier than for a similarly sized pipe arch. Regardless of which culvert shape is used, it must be sufficiently wide and embedded deep enough (30 to 50% of culvert height) to allow natural stream processes (scour, deposition, and thalweg migration) to occur within the enclosed channel.

Properly embedding a stream simulation culvert raises the stream channel to the widest part of the pipe and creates deeper fill which can withstand greater vertical and lateral channel adjustments. The channel bed within a stream simulation culvert should not exceed

Figure TS14N–24 Undersized perched culvert (left) replaced with larger pipe designed using stream simulation option

a slope ratio of 1.25, defined as culvert slope divided by channel slope. Slope ratios greater than 1.25 require use of the hydraulic design method.

Stream simulation method culverts are sized according to the desired culvert bed width. Culvert bed width is the width of the bed inside the culvert, once the culvert is embedded in the channel (W_{cb}), where:

$$W_{cb} = 1.2\ (\text{bankfull width}) + 2\ \text{ft} \quad \text{(eq. TS14N–27)}$$

Notes:
Circular pipe embedded 30 percent,
culvert diameter = $1.1(W_{cb})$
Circular pipe embedded 50 percent,
culvert diameter = $1.0(W_{cb})$
Concrete box embedded 30 to 50 percent,
culvert span = $1.0(W_{cb})$

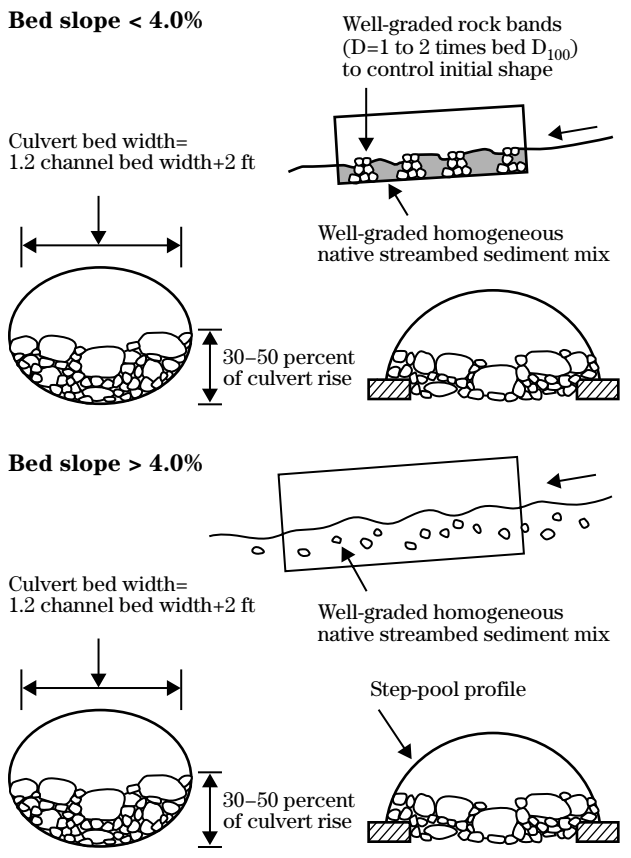
The preceding equation should be adhered to unless compelling evidence indicates otherwise. Deviations could lead to significant consequences, including inlet contraction scour, and smaller culverts will increase the chance of adverse outcomes over the design life of the crossing. If the stream is confined in a relatively narrow, stable channel, it may be possible to drop the 2-foot constant from the preceding equation. However, designing for the widest possible culvert helps ensure that terrestrial wildlife, such as turtles, small furbearers, and even deer, are also able to cross under the road.

Bed configuration within the culvert barrel should be based on channel composition in reaches adjacent to the crossing. Figure TS14N–25 illustrates two design scenarios for culvert bed composition at slopes less than and greater than about 4 percent. The 4 percent threshold is based on observations that indicate channels and culverts in streams with an energy gradient of 4 percent or less tend to have mobile beds at frequent intervals (WDFW 2003). Streams with gradients higher than 4 percent tend to have larger substrates arranged in step-pools or cascades where bed load mobility is limited except at very high flows.

The major difference between the two scenarios depicted in figure TS14N–25 (modified from WDFW 2003) concerns substrate composition and arrangement inside the culvert barrel. Culvert beds in streams with bed slopes shallower than 4 percent should be composed of native channel material with bands of larger rock to control grade and channel shape. Rock bands should be composed of well-graded rock one to two times D_{100} (the largest bed particle). The crest of each rock band should be dipped in the middle to direct the thalweg, and bands should be spaced at the lesser of five times channel width or as necessary to provide a vertical difference across adjacent crests no greater than 0.8 feet. Bands should never be closer than two channel widths or 25 feet (whichever is less) from either the inlet or outlet of the culvert.

Culvert beds in streams with bed slopes greater than 4 percent should be composed of native or engineered material arranged as a monolithic structure where the largest particles are in contact with each other.

Figure TS14N–25 Low (<4%) and high (>4%) bed slope stream simulation culvert design schematics (Note that culvert slope is similar to streambed slope)



This approach forms a network of continuous support along the whole length of the culvert and depth of the fill. No grade control within the barrel of the culvert is needed since channel beds at these gradients are very coarse and stable.

Stream simulation design culverts are easiest to install where channel slope and bed material match culvert slope and bed material. Difficulties often arise as the slope ratio approaches 1.25, and the designer must pay special attention to the sizing and arrangement of fill materials inside the culvert barrel. Under these circumstances, the designer should adhere to the following data collection, analysis, design, and construction protocol:

- Stringent assessment of site suitability including:
  - — slope
  - — channel geometry
  - — channel stability and geomorphic trajectory
  - — pebble count and subsurface sediment sampling
  - — hydraulic characteristics of design flows and depths
- Design sequence
  - — culvert bed mix composition
  - — use reference reach, incipient motion, or paleohydraulic analyses
  - — intended bed gradation and configuration (rock bands or homogeneous mix of native material)
  - — transitions to adjacent upstream and downstream channel reaches
- Construction
  - — Ensure completed project complies with design drawings by producing as-built drawings.

Hydraulic design option

Historically, the hydraulic design option (fig. TS14N–26 (modified from WDFW 2003)) has been the standard engineering method for designing fish passage at culverts. This design method requires knowledge of the swimming ability, migration timing, and size of the target species. Design criteria are usually based on the swimming abilities or size of the weakest species of fish, where known, and usually include rigorous engineering and hydrologic calculations where site-specific data are unavailable or of inadequate duration. These culverts are often the most susceptible to future longevity, function, and maintenance problems because they are generally smaller than culverts designed by either the no-slope or stream simulation options. Designers should strive to keep culverts designed under this approach as short as possible because passage criteria are usually based on the fish's prolonged swimming speed.

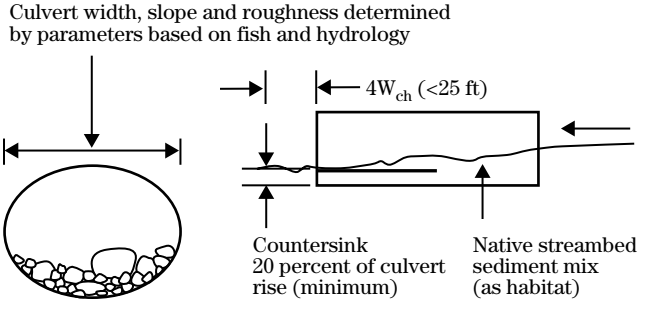
A general design sequence for developing a hydraulic design culvert is:

Step 1 Determine culvert length based on road fill geometry.

Step 2 Determine target species, sizes, migration timing, and swimming capabilities to calculate maximum barrel velocities and lengths.

Step 3 Determine design flows at which criteria from step 2 must be satisfied. For example, WDFW (2003) suggests using the 10-percent exceedance flow for adult salmonids of a target species as the high design flow. For adult passage at low flows, NOAA Fisheries Service (2000) recommends using the 50 percent annual exceedance flow or 3 cubic feet per second (whichever is greater), and for juveniles, the 95 percent annual exceedance flow or 1 cubic foot per second (whichever is greater).

Figure TS14N–26 Hydraulic design option culvert schematic



Step 4 Select culvert size, shape, roughness, and slope that satisfy barrel velocity criteria.

Step 5 Calculate or model hydraulics within the selected culvert to ensure that flow is subcritical throughout design flow range for fish passage.

Step 6 Determine channel backwater elevation at culvert outlet throughout design flow range for fish passage.

Step 7 Set culvert elevation so that low and high design flows for channel backwater are at least as high as the water surface in the culvert.

Step 8 Verify that selected culvert will provide adequate flood-flow capacity.

Step 9 If necessary, adjust channel profile to match needed culvert elevation.

The hydraulic design process might include iterations between steps 4 and 9 to arrive at the final design option that simultaneously considers the hydraulic effects of culvert size, slope, and configuration against the physiological requirements of migratory fish. A hydraulic design culvert should be countersunk at least 20 percent at the outlet and set at a shallow grade (<1%). Although stream substrates can settle out inside the culvert barrel, they are often removed by subsequent high flows. Finally, low-flow hydraulics within culvert corrugations should be considered where passage is essential for small-bodied or weak swimming fish.

Some useful hydraulic design analysis tools and reference literature are available on the Internet. FishXing (fish crossing) is a system of software and learning resources specifically aimed at the issue of fish passage within culverts. This software (including documentation) downloads an annotated bibliography of fish passage through culverts, and additional topical resources can be accessed at:

http://www.stream.fs.fed.us/fishxing/

Tide gates and floodgates

Tide gates and floodgates are usually a pipe or culvert outfitted with a gate (flap) on the outlet end that allows water to flow in only one direction. They are usually incorporated into earthen dikes and, in many cases, include pump stations. Floodgates operate in nontidal areas to prevent floodwaters from backing up into smaller tributary streams or drainage ditches. Tide gates are designed to keep saltwater out of agricultural fields, drainage ditches, and freshwater streams that flow into estuary and coastal areas. For both structures, as long as positive head remains on the upstream side, the flap remains open and allows water to drain. When the receiving water body rises (for a floodgate), or when the tide comes in (for a tide gate), the outlet flap shuts and prevents saltwater or floodwater from entering the culvert. Pumps may be necessary to move water over the dike.

Flaps (gates) can be actuated manually, mechanically, electrically, or (for most flaps) by the difference in head pressure across the culvert or pipe. Flaps can be any shape and are usually hinged to either the top or the side of the culvert outlet. Older gates are usually composed of heavy steel that do not open very wide or remain open for extended periods. These factors significantly diminish passage conditions for migratory fish. In recent years, fish passage has been improved by replacing heavy steel flaps with lightweight aluminum or plastic flaps (fig. TS14N–27). Aluminum and plastic tide and floodgates are attractive to landowners because they open under much lower head differentials, pass debris easier, and have greater conveyance capacity.

Most dikes associated with floodgates and tide gates are constructed high enough to hold back drainage until stage in the receiving water begins to drop and the flap once again opens. In coastal and estuarine settings, dike height depends on tidal elevation where the dike crosses over at the tide gate—the higher the tidal elevation, the lower the dike. Generally, a flap opens and drains the freshwater twice in a 24-hour period. For both tide gates and floodgates, suspended sediment, floating debris, and bed load tend to settle to the bottom of the channel when the upstream drainage is backwatered by a closed flap. When the flap opens again, some, but not all, of this material is carried through the culvert and downstream. Consequently, all

of these channels require periodic dredging and some form of debris removal.

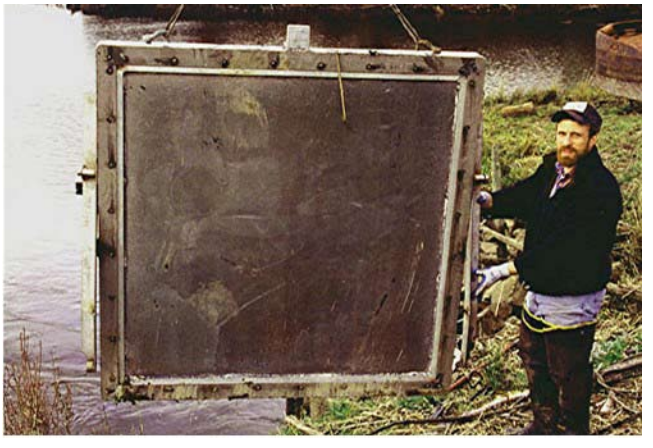
Generally, a channel reach affected by periodic tide gate or floodgate closures provides marginal fish habitat. Woody riparian vegetation is difficult to establish and maintain along these channel reaches because of frequent inundation by salt and freshwater. In addition, adjacent landowners discourage brush and tree growth that could plug the gates with woody debris. A branch that becomes wedged in the flap can allow saltwater to move far up the freshwater channel, damaging crops and affecting municipal and livestock water supplies. If a floodgate fails, acres of farmland, as well as flood plain infrastructure, can be damaged. However, the negative impacts to fish and wildlife may be minimal.

Estuarine tide gates can cause long-term negative impacts to fish and wildlife, not when they fail, but while they are functioning as designed. Estuaries are by far the most biologically productive ecosystem in the world and are defined as marine areas partly enclosed by land (a bay) that receive freshwater runoff from uplands. When this outlet is restricted, freshwater mixes with trapped saltwater and creates an area intermediate between freshwater and saltwater (brackish). Estuaries are critically important nurseries for juvenile marine fish, as well as numerous invertebrates such as crabs, lobsters, clams, and oysters. The yearly death and regeneration of marine plants, coupled with a constant flushing of detritus and other materials from incoming streams, forms the basis of this estuarine food web. In addition to the rich production of marine plants and animals, an entire community of terrestrial predators and herbivores, including waterfowl, shorebirds, raptors, furbearers, and marine mammals, creates a productive, diverse ecosystem.

Tide gates can be partial or complete barriers to migratory fish passage. However, their greatest negative impacts are related to the severe ecological changes they cause to the estuary. Tide gates and their associated dikes are often set well below the high-tide line. Consequently, they create a distinct demarcation between freshwater uplands and saltwater intertidal habitat, destroying the gradual change between freshwater and marine habitats. Plants and invertebrates found only in estuarine environments disappear and salt marshes are replaced with well-drained uplands that are usually managed for agricultural crops or pasture. Salt marshes on the outside of the dikes become mud flats. Shallow beaches, formed and maintained by the highest tides and conditioned by long exposures to air and freshwater precipitation, become uplands. Further, tide gates allow mixing of freshwater and saltwater only twice per day, rather than 24 hours per day.

Figure TS14N–27 Improved fish passage

(a) Two 1,800-lb steel tide gate flaps created a partial passage barrier for several fish species and age groups.



(b) Two 100-lb aluminum replacement flaps provide more efficient fish passage.

Juvenile marine fish, including young anadromous and catadromous fish, historically moved in and out of shallow estuary areas with the tides and were rewarded with a continuous food supply. Tide gates and saltwater dikes generally restrict these small fish to predominantly mud flats areas with less food diversity. Juvenile salmonids that are still adjusting to marine conditions cannot move between fresh and saltwater. Geomorphic and hydrologic changes in estuarine ecosystems attributable to dikes and tide gates force small fish to live in less suitable, deeper waters where longer exposures to predators can significantly increase mortality.

However, some landowners are allowing older, passive tide gates to be replaced with new self-regulating tide gates (SRT). These SRTs have various designs, but primarily function to allow not only better fish passage, but also more interchange of marine and freshwater. An SRT is equipped with a flotation device that causes the gate to open wider, more quickly, and remain open longer than conventional aluminum flap gates (fig. TS14N–28). Consequently, SRTs provide significantly improved fish passage conditions over older systems. Self-regulating tide gates also provide a range of improvements to land managers because they can be adjusted to shut completely at a preset tidal elevation to limit saltwater intrusion, or set to remain open throughout a given tidal change.

Figure TS14N–28 Self-regulating tide gate

Fishway operation and maintenance

Passage facilities must be operated and maintained properly for optimum success. Although operation and maintenance (O&M) activities vary according to the frequency with which personnel must physically visit a given facility, certain O&M elements are essential to keep a passage structure working properly. A critical O&M element is to post structural operating criteria at the facility site so that O&M personnel or the facility operator can properly adjust any controls to provide optimum passage conditions. Other essential O&M considerations include:

- specifying what entity is responsible for the daily operation and maintenance of a passage structure
- checking a passage structure at regular intervals to ensure it is operating within design criteria
- cleaning trashracks and debris collectors regularly
- adjusting gates, orifices, valves, or other control devices as needed to regulate flow and maintain a passage structure within operating criteria
- periodically checking staff gages or other flow-metering devices for accuracy
- annually inspecting passage structures for structural integrity and disrepair
- inspecting gate and valve seals for damage
- replacing worn or broken stoplogs, baffles, fins, or other structural components
- removing excessive sediment accumulations from within passage structure periodically

One of the most critical considerations regarding fishways, O&M is to ensure that excessive debris and sediment accumulations are removed as soon as possible. Sediment aggregations or debris caught in any part of the passage facility usually affects structural hydraulics and diminishes fish passage success.

Fish screen design

Fish screens preclude adult and juvenile fish from entering flow diversion structures, pump intakes, diversion channels, pipes, or penstocks. Although most screening facilities are designed to exclude juvenile fish from entrainment into diversions, pumps, or penstocks, adult screens can be constructed for the same reasons or to discourage false attraction into dead-end watercourses. Fish screens are often located at the inlet of a gravity diversion or attached directly to pump intakes. Most gravity diversion screens are configured with a bypass system to direct fish back to their stream of origin, especially if the screening structure is any distance down a diversion canal or ditch (fig. TS14N–29).

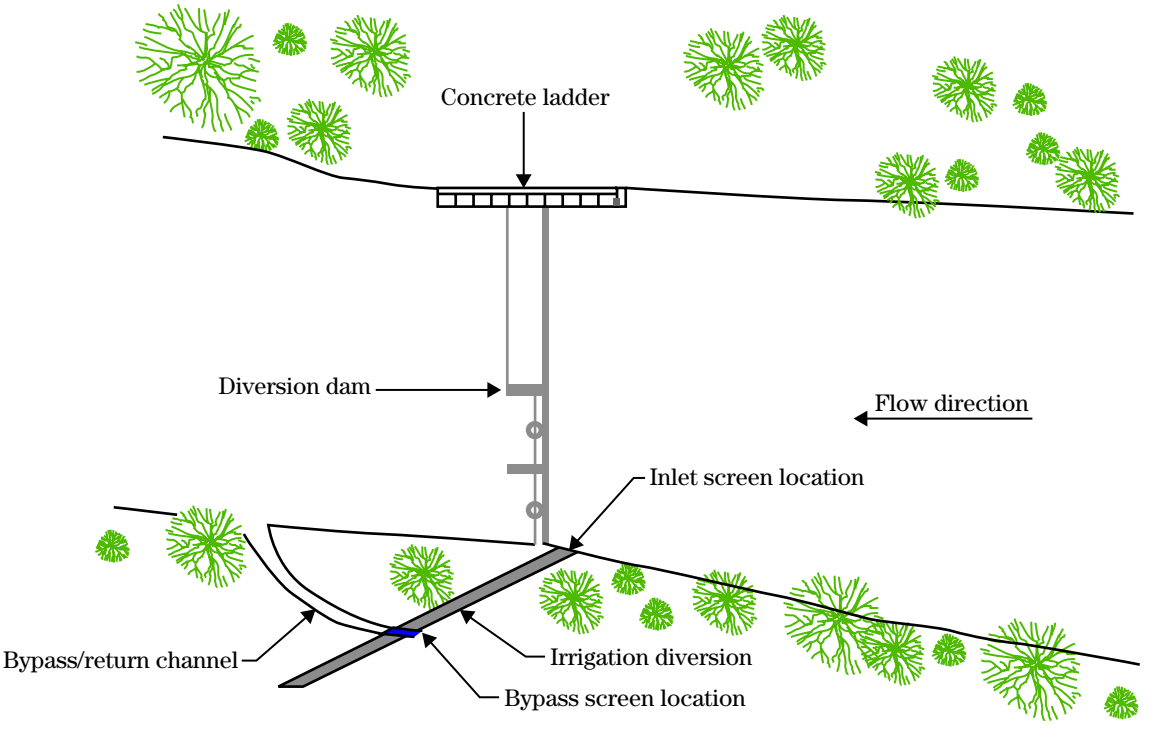
Fish screen biological design criteria

Fish screens are designed to limit mortality and injury to fish, while precluding entry into a water diversion structure or pump intake. Considerations required when designing a fish screen include (adapted from Nordlund 1997).

- preventing physical contact with the screen
- eliminating impingement onto the screen
- eliminating entrainment through the screen mesh
- maximizing bypass design to conduct fish quickly back to their stream of origin
- minimizing predation in the screen forebay, bypass pipe, and outfall
- managing for debris accumulations in bypass pipes, head gates, or trashracks
- minimizing excessive delay of fish due to poor hydraulic guidance conditions

Biological design criteria and site considerations are necessary when identifying appropriate screen location, type, and design. Required criteria include the

Figure TS14N–29 Potential fish screen configurations and locations



swimming and leaping capabilities of target species and any site-specific attractors and distractions that may affect screen function. Screen hydraulics that influence swimming capabilities of juvenile fish are sweeping and approach velocities, water temperatures, body size, swim durations, and dissolved oxygen levels. Behavioral characteristics that affect juvenile migration can vary considerably and may include desire to move downstream and reluctance to enter small bypasses. These considerations and criteria should guide biologists and designers throughout the screen design process. Once biological design criteria are identified, a designer should evaluate several hydraulic characteristics of the diversion to evaluate the potential for fish entrainment or attraction.

Certain aspects of fish screen design criteria are now well understood for some species (such as maximum approach velocity, sweeping velocities, and minimum mesh opening), but data for many species are lacking. Further, years of operation and research on general screen types has produced a set of useful design guidelines. For example, NOAA Fisheries Service (1995) has developed fish screening criteria for salmonids and suggests the following criteria for protecting juveniles:

- Approach velocity should be no more than 0.40 feet per second for active screens and 0.20 feet per second for passive screens. Active screens are juvenile fish screens equipped with automatic cleaning systems. Passive screens have no cleaning mechanisms. Approach velocity is flow diversion rate divided by effective screen area, where effective screen area is equal to maximum flow diversion divided by allowable approach velocity.
- For rotating drum screens, submergence should not exceed 85 percent nor be less than 65 percent of the screen diameter. Submergence greater than 85 percent significantly increases the possibility fish will be entrained over the screen and the chance fish will be impinged without entrainment. Submerging rotating screens less than 65 percent reduces self-cleaning capabilities.
- Screens longer than 6 feet should be angled, with sweeping velocity greater than approach velocity. Optimal sweeping velocities are between 0.8 and 3 feet per second.
- All screens should be designed to provide uniform flow across the screen surface.

Fish screen types

Several types of fish screens are available to the designer. Each functions under different flow conditions and diversion configurations and require varying operation and maintenance requirements. The following sections describe most of the typical fish screens in use today (Nordlund 1997).

Vertical fixed plate screen—The vertical fixed plate screen is a perforated metal plate acting as a physical barrier with no moving or mechanical parts. It can be used for domestic industrial water supply and agricultural irrigation diversions and can be placed along the bank of a river, thereby eliminating the need for a bypass channel. This screen type requires manual cleaning and debris removal, and many designs incorporate an accessible trash rack in front of the screen (WDFW 2000b). Design of the structure should include practical considerations for debris removal and cleaning operations (fig. TS14N–30 (WDFW 2000b)). In addition to the standard vertical alignment, these screens can be angled upstream, downstream, inclined, or declined as needed to fit site geometry and design hydraulics.

Vertical traveling screen—A vertical traveling screen is similar to the vertical fixed plate screen, except that the screen media rotates on a conveyor that automatically cleans the structure by moving debris downstream into a ditch or canal (fig. TS14N–31 (WDFW 2000b)). Vertical traveling screens are commonly used for pump intakes and can be installed in deep water. These screens require a power source (electric hookup, solar panels, paddlewheel) to rotate the screen and function properly. Vertical traveling screens can reduce the amount of manual maintenance and screen cleaning required at a facility, but these screens are more difficult to install and properly seal than fixed screen designs. Originally, panel or belt-type traveling screens were designed for debris management at pump stations. Although outfitting traveling panels with adequate screen media will protect fish from entrainment, designers should provide detailed design specifications wherever these screens are planned for installation and operation (WDFW 2000b).

Rotary drum screens—Rotary drum screens are very effective in screening juvenile fish and are perhaps the most common screening technology in use across the Pacific Northwest (fig. TS14N–32 (WDFW 2000b)). Rotary screens are usually installed at gravity diversions and have been applied singularly or in multiples in canals sized for diversion rates from a few to thou-

Figure TS14N–30 Vertical fixed plate screen

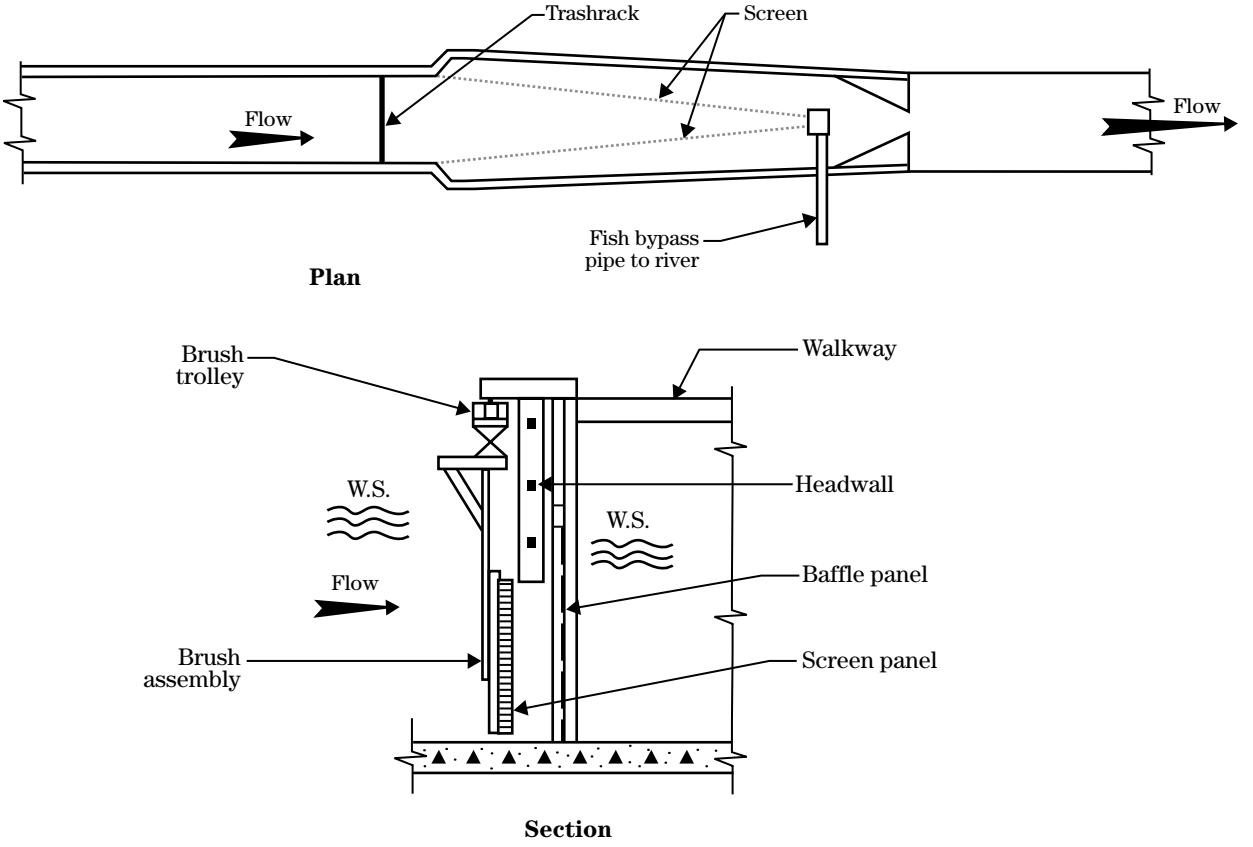
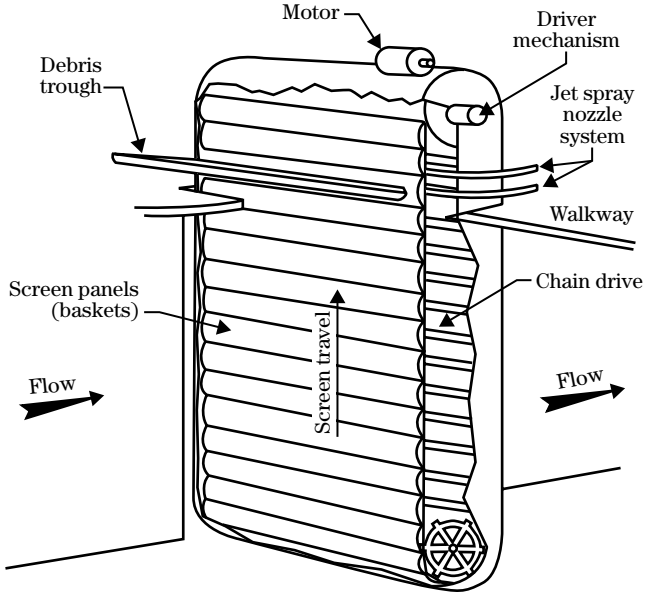


Figure TS14N–31 Vertical traveling screen



sands of cubic feet per second. These screens are comprised of a rotating cylinder with a mesh surface. As the cylinder rotates, it carries debris over the screen where it washes or falls off the backside of the screen into a diversion canal or ditch.

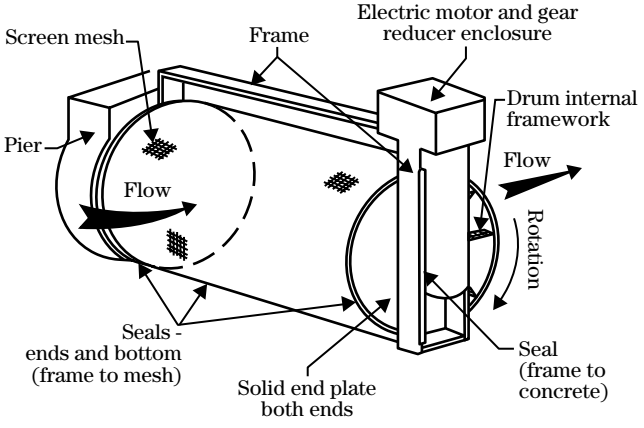
Rotary screens generally have less cleaning and maintenance requirements than a fixed or moving vertical plate screen, but side and bottom seals must be regularly inspected and replaced to ensure a fish tight facility. Each rotary drum is driven by a motor or mechanized propulsion system (commonly, a paddle wheel, turned by the flowing water), so an adjacent power supply is necessary. Drum seals, drive motors, bearings, and gears often wear out, so long-term maintenance and equipment costs can be a factor. Rotary drum screens only operate under a relatively narrow range of water surface fluctuations, so site hydraulics must be well defined prior to selecting this design option (WDFW 2000b).

Pump intake screens—Pump intake screens are designed to protect fish from being sucked into the end of an intake pipe. Pump screens are generally designed as box or cylindrical chamber composed of wire mesh (fig. TS14N–33). These screens are usually fully submerged and are cleaned with an air jet or hydraulic flushing system. Pump screens are used in a wide range of applications from small irrigation pump diversions to large scale domestic and industrial water supply intakes. A primary disadvantage of pump screen installations is that the system is completely submerged (making it harder to inspect or repair), backflush systems may not always operate as intended, and expensive cleaning and maintenance routines may be required.

Numerous manufacturers offer off-the-shelf pump intake screens for applications where intakes are sized for 5 cubic feet per second or less. As with other screening facilities, pump screens should be designed to minimize the potential for fish impingement and injury on screen media while pumps are operating.

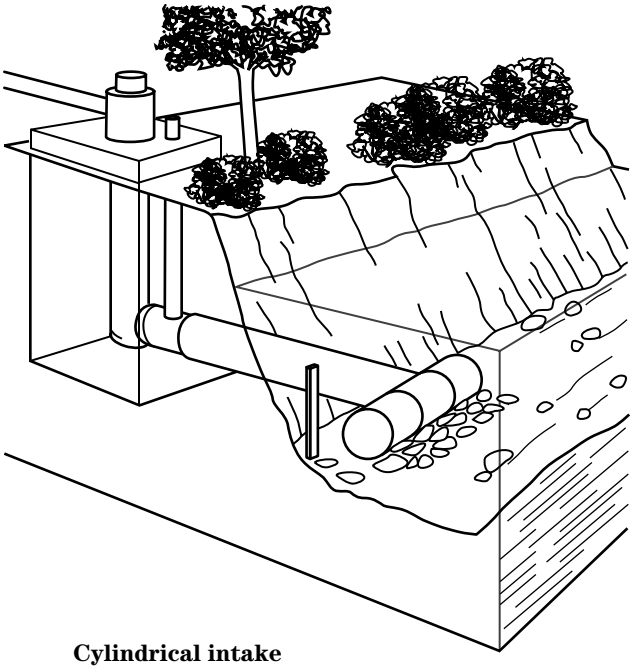
Infiltration galleries—Infiltration galleries can be used as a natural approach to pump or diversion intake design. Water infiltrates riverbed substrate, which acts as the fish screen, into a system of perfo-

Figure TS14N–32 Rotary drum screen



Isometric view

Figure TS14N–33 Pump intake screens



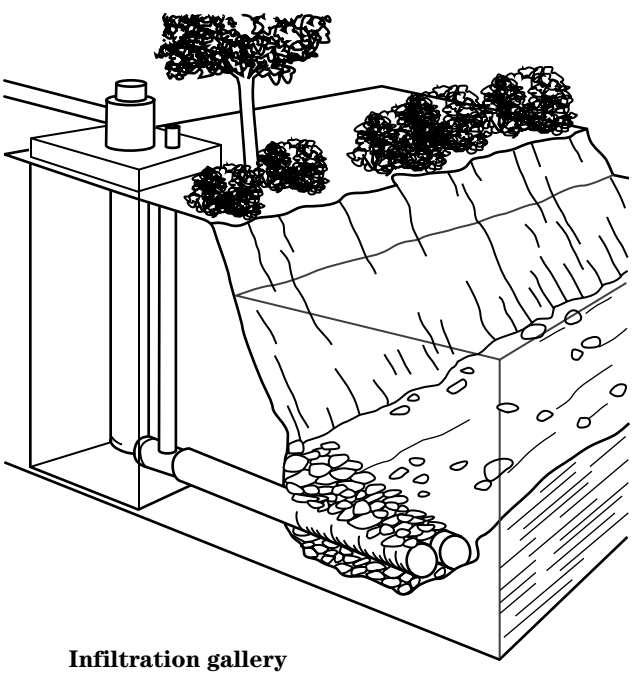
Cylindrical intake

rated pipe (fig. TS14N–34 (WDFW 2000b)). Infiltration galleries can be used for both pump and gravity diversions and have been installed for domestic, industrial, and agricultural water supplies. The key to installing an infiltration gallery is properly locating the system at a stable river section with no deposition of fines and sands to clog the filter fabric around the infiltration pipes. Typically, these locations are along higher gradient riffles or in deep pools that scour frequently. In general, infiltration galleries have higher failure risks because of clogging from debris and sediment. Further, clogged systems can incur high maintenance costs and require invasive instream construction methods to uncover system components buried beneath river substrates.

Several additional screen types are available to the designer and include modified horizontal plate and inclined plane configurations. The reader is encouraged to consult WDFW (2000b) at the following Web site for additional details and design criteria:

http://wdfw.wa.gov/hab/ahg/screen51.pdf

Figure TS14N–34 Infiltration gallery



Infiltration gallery

Screen design approach

When designing a screen, the primary objectives are to meet diversion requirements; minimize trapping, injury and harm to fish; and minimize cost, maintenance and repair. The size of the screen is ultimately a function of diversion requirements, screen opening size, and fish swimming capabilities. The following section describes a simplistic approach for screen design.

Screen mesh sizing

Screen mesh size is the opening in the screen face material (fig. TS14N–35). Screen openings can be round, square, rectangular, or any combination thereof. A variety of screen mesh materials are available to the designer and regional or local criteria may be developed for target species. The designer should consult local fish and wildlife agency for more information. Screen media should be smaller than the smallest life stage of the smallest target species present at the project site. An example of screen mesh size requirements based on testing results for screen openings for fry-sized salmonids adopted by NOAA Fisheries is presented in table TS14N–9. These openings represent the minimum screen opening dimension in the narrowest direction (Nordlund 1997; WDFW 2000b).

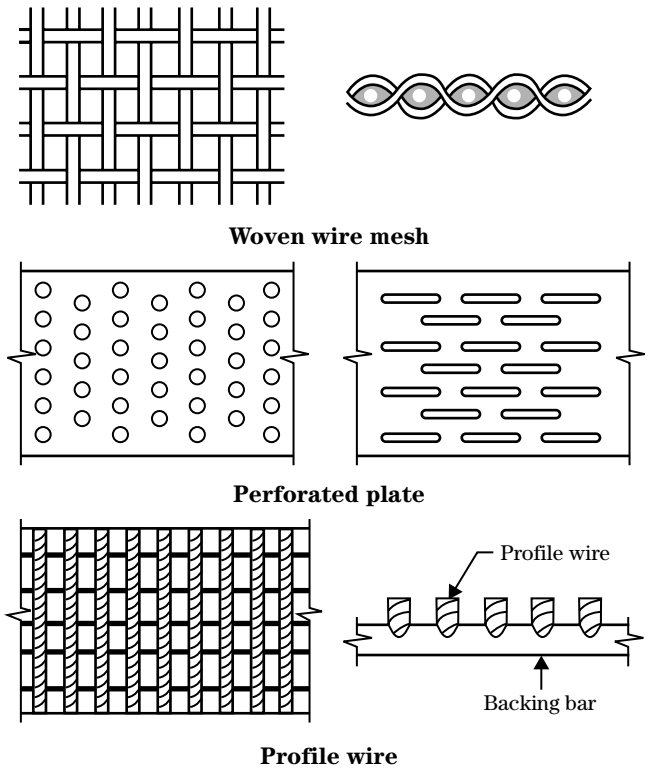
A primary screen parameter in evaluating screen design is the relationship between screen mesh size ($A_{opening}$) and the overall area of the screen (A_{screen}). For the purposes of this handbook, the ratio between the screen mesh size and the overall screen area is called the screen size ratio (eq. TS14N–28 (WDFW 2000b)). Screen size ratio varies depending on the type of screen materials specified for the project and target species.

$$A\% = \frac{A_{openings}}{A_{screen}} \qquad \text{(TS14N–28)}$$

Fish screen sizing (length and height)

The next step in designing the screen is determining the overall size (area) needed for the screen. The overall size is a function of the necessary flow diversion rate, screen size ratio, approach and sweeping velocities, and head losses through the screen. The general size of the screen is determined using the following steps.

Figure TS14N–35 Fish screen wire and mesh configurations



Step 1 Determining velocities

Approach velocity is the velocity perpendicular to the screen that can impinge fish upon the screen and injure or kill the fish. Sweep velocity is the velocity parallel to the screen that sweeps fish off the face of the screen and directs them into the bypass structure. Approach and sweeping velocities are dependent upon diversion flow rate and the angle of fish screen alignment (fig. TS14N–36 (WDFW 2000b)). The sweeping velocity should always exceed the approach velocity so that fish are swept off the face of the screen. For lateral diversions on the riverbank, the approach velocity is negligible, whereas screens in bypass channels must be placed at an angle along the channel to ensure that sweeping velocity is larger than approach velocity (eqs. TS14N–29 through TS14N–31). In short, screens must be designed for orientations that ensure sweep velocity is larger than approach velocity.

$$1 < \frac{V_{sweep}}{V_{approach}} \qquad \text{(eq. TS14N–29)}$$

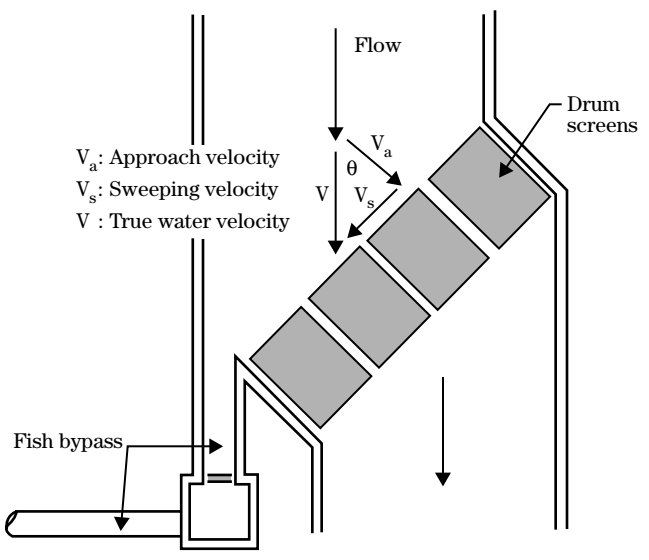
$$V_{sweep} = V \cos\theta \qquad \text{(eq. TS14N–30)}$$

$$V_{approach} = V \sin\theta \qquad \text{(eq. TS14N–31)}$$

Table TS14N–9 Examples of screen materials for fry-sized salmonids <60mm (minimum 27% open area)

Woven wire screen	3/32 in = 0.09375 in = 2.38 mm
Perforated plate screen	3/32 in = 0.09375 in = 2.38 mm
Profile bar screen	0.0689 in = 1.75 mm

Figure TS14N–36 Sweeping and approach velocities



where:

V = channel velocity (ft/s)

V_{sweep} = lateral velocity along the face of the screen (ft/s)

$V_{approach}$ = velocity along the approach angle, perpendicular velocity normal to the face of the screen (ft/s)

θ = angle between the direction of channel flow and the screen (approach velocity and the sweep velocity)

Step 2 Screen dimensions

The next step is to determine the area of screen opening to meet diversion requirements. This is an iterative process, whereby the designer estimates the area of the screen that will provide adequate flow into the diversion. Head losses are calculated and the area estimate is revised until the flow-diversion rate criteria are met. Final screen height and length are determined at the end of an iterative process to calculate flow diversion and required screen opening (eqs. TS14N–32 through TS14N–34).

The equation for flow through an orifice is the iterative design analysis.

$$Q = CA_{screen}\left(2g\Delta h\right)^{0.5} \qquad \text{(eq. TS14N–32)}$$

Head loss (Δh) can be estimated using the following equation.

$$\Delta h = \frac{1}{2g}\left(\frac{Q}{CA_{screen}}\right)^2 \qquad \text{(eq. TS14N–33)}$$

where:

Q = diversion discharge (ft^3/s)

g = gravitational acceleration (ft/s^2)

C = loss coefficient for fine-meshed screens (0.6 for fine-meshed screens (Colorado School of Mines 2004))

A_{screen} = estimated screen area to meet diversion requirements

$$A_{screen} = xyA_{\%} \qquad \text{(eq. TS14N–34)}$$

where:

x = length of screen required to meet diversion requirements (ft)

y = height of screen, plus head losses, required to meet diversion requirements (ft)

Bypass design

A fish bypass system is a flow route to transport both juvenile and adult fish from the face of a screen back to a river. Fish screens placed directly in or on the banks of a river require no bypass system. Bypass routes should transport fish back into a river or stream as quickly as possible, without injury or increased chance of mortality from predation. Major components of a fish bypass system include the entrance, transport conduit, and outfall or exit. Major design considerations for each of these components are summarized.

Bypass entrance

- Orient bypass entrances at the downstream terminus of a screen face.
- Include additional entrances if sweeping velocities will not move fish to one within 60 seconds of encountering the screen face.
- Entrance flow into the bypass system should always be 10 percent greater than the true water velocity approaching it. In screen sites with complicated or uncertain hydraulics, design bypass entrance flow to be 25 percent greater than approaching true water velocity.
- Bypass entrances should extend from the floor to the canal water surface and be a minimum of 18 inches wide (for diversions greater than 3 ft^3/s) or 12 inches wide (for diversions less than 3 ft^3/s). These widths allow schooling fish to move through without delay.

Bypass conduit

- All surfaces and joints should be smooth to reduce the risk of injury to fish.
- Maximum velocity should not exceed 30 feet per second with no hydraulic jumps. Optimum pipe velocity is around 6 to 12 feet per second.
- Flow inside the conduit should never be pressurized.
- Avoid extreme bends, and ensure that the ratio of bypass pipe centerline radius of curvature to pipe diameter (R/D) is greater than or equal to 5. Greater R/D may be required for supercritical pipe velocities.

- Size bypass conduit to minimize debris blockage, sediment deposition, and facilitate cleaning. Pipe diameter should be 24 inches or greater, but never less than 10 inches. Equip pipes longer than 150 feet with access valves.
- Never include closure valves inside a bypass pipe.
- Minimum depth of free surface flow should be at least 40 percent of bypass pipe diameter.

Bypass outfall

- Locate bypass exit where ambient river velocity is greater than 4 feet per second.
- Select an outfall location free of eddies and reverse flow to minimize predation. Require predator control systems where necessary.
- Ensure that outfall configuration will not direct fish into the river bottom.
- Design the exit so that the impact velocity of the outfall nappe or jet will not exceed 25 feet per second.

Maintenance and operation

Fish screens require periodic maintenance and cleaning to keep the diversion operational and the screen functioning properly. Trash racks, mechanical sweeper arms, manual cleaning, hydraulic flow jets and airburst features, backwash systems, and paddle wheels are used to keep the screen debris free. The designer should include either a mechanical debris removal feature or maintenance personnel for clearing the fish screen as part of a long-term operation and maintenance plan.

Example problem: Preliminary design for fish passage

An irrigation district has been informed by a local fish and wildlife agency that a 10-foot-high concrete diversion dam is a fish passage barrier. The recommendation to the district is to modify the structure to provide passage for Upper Columbia steelhead that migrate and spawn between January and May each year. The preferred plan is to use a historical overflow channel to construct a permanent, stable, natural-type step-pool rock ladder around the diversion to provide fish passage. The plan is to construct a series of 10 boulder weirs along the fish passage channel which is approximately 200 feet long (20 ft spacing). The channel will be slightly wider than the weir length.

An analysis of hydrology and hydraulics of the river and diversion dam provided stage discharge relationship information and helped identify the Q_{hf}, Q_{avg}, and Q_{hf} design discharges for the fish passage channel. Fish passage will be provided for all design flow conditions. They are:

Q_{hf} = 100 ft^3/s
Q_{avg} = 30 ft^3/s
Q_{lf} = 15 ft^3/s

The first step in sizing the weir features is to determine the general geometry. This is an iterative process. For the high-flow condition (Q_{hf}) the weir invert elevation is set such that there is 2 feet of head (H) on the fish passage diversion inlet (passage exit). Using equation TS14N–18, the length of the weir is back calculated.

$$Q = C_d\left(\sum_{i}^{N}\left(L_i - 0.1NH\right)\right)H^{1.5}$$

where:
L_i = incremental widths (ft)
N = number of contraction sides (2)
H = head on weir (2 ft)
C_d = coefficient of discharge = 3.1

English units

$$100\ \text{ft}^3/\text{s} = 3.1\left(\frac{\sqrt{\text{ft}}}{\text{s}}\right)\left[\left(L - 0.1(2)(2\ \text{ft})\right)\right](2\ \text{ft})^{1.5}$$

$$L = 12\ \text{ft}$$

For the low-flow condition (Q_{lf}), the minimum amount of head (H) on the weirs is selected as 0.7 feet per guidance on depth requirements for steelhead. The configuration of the boulder weirs is similar to figure TS14N–13.

$$15\ \text{ft}^3/\text{s} = 3.1\left(\frac{\sqrt{\text{ft}}}{\text{s}}\right)\left[\left(L - 0.1(4)(0.7\text{ft})\right)\right](0.7\ \text{ft})^{1.5}$$

$$L \approx 7\ \text{ft}$$

The total low-flow (Q_{lf}) weir length is on the order of 7 feet for two openings and four contraction walls.

The next step, using equation TS14N–3, is to check the weir velocity.

$$V_{weir} = \frac{Q}{L0.67H}$$

where:
V = velocity (ft/s)
L = weir length

$$V_{weir-hf} = \frac{100\ ft^3/s}{(12\ ft)(0.67)(2\ ft)} = 6.2\ ft/s$$

$$V_{weir-hf} = \frac{15\ ft^3/s}{(7\ ft)(0.67)(.7\ ft)} = 4.5\ ft/s$$

Velocities for the (Q_{hf}) and (Q_{lf}) are checked against the swimming abilities of steelhead and are well within range for the fish (table TS14N–10). The channel velocity is near the steelhead sustained swim speed, which indicates that there is room for design modification, if needed, including narrowing the flow channels and raising water surface and drop heights.

The next step, using equation TS14N–19, is to estimate the scour depth below the downstream bed at the toe of the rock weir. Note that the equation uses metric units.

$$Y_s = 6\Delta h^{0.25} q^{0.5} \left(\frac{Y_d}{D_{90}}\right)^{\frac{1}{3}}$$

Table TS14N–10 Steelhead swimming design criteria

Species	Sustained speed		Cruising speed		Burst speed		Maximum jump height	
	ft/s	m/s	ft/s	m/s	ft/s	m/s	ft	m
Steelhead	4.6	1.40	13.7	4.18	26.5	8.08	11.2	3.4

where:
Y_s = depth of scour (m)
Y_d = downstream depth of flow (m) = 2 ft = 0.6 m
q = unit discharge (m^3/m–s = 100 ft^3/s/12 ft = 8.3 ft^2/s = 0.8 m^2/s)
D_{90} = sediment diameter with 90% of material finer (mm = assumed 6-in material = 152.4 mm)
Δh = difference in head between upstream water surface and downstream water surface including velocity (m = 1 ft = 0.3 m)

$$Y_s = 6(0.3)^{0.25}(0.8)^{0.5}\left(\frac{0.6}{152.4}\right)^{\frac{1}{3}} = 0.6\ m = 2.0\ ft$$

The next step, using equations TS14N–20 and TS14N–21, is to check the assumed step length and drop to scour ratio to see if they are similar to those found in natural systems.

For slopes between $0.05 < \frac{h}{L} < 0.50$

$0.05 < \frac{1\ ft}{20\ ft} < 0.50$ Check, spacing could be moved closer together and shorten reach overall.

The drop to scour ratio is $1.0 < 1 + \frac{Y_s}{h} < 3.0$

$1.0 < 1 + \frac{2.0\ ft}{1\ ft} < 3.0$ Check.

The final steps are sizing the rock material for the weir crest, scour hole toe protection, and downstream tailwater area.

The weir and scour velocity (V_1) is determined using equations TS14N–25 and TS14N–24.

$$V_y = \sqrt{2g\Delta h}$$

$$V_y = \sqrt{2(32.2\ ft/s)(1\ ft)} = 8.0\ ft/s$$

$$V_1 = \sqrt{V_{weir}^{\ 2} + V_y^{\ 2}}$$

$$V_1 = \sqrt{(6.2\ ft/s)^2 + (8.0\ ft/s)^2} = 10.1\ ft/s$$

Evaluating table TS14N–10 and equation TS14N–23 indicates that the rock size on the weir should be approximately 1 to 2 feet in diameter. These values compare well with the general rule that the rock size

should be larger than the drop height of 1 foot. A selection of 2-foot-diameter material for weir and scour pool area protection is recommended. In addition, this size material should also be laid as a subsurface armor layer along the entire length of the furthest downstream step and tailwater area to provide a keystone grade control feature for the entire channel.

$$D_{50} = \left(\frac{V_1}{2.57}\right)^2$$

$$D_{50} = \left(\frac{10.1 \text{ ft/s}}{2.57}\right)^2 = 15.4 \text{ ft}$$

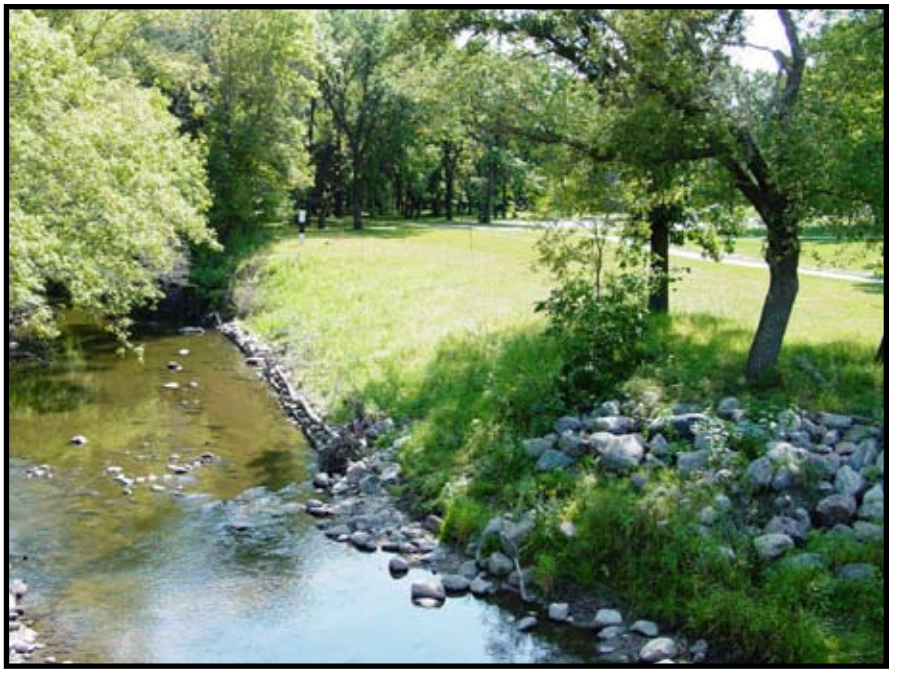
The final rock size estimate is for the bed material along the tailwater area using equation TS14N–3 in conjunction with a modified version of equation TS14N–23 and cross-checking with table TS14N–10. All approaches indicate that the tailout material should be composed of large cobbles greater than 5 inches in diameter. The following approach is appropriate if the downstream tailwater area has a flat bed slope; otherwise, the resultant velocity vector including the vertical direction must be used.

$$V_{weir} = \frac{100}{(12)(0.67)(2)} = 6.2 \text{ ft/s}$$

$$D_{50} = \left(\frac{6.2 \text{ ft/s}}{2.57}\right)^2 = 5.8 \text{ in} = 0.5 \text{ ft}$$

Technical Supplement 14O

Stream Habitat Enhancement Using LUNKERS



(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: LUNKERS provide streambank stability and edge-cover aquatic habitat.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14O

Stream Habitat Enhancement Using LUNKERS

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Technical Supplement 14O

Stream Habitat Enhancement Using LUNKERS

Purpose

Little Underwater Neighborhood Keepers Encompassing Rheotactic Salmonids (LUNKERS) are a technique to provide both streambank stability and edge cover aquatic habitat. While their use has primarily focused on providing trout habitat, they are applicable to other species, as well. This technical supplement provides guidance for the analysis, design, and installation of these structures. Particular focus is on the placement, anchoring, and finish-grading of LUNKERS structures. A step-by-step design procedure is provided.

Introduction

LUNKERS were introduced in 1982 by the Wisconsin Department of Natural Resources fisheries personnel as an alternative methodology to habitat improvement techniques then in use in trout streams. They are constructed structures that provide fish habitat in the form of edge cover. These structures resemble stout construction pallets (fig. TS14O–1). While they are often made out of wood, stone has also been used successfully. They are used in sets and are often incorporated into other bank stabilization measures. In figure TS14O–1(b), the LUNKERS are under the stone. The arrow points to one that can be seen in the picture. While their actual name is LUNKERS, the individual units are often referred to simply as a LUNKER. While their use is often associated with cold-water fisheries, they have been applied to many sites throughout the United States.

Geomorphic design considerations

For LUNKERS to function properly and provide the intended benefits, consideration must be given to their location and placement. With some exceptions, most of the criteria in use have been developed as rules of thumb by experience. The criteria that determine whether LUNKERS are an appropriate project element include:

Stream gradient and flow—LUNKERS depend on flow entering the upstream end of the structure, then sweeping beneath and through them to maintain the underbank void created by the spacer blocks. LUNKERS should not be used if the current is not fast enough or the LUNKERS cannot be constructed to produce adequate current velocities that both

Figure TS14O–1 (a) LUNKERS being installed as part of a bank stabilization project; (b) Completed LUNKERS project; LUNKERS are under the stone (*Photo courtesy of Mike Martyn, USACE*)

(a)

(b)

discourage new sediment deposition and also mobilize previously accumulated sediments. It may be necessary to place in channel boulders or use flow deflectors to force flows through the structure. These should be positioned during construction.

Channel substrate and surrounding land elevation—Traditional LUNKERS placement involves setting of the structure on a firm base to ensure stability. In low-gradient streams where post settlement alluvium is often several feet deep, LUNKERS may be installed into an excavated portion of the streambed and into the bank. However, in many stream systems, it will be necessary to install a stone base that is keyed into the bed at a depth that takes into consideration any anticipated scour. More information on stone sizing and scour calculations is provided in NEH654 TS14C and 14B, respectively. Figure TS14O–2 shows LUNKERS being installed over a rock base. The construction area had been dewatered when this photograph was taken.

Sinuosity—LUNKERS function optimally when placed on the actively eroding bank or outside bend. The lower two-thirds of a bend are preferable. This ensures that the water flow and force will always be directed into and through the structure. They generally should not be placed in straight reaches to provide overhead cover unless measures, such as low deflectors, can be used to direct water flow into the structure.

Figure TS14O–2 LUNKERS installed over a stone base

Depth—The primary building component of LUNKERS is rough lumber. The permanence of the structure is maintained by complete immersion beneath the water surface. Periodic wetting and drying will encourage premature decay and eventual failure. Installation must result in the top planks being completely submerged below the known low water stage. The minimum depth necessary is generally 1.5 feet. Grade control structures have been successfully used to maintain the necessary depth. Additional guidance for the design of grade control structures is provided in NEH654 TS14G.

Design and construction

Materials and equipment

Materials and equipment used to successfully construct and install LUNKERS vary, but some general guidelines are as follows:

LUNKERS material—The usual building component is rough-sawn and untreated wood. Oak is preferable due to its density, which contributes to the structure's ability to be handled by heavy equipment, withstand considerable weight placed on it, and resistance to rot. Newly cut (green) oak is often specified for ease of construction, since dried wood is difficult to drive nails into and may require screws.

Stone—Typically, stone is used to provide a firm base for the LUNKERS. The design and placement of stone is described in NEH654 TS14K. Since the LUNKERS typically are constructed out of wood and will float if not secured, large anchor stone is also used to hold them in place. This is typically cut stone to achieve a firm contact. This is especially important if the LUNKERS units are to be placed without dewatering the site. In addition, soil anchors can be used to provide further anchoring.

Additional bank protection—LUNKERS are rarely used by themselves. They are often part of larger bank stabilization or riparian restoration projects. These wider projects may range from grass seed and erosion fabric to more complex plantings and soil bioengineering practices. Therefore, it may be important to include these practices to achieve the ecological restoration goals for the project.

Equipment—Typical hand tools used in most LUNKERS installations include shovels, pry bars, picks, and chain saws. However, the size of the materials, as well as the grading and excavation that are typically required, necessitates the use of heavy construction equipment, as well. Typically, an excavator or a backhoe is used. Buckets are commonly modified to facilitate the placement of the LUNKERS (fig. TS14O–3). Note the forks incorporated into the bucket that keeps the LUNKERS level and the anchor stones in place.

Figure TS14O–3 LUNKERS being installed "in the wet"

Construction of LUNKERS units

The following is a step-by-step procedure for constructing a LUNKERS unit. The procedures used to construct these structures are often modified based on the available material. Figure TS14O–4 shows views of a typical completed LUNKERS structure.

Figure TS14O–5 provides conceptual plans for the construction of LUNKERS.

Step 1 Build a spacer (Note: three equal-sized spacers are needed for each LUNKERS) (fig. TS14O–5a).

Measure and cut two 6-inch lengths from the 6- by 8-inch beam to form two rectangular blocks.

Measure and cut the *bottom piece* from one of the 2- by 8-inch planks. This piece will be approximately 24 to 30 inches in length; however, the exact length of each piece depends on the recommended size of the LUNKERS.

Measure and cut the *top piece* from one of the 2- by 8-inch planks. This piece must be 50 percent longer than the bottom piece. (Example: if a 24-inch bottom piece is cut, then this piece must be 36 inches.)

Place the *bottom piece* so that one end fits flush with each of the 8-inch side of each of the rectangular blocks. Secure with two or three nails on each end. The spacer will now look like a low bench or table.

Flip the table over, and place the top piece cut above, flush to what will be the *streamside* of the LUNKERS. There will be an overhang, past the second block. (This will be the *bankside* of the LUNKERS) Secure to each block with two or three nails.

Repeat the above steps to result in three equal-sized spacers.

Step 2 Form the bottom of the LUNKERS (fig. TS14O–5b).

Bridge the three spacers across the *bottom pieces*, using a 2- by 8-inch plank. Be certain the length of the plank is flush with the *streamside* of the spacer and that the spacers are evenly placed. Secure the plank with nails above the rectangular blocks, taking care not to hit previously driven nails.

Bridge the three spacers across the bottom at the second set of blocks, using another 2- by 8-inch plank. Be certain the plank is flush with the *bankside* of the spacer. Secure each plank with nails above the rectangular blocks. The bottom of the LUNKERS is now complete.

Step 3 Assemble the LUNKERS (fig. TS14O–5c).

Form the top of the LUNKERS, flip the LUNKERS bottom over. Bridge the three spacers above the blocks with two 2- by 8-inch planks as done in step one, ignoring the overhang. Secure with nails.

Use a third 2- by 8-inch plank, placed evenly between the two top planks, and secure with two or three nails to each spacer. Depending on the size of the LUNKERS, there may or may not be spaces between the three top planks. The top of the LUNKERS is now complete.

Step 4 Prepare the LUNKERS for placement (fig. TS14O–5d).

Finish the LUNKERS according to the project's needs. In some cases, it is necessary to install two to four standard length rods to help anchor the LUNKERS into the streambed. Start by drilling two 9/16–inch holes in the top plank of the outer two spacers on the *streamside*. These holes should be placed on the inside of the streamside plank, as close to the *streamside* rectangular blocks as possible, without drilling into the blocks themselves.

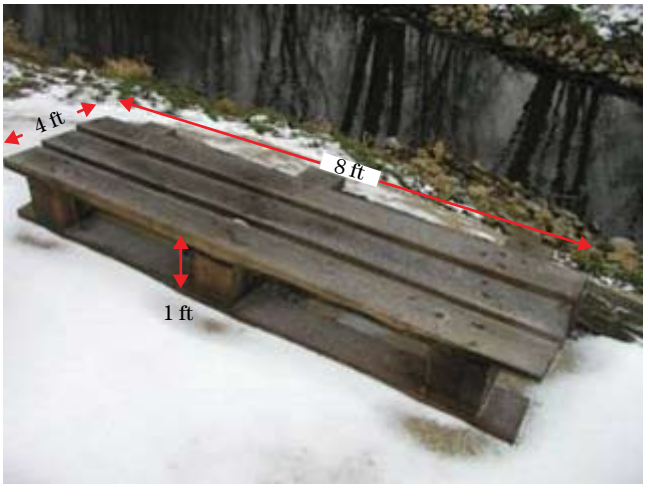
If needed, drill two 9/16-inch holes in the top plank of the outer two spacers on the *bankside*. These holes should be placed on the inside of the *bankside* plank, as close to the *bankside* rectangular blocks as possible.

If needed, drill holes to attach soil anchors.

Install the cover board on the bankside of the LUNKERS, covering the openings under the overhang. Nail in place.

Figure TS14O–4 Completed LUNKERS structure

(a) Front view



(b) Side view

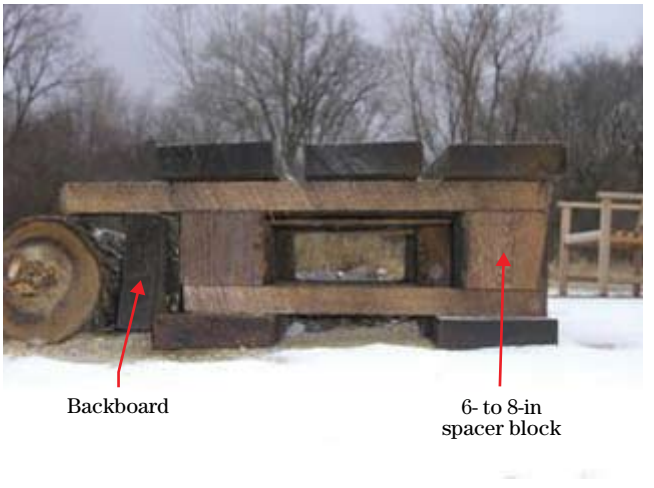
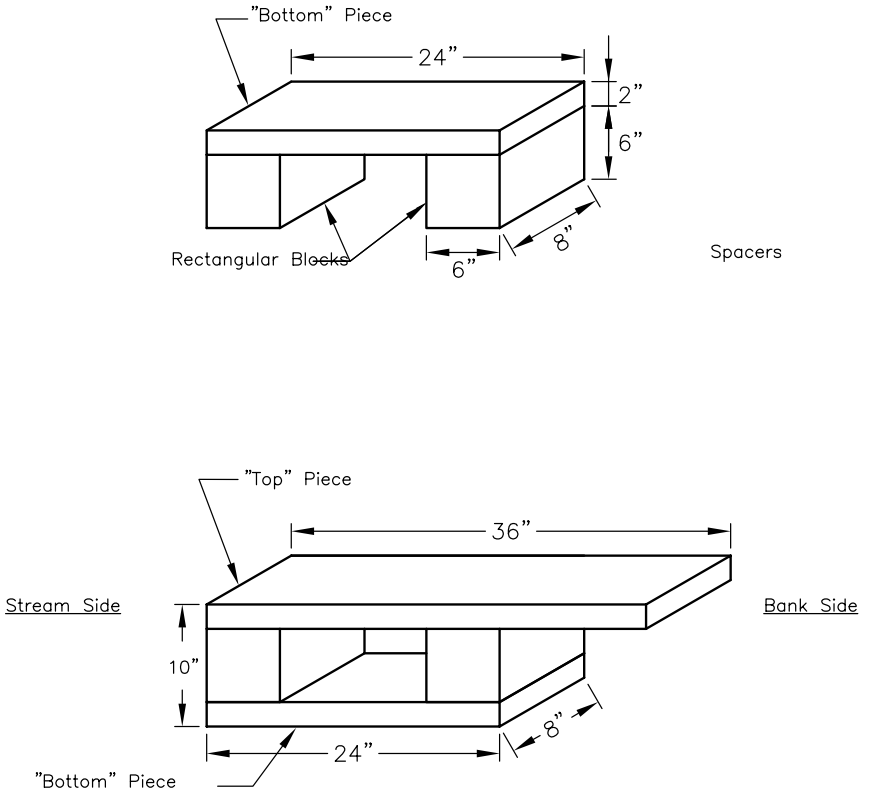


Figure TS14O–5 LUNKERS construction concept plan

(a) Step 1



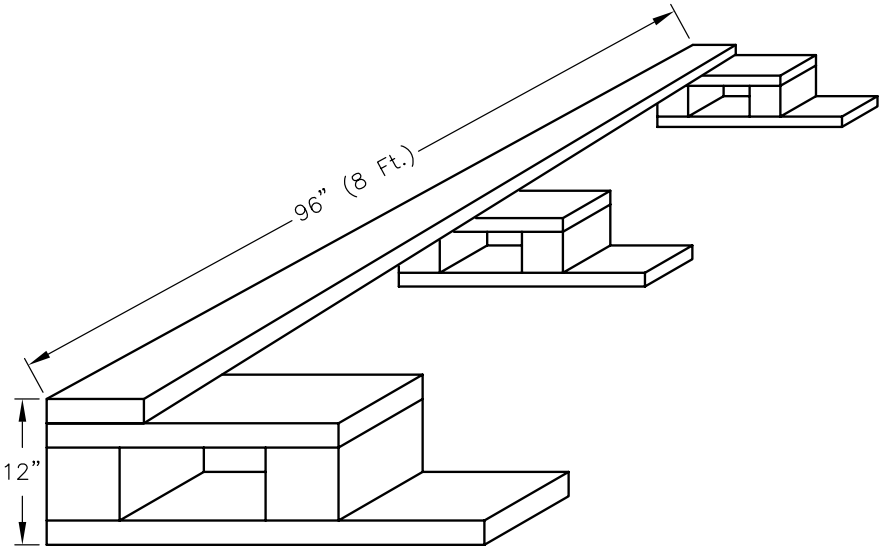
Step 1: Bulid SPACERS

Build 3 equal spacers as shown using oak or other wood that is strong and rot resistent. Use 20d nails min.

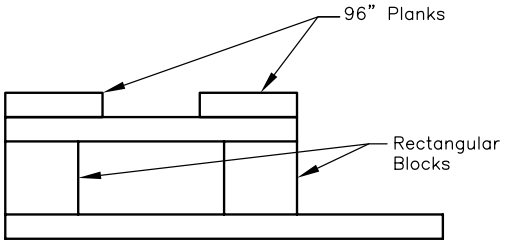
NRCS Natural Resources Conservation Service United States Department of Agriculture	LUNKER ASSEMBLY	Designed J. Fripp C. Haag, K. Worster Date 08/05 Drawn K. Miller, J. Renteria 08/05 Checked Approved	File Name Lunker–1.dwg Drawing Name Lunker Assembly 09/15/05 Sheet of

Figure TS14O–5 LUNKERS construction concept plan—Continued

(b) Step 2



Perspective Side View (One Plank in Place)



Simple Side View

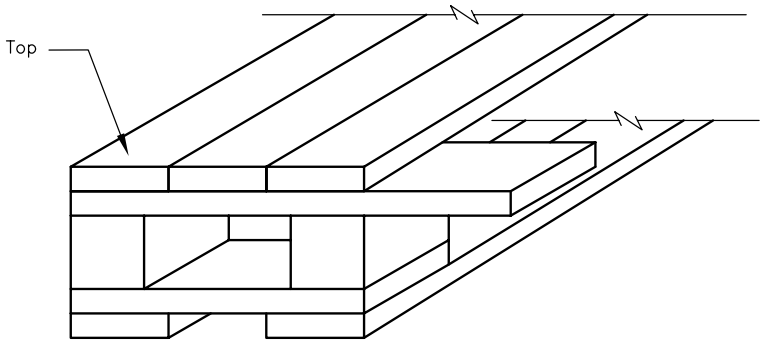
Step 2: FORM THE LUNKER BOTTOM

Bridge the spacers with two 8 ft. planks.
The spacers should be evenly spaced.
Secure each plank with 20d nails.

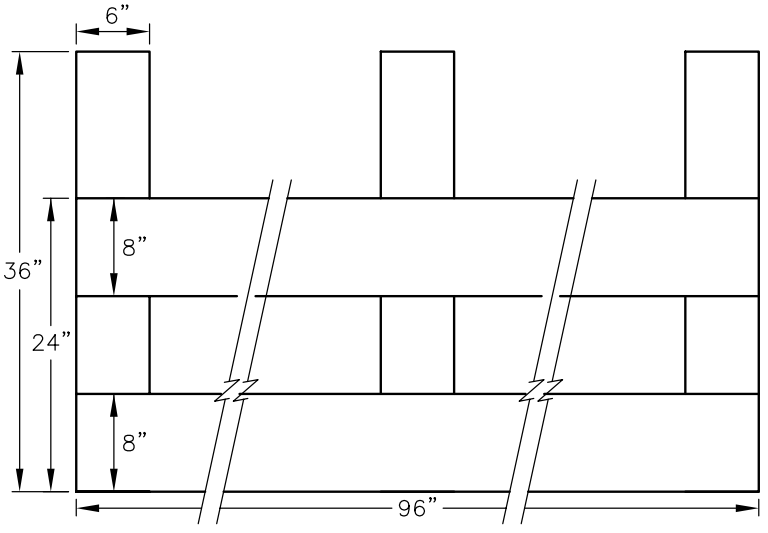
NRCS Natural Resources Conservation Service United States Department of Agriculture	LUNKER ASSEMBLY	Designed J. Fripp C. Haag, K. Worster Date 08/05 Drawn K. Miller, J. Renteria 08/05 Checked Approved	File Name Lunker–1.dwg Drawing Name Lunker Assembly Step 2 09/15/05 Sheet of

Figure TS14O–5 Step 3: LUNKERS construction concept plan—Continued

(c) Step 3



Perspective Side View (Three Planks in Place)



Top View (Two Planks in Place)

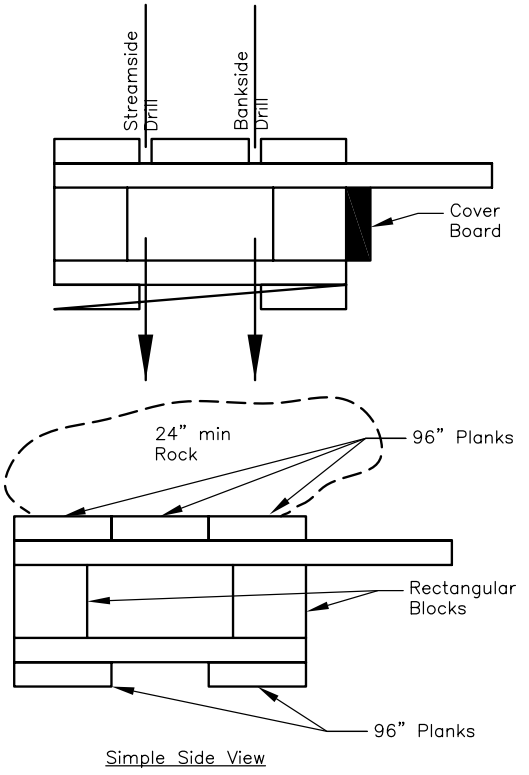
Step 3: FORM THE LUNKER TOP

Bridge the spacers with three 8 ft. planks.
Secure each plank with 20d nails.

NRCS Natural Resources Conservation Service United States Department of Agriculture	LUNKER ASSEMBLY	Designed J. Fripp C. Haag, K. Worster Date 08/05 Drawn K. Miller, J. Renteria 08/05 Checked Approved	File Name Lunker–1.dwg Drawing Name Lunker Assembly Step 3 09/15/05 Sheet of

Figure TS14O–5 Step 4: LUNKERS construction concept plan—Continued

(d) Step 4



Step 4: PREPARE LUNKERS FOR PLACEMENT

Install cover board on bank side of Lunker.
The placement of the Lunker is dependent on the base material.
Three techniques are generally used.

1. Place soil anchors attach and soil anchors to Lunker.
   or
2. Place large rock on top of Lunker structure.
   Place Lunker with rock to weight it down.
   or
3 Drill 9/16" holes through Lunkers.
   During installation insert rebar through holes to temporarily stabilize the structure as rock and soil is placed on top.

NRCS Natural Resources Conservation Service United States Department of Agriculture	LUNKER ASSEMBLY	Designed J. Fripp C. Haag, K. Worster Date 08/05 Drawn K. Miller, J. Renteria 08/05 Checked Approved	File Name Lunker–1.dwg Drawing Name Lunker Assembly Step 4 09/15/05 Sheet of

Placement of LUNKERS units

The placement of LUNKERS structures follows the preparation of the bank by removal of all undesirable trees and debris. A trench that will receive the structures must be excavated into the bed and bank. It may be necessary to place a graded stone riprap base for the LUNKERS. The design of this stone should consider anticipated scour in the bend. Additional information on the use of stone is provided in NEH654 TS14K. The resulting excavated area and base should be below the low-water level so that the structure is covered under normal low flows.

It is best for the structures to be placed so that the current will flow through them. Before excavation, designers may track the current by using a floating twig or wooden block, as it follows the targeted bank; then flag the upstream end as a guide for excavation. Boulders may be used to force flows through the LUNKERS. During construction, flexibility in placement of these boulders is essential. LUNKERS are typically used in a sequence. Three to four units is a common set. If too few are used, there may not be sufficient flows to flush sediment through the structure. If too many are linked together, the current that runs through the last (downstream) structure may lack sufficient energy to scour, so that the last structure in effect becomes a sediment trap.

Once the receiving area is prepared, the excavator lifts and delivers the LUNKERS to the trench, where it is hand placed to rest in its final orientation. The 4-foot perpendicular stringers will abut the old bank and serve as anchor points. Metal rods can be driven into the stream bottom to pin the LUNKERS to the streambed. Large stone is placed to anchor the structure. Depending on the forces expected from the stream, it may be necessary to include soil anchors to provide additional stability. More information on the design and application of soil anchors is provided in NEH654 TS14E.

A well-distributed gradation of rock riprap is then placed in the existing space from the back edge of the face rock to the preexisting old bank edge. Minimum rock fill thickness is 18 inches. This ensures that the structure will not be isolated by water backcutting during flood events. The backboard of the LUNKERS prohibits the unintentional filling of the open space by rock or sediment. Soil bioengineering practices may be installed above the structures. More information on soil bioengineering practices is provided in NEH654 TS14I.

It is optimal to place the LUNKERS without dewatering the site, or in the wet, as this allows the designer to perform small adjustments on the flow deflectors. Placement during baseflows also assures that the structure will remain underwater and not be subject to damaging wetting and drying cycles.

Conclusion

Overall, LUNKERS have been a reliable feature of many stream habitat restoration and enhancement projects for more than 20 years. Since 2000, Targeted Management Runoff (TRM) projects on the West Branch of the Sugar River in southwestern Wisconsin have resulted in the placement of 1,020 LUNKERS. No specific, comprehensive evaluation of the permanence and functional success of the LUNKERS have been conducted, but anecdotal observations taken during fisheries surveys have noted that the structures are stable and show no significant backcutting, lateral erosion, or loss of backfill in the bank.

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Gullies and Their Control

Issued August 2007

Cover photo: Gully erosion may be a significant source of sediment to the stream. Gullies may also form in the streambanks due to uncontrolled flows from the flood plain (valley trenches).

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14P

Gullies and Their Control

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Technical Supplement 14P

Gullies and Their Control

Purpose

This NEH654 TS describes the formation and control of gullies. The severity of gully development depends on a number of factors including soil type, vegetation, rainfall, concentrated flow, and human disturbances. Gullies can erode hillslopes and fill stream channels with sediment. Unchecked, they erode and deliver sediment through a variety of processes that cause loss in soil productivity, channel entrenchment, and headward expansion into the landscape. To best select a design alternative, the desired results of the landowner must be understood along with the character of the gully and its potential impacts.

Introduction

Gullies are entrenched channels extending into areas with previously undefined or weakly defined channels (Schumm, Harvey, and Watson 1984; Hansen 1995). They may occur in concentrated flow areas, such as those which can be identified on topographic maps, indicated by small contour crenulations (Hansen and Law 2004). Figures TS14P–1 and TS14P–2 are excerpts from the U.S. Geological Survey (USGS) Topographic Contour Map. Most gullies are located remotely with aerial photos or reconnaissance and evaluated in the field for activity level. Many gullies are easily seen on aerial photos.

Gullies can be thought of as extensions of a watershed drainage system up into the landscape. Under extreme conditions, gullies can expand into hillslopes extending up to the topographic watershed divide. Active gullies are recognized by headcuts (primary nickpoints), where there is an abrupt drop in elevation (figs. TS14P–3 and TS14P–4). The channel below the headcut or nickpoint is enlarged by plunging flow and erosion. Secondary nickpoints may be located downstream, showing additional base-level adjustments. Several processes are involved in nickpoint migration including cavitation, plunge development, soil piping, bank failure, and freeze-thaw cycles.

Nickpoints travel upstream as gully systems enlarge and expand in response to rainfall, runoff, and changed cover conditions. Restrictive channel materials such as bedrock or tree roots can halt or slow nickpoint migration. As figure TS14P–5 shows nickpoint migration upstream was halted by the trees in this ephemeral channel. The pastureland in the background was probably farmed. If the farmer had cut these trees, channel entrenchment and nickpoint migration would have been likely.

Figure TS14P–1 Leeds, SC: lat. 34°44' N, long. 81°25' W. Contour interval on map is 10 feet. Arrow shows contour crenulations corresponding to gully (first order stream) (fig. TS14P–2).

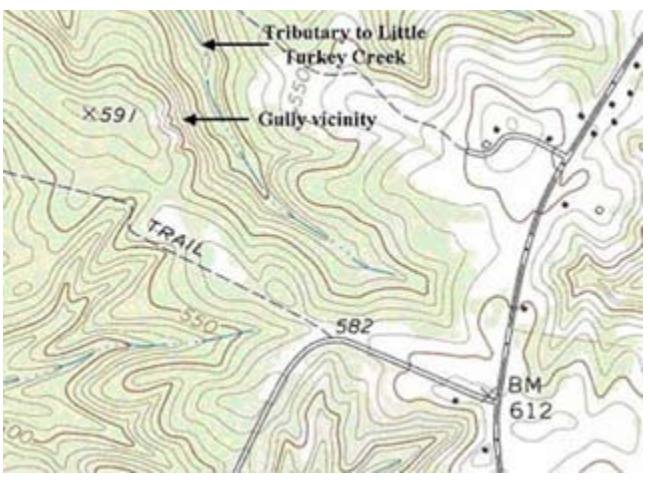
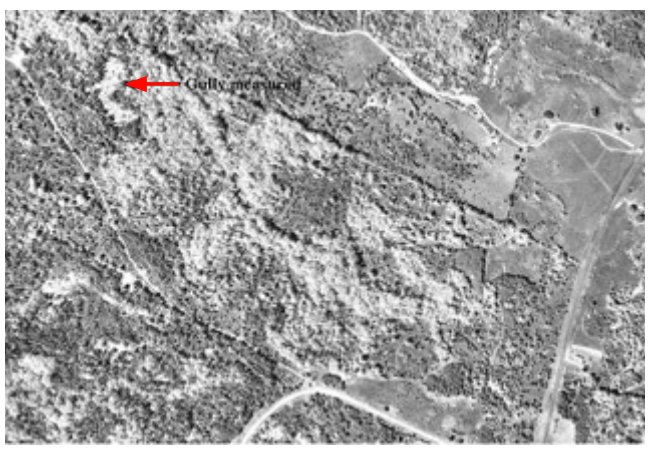


Figure TS14P–2 Aerial photo taken in 1974 highlights vicinity of gully marked in figure TS14P–1. The long gully (arrow) is about 0.25 acres and has delivered more than 53 tons of sediment over the last decade.



As the channel elevation decreases and approaches the ground water level, the length of the period of saturation or flow increases at the headcut and channel banks. The processes responsible for headcut migration vary somewhat with the position on the landscape and land use and cover conditions. Surface flow and plunge action exert pressure to undercut, widen, and collapse the nickpoint. Saturated soil is susceptible to cavitation enlargement and slope failure, winter frost heaving, slope raveling. Storm runoff causes plunge enlargement and material removal. Soil piping in certain soils has also led to gully development (Heede 1976).

As gullies expand, storm runoff increases, with declines in infiltration, ground water, baseflow, and evapotranspiration. The increased drainage density, soil exposure, erosion, and sediment delivery cause adjustments to both the adjacent uplands and downstream bottomlands. In this process of channel entrenchment, ground water tables may be lowered, resulting in declining baseflows and conversion of perennial streams to intermittent or ephemeral flow. Lands adjacent to entrenched gullies have reduced moisture available for plant growth, as the water table is lowered. Aggraded channels in downstream valleys have reduced capacity for flow, resulting in more frequent and extensive flooding. When the capacity of the channel cannot efficiently move the sediment load, braided channels develop with multiple divisions and frequent shifts. Gullies not only alter and deplete the physical character and biological capability of the affected landscape, but their downstream effects can also be pronounced.

Figure TS14P–3 Gully erosion is episodic and characterized by the incision and widening of concentrated flow channels (upslope portion), headcutting, nickpoint migration, and widening (foreground)

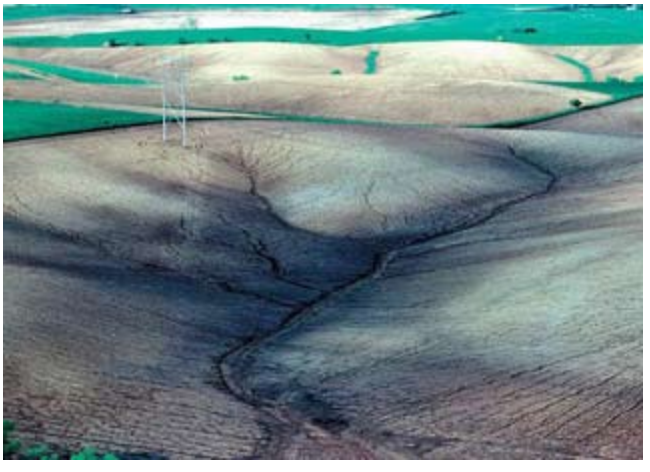
Figure TS14P–4 Active gully headcut (nickpoint) is enlarged by concentrated flow and erosion

Figure TS14P–5 Headcut halted by tree roots in Union County, SC

Classical gullies and ephemeral gullies

The previous section of gullies focused primarily on what have been termed classical gullies by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS). These are concentrated flow erosion features that are too large or deep for normal farming implements to cross and may occur on all land uses. As farming equipment grew larger and more powerful over time, the ability to farm through gullies was attained on cultivated fields. Cropped fields that would normally have developed classical gullies instead show concentrated flow features that lie in field drainageways or where opposing slopes occur. These are termed ephemeral gullies because their occurrence is ephemeral, depending on rainfall and runoff conditions, the soil's resistance to erosion, and land use and treatment. Normal farming practices may completely or partially fill in the small concentrated flow channels. Occasionally, these ephemeral gullies may recur in the same place later in the year. Figure TS14P–6 shows a cropped field with severe erosion in the foreground and some treatment of ephemeral gully erosion with grassed waterways in the background. Note that ephemeral gullies and severe sheet and rill erosion persist above some of the waterways.

Figure TS14P–6 Ephemeral gullies and severe sheet and rill erosion in unprotected northwestern IA field



Gullying processes in streams

Entrenched perennial channels may exhibit some of the same features and processes as gullies. For example, Isaacs Creek in Union County, South Carolina (fig. TS14P–7), is a small perennial gully channel (drainage area about 2.5 mi^2), which has entrenched through sediments that were deposited from past hillslope gully erosion. The channel has eroded to a massive root system that was likely part of the original valley surface. Bank undercutting and widening are a result of the grade control provided by the root system. The humid climate with well-dispersed rainfall has helped to retain the riparian character on the abandoned terrace.

The channel evolution model, developed by Schumm, Harvey and Watson in 1981, has been used to describe the evolution of gullies. Rosgen (1992, 1994) also developed criteria for gully-type channels that include low width-to-depth ratio, high entrenchment, moderate slope, and low sinuosity. Gully channels are dominated by streamflow and internal channel dynamics, with no access to a flood plain during severe storm events. Channel degradation and the headward expansion of secondary nickpoints may occur, but are generally less obvious than gully headcuts into hillslopes.

Figure TS14P–7 Perennial gully channel

Under some conditions, perennial gully-type channels may develop slowly, dependent on wearing through a resistant layer at the channel nickpoint, eventually enlarging and entrenching as the underlying erodible materials are exposed by nickpoint failure. Gully development in alluvial valley channels may occur when there is a marked change in flow, bank vegetation, channel path, base level, and/or reduction in sediment loading, causing channel degradation.

Where surface drainage enters a stream in an uncontrolled fashion, concentrated flow erosion and gravitational collapse combine to form valley trenches or sidewall gullies and sometimes edge-of-field gullies. These may also form in response to changes in base level of the receiving stream due to incision (fig. TS14P–8). Controlling this type of erosion requires conveying the water safely from the higher elevation to the stream, without eroding. Treatments include the construction of rock chutes, diversions to a safer entry level and location, or construction of pipe drops, which capture the water from the higher elevation into a pipe, which conveys the runoff water safely through the bank and into the stream.

Valleys buried by sediments from severe erosional features may change in response to implementation of erosion control. TS14P–9 shows a valley buried with 10 to 12 feet of sediments from gully erosion in the early 1900s. Since then, channel entrenchment has reached the original channel, and is marked by exposed tree stumps. Since then, channel widening into the erosive alluvial materials has widened this channel to 36 feet wide, with a bankfull depth of 2 feet (Rosgen F stream type). There are some signs that the channel is trying to build a narrow flood plain within this entrenched channel.

Channel evolution in aggrading valleys may eventually lead to braided channels (Rosgen D stream type). Channels aggrade when sediment supply is greater than the stream's ability to transport it, resulting in accumulating sediments, channel filling, shifts in channel location, and multiple channels. When sediment supply declines after channel filling periods, channel evolution may initially produce a sinuous, low-gradient channel (Rosgen E stream type). However, during bankfull and larger floods, the sinuous path may become unstable in the erosive, alluvial sediments, causing avulsions, meander cut-offs, bank failure, and development of an entrenched Rosgen G gully channel (Rosgen 1994). As temporary or permanent base levels are reached, lateral channel adjustments may occur, eventually increasing width through bank failure and sediment removal, developing into Rosgen F or C channels.

Figure TS14P–8 Gully formation adjacent to a degrading stream

Figure TS14P–9 Valley buried with sediments from gully erosion

Further adjustments in channel type may occur, and channel sections may undergo phases of aggradation, degradation, and quasi-stability (Schumm, Harvey, and Watson 1984). Channel gradient changes may signal the shifts from sediment accumulation to enlargement phases. The adjustment process of gully channels can be slow or fast, depending on factors such as storm severity and frequency, land use, soil erodibility, and vegetation.

Some of the periods of highest sediment yield in the Southwest coincide well with periods of drought, rather than periods of rainfall (Leopold and Rosgen 1992). The key difference in arid and semiarid terrain is the lack of vegetation during dry periods. Erosion is more severe than periods with higher rainfall and more vegetative cover.

Issues contributing to gully formation or enlargement

Some critical conditions (alone or in combination) could cause rejuvenation of gullies and channel forming processes with rapid erosion and expansion of the drainage network (Schumm, Harvey, and Watson 1984). Land use, soil, climate, rainfall, and hydrology are some of the leading considerations in evaluating gully processes and their control.

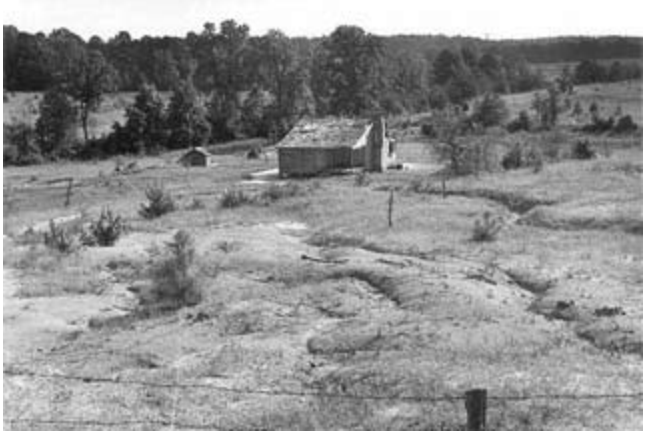
Warning signs of channel incision and degradation may include pistol butted trees, jack-strawed or heavily leaning trees, soil cracks, tree roots under tension, flow restricting geology near the surface, slopes with geologic dips nearly parallel, very steep side slopes, and/or colluvial materials (Hansen and Law 1996). Indicators of imminent channel bank collapse may or may not be present before actual failure.

Land use practices

Land use practices that alter cover, soil or hydrologic function can act as trigger mechanisms to gully formation and development. Practices that disturb and compact soils contribute to soil detachment and concentration of surface flow. Practices such as farming, road construction, grazing, mining, water transmission (ditches, trenches, terraces, or waterways), urbanization, development, and impermeable surfaces have the potential to alter conditions by changing the balance of rainfall absorption, runoff, or flow capture from adjacent areas. Figure TS14P–10 shows a slope failure likely due to excessive road drainage. Soil properties altered by years of cultivation show major reductions in subsurface soil percolation and macropore space, resulting in increased surface flow (Hoover 1949). As illustrated in figure TS14P–11, early farming and other practices had a severe effect on many landscapes, reducing soil productivity from years of surface erosion. In many instances, the soil surface is gone, exposing subsoils to continued erosion.

Figure TS14P–10 Slope failure due to excessive runoff from a road

Figure TS14P–11 Early farming and other practices had a severe effect on many landscapes



Disturbed soils lose much of their structure and have increased risk for gullying, especially if left exposed or subjected to concentrated flow for extended periods. Loss of vegetation alters the balance of rainfall, infiltration, evapotranspiration, surface cover, root strength, and runoff.

Failure to use preventative practices or heed warning signs of rill entrenchment may allow gullies to form. Although gully formation and enlargement are typically episodic, they are not instantaneous. Careful observation and treatment in the initial phases can slow or halt development (Schumm, Harvey, and Watson 1984).

Soil properties

Certain soils and landforms are especially susceptible to gully formation. Soils with weak cementation, poor consolidation, and low cohesion (alluvium, colluvium, loess, ocean, or lake deposits) have more risk. Oxisols are susceptible to gully formation due to their high degree of physical and chemical weathering. Soils that are altered by physical, chemical or biological activity may develop weaknesses that increase their erodibility. Soils sorted by water or wind often form deposits in layers of uniform-sized materials, losing much of their natural cohesive forces and erosion resistance. Soil chemical imbalances, such as high sodium absorption ratios (SAR) or low dithionite extractable iron, are more prone to be highly erodible (Heede 1976; Singer et al. 1978).

Figure TS14P–12 shows soil-piping in road fill materials within the Alkali Creek, Colorado, drainage (Heede 1982). The outlet of this failure was about 100 feet downslope. Other soil pipes found in the field survey were associated with cattle trails that concentrated flow into areas with soil cracks, dispersive soils, or other weaknesses.

Micaceous, granitic, and saprolitic soils are susceptible to gully formation. Schumm, Harvey, and Watson (1984) show that gullies are more apt to develop in landforms with comparatively steep, narrow valleys on a unit area basis. Water accumulation on low-permeability soil or hard layers, such as a fragipan or bedrock, can contribute added flow to a gully headcut or stream channel.

Climate

Certain climate, soils, and bedrock types limit the abundance and permanence of plant cover, resulting in extended periods of soil exposure. Arid and semi-arid areas or nutrient deficient or depleted soils have increased risk for gully development because the presence of plants can be tentative and fragile. In these circumstances, understanding and maintaining the natural balance can be the key to gully prevention. Native plant cover needs to be protected. If native cover is gone, restoration, replacement or other stabilization measures may be needed to control exposed soil, erosion, and the erosion caused by concentrated flow. The ability to maintain quality plant cover, infiltration, and root support across drainage areas will often prevent severe erosion and gully formation. Even minor gullying can alter soil moisture conditions and contribute to poor plant cover. To restore arid climates, mechanical means may be needed to capture and collect rainfall for plant recovery (Cohen 1994; Fayang 2004).

Hydrologic and hydraulic controls

Hydrologic alterations that modify the normal flow patterns can occur naturally or be affected by land use and treatment. Geologic controls, such as faults, may affect channel dimensions, which may confine and focus flow energy within the channel, leading to en-

Figure TS14P–12 Soil-piping in road fill materials

trenchment. Roads along streams may also impinge on the natural ability to dissipate energy on a flood plain. Wildfires on erosive soils may reduce cover or develop nonwettable soil layers that contribute to gully development. Excessive traffic and hoof shear from wildlife or cattle can develop trails that concentrate flow, eventually leading to rill and gully formation. Stream capture as a result of erosion from an adjacent area can also generate severe erosion, gully formation, or channel entrenchment. Severe storms can cause erosion and sediment delivery even from relatively small gullied areas (Hansen and Law 2004). Figure TS14P–13 shows how erosion associated with 5 inches of rainfall in 1994 resulted in 5 tons of sediment from a small gully trapped in a filter fabric fence in Chester County, South Carolina.

Gully erosion control

Some of the earliest treatments began in the 1930s with the Civilian Conservation Corps. Adjustments in treatment methods have been made in response to past failures and to take advantage of new equipment or operation developments. This NEH654 TS illustrates gully treatments from a variety of different areas. However, many approaches are site specific. Adjustments may be needed for differences in soils, rainfall, and climate conditions.

The prescription for gully treatment needs to address the severity of conditions. It should also look for specific ways to take advantage of existing conditions to produce stability. Treatments that depend on vegetation for stability are easier in moist and humid climates with productive soils, but may be more difficult to establish, develop, and maintain in nutrient-deficient soils. Vegetative-based treatments are also problematic in arid or semiarid climates.

Control of concentrated flow on forested or vegetated hillslopes is a reasonable approach, but becomes more difficult for areas affected by impermeable surfaces and larger drainage areas prone to rainfall with high intensity and duration. To achieve reliable results in both the short and long term, several of the treatment options may be combined to achieve results and reduce risk of failure.

Treatment of classical gully erosion involves protecting the headcut from further erosion, diverting overland flows away from the gully, changing land use, grading and filling in the gully, stabilizing with trees and vegetation, or by constructing a small earthen dam to impound water in the gullied area.

Treatment of ephemeral gullies includes the use of grassed waterways, terraces, diversions, water and sediment control basins (WASCoBs), accompanied by reduced tillage methods. Figures TS14P–14 and

Figure TS14P–13 Erosion resulted in sediment from a small gully trapped in filter fabric fence

Figure TS14P–14 Ephemeral gullies on cropland treated with grassed waterways, northwestern IA

TS14P–15 show ephemeral gully erosion treatments, using grassed waterways, terraces, contour farming, and conservation tillage.

Treatment options

A variety of methods have been used to stabilize, rehabilitate, or restore the effects of gully erosion (Heede 1976; Hansen 1991, 1995; Wirtz et al. 1992; Hansen and Law 1996; Law and Hansen 2004; Liu and Li 2004). Stabilization halts expansion of erosion and gully networks, reducing sediment yield, and improving water quality. Rehabilitation not only stops the erosion expansion but also improves other resources such as timber, recreation, and wildlife. Restoration is a more comprehensive effort to return the affected land to an acceptable condition for hydrology, soil productivity, and biologic response. Although a complete reversal in history of gullied terrain may not be possible, restoration is intended to stop the gully-forming cycle and produce sustainable results.

Reforestation/revegetation

Since the 1930s, establishment of forests and woodlands have been successful in reducing surface runoff and erosion associated with abandoned, cultivated, and other abused lands in many areas. Many active gully systems eventually healed themselves following replanting efforts by the Civilian Conservation Corps. This treatment was inexpensive, but the success was not immediate or assured. Issues can include a need for fertilization, mulching, and irrigation to get the plants established. Without the nutrients and moisture added by these treatments, a decade or more may be necessary for significant development of roots and litter.

Figure TS14P–15 Terraces and filter strips on cropland in Shelby County, IA

Areas with less severe erosion and gully activity are more apt to successfully respond to reforestation as the primary treatment. Severely eroded sites depleted of nutrients produce only anemic forest or grass conditions. Evidence also suggests that some of the early failures may have been the result of poor planting practices, initial seedling health, lack of follow-up checks and maintenance. Even after forest recovery, gullies remain susceptible to reactivation if conditions change.

Vegetative techniques are key elements to reversing land uses or conditions that have artificially left watersheds barren. They improve soil cover, promote water absorption, root development, and soil stability. Selection of species and accompanying treatments depend on climate and soil conditions. Some adjustment from normal practices may also be necessary to develop and maintain vegetation health. Soils with a hardpan or fragipan may need to be ripped on the contour to break up the relatively impermeable layers. On saprolite or other nutrient imbalanced soils, fertilization with lime, nitrogen, phosphorus, and potassium has enabled plant response and recovery on many sites. When needed, ample mulch is also used to help retain the seed and moisture to improve generation success.

Treatments that disturb the ground often need rapid cover and revegetation, but drainage control and other treatments can buy time and stability. Seed mixtures generally include grasses with quick response, such as brown top millet or winter wheat. The goal is to provide cover that will help control erosion, but still allow the desired perennial plants to germinate and grow. When available and appropriate to the area of treatment, native grass, forb, shrub, and tree species should be selected to provide immediate cover, long-term erosion control, and soil productivity, requiring little maintenance. When rapid cover is unlikely or added insurance is needed, straw or other mulch is recommended to hold seed on the site until moisture

and temperature conditions improve for germination. Drainage modifications may be needed to manage the flow of water into and across the site. When possible, vegetative treatments should benefit soil, water, and wildlife objectives and fit in with the natural system.

Gully plugs

A gully plug is a small earthen dam constructed at one or more locations along the gully. Branch packing and wattle check dams can also be used as gully plugs for small gullies. Information on these and other soil bioengineering applications of gully plugs is provided in NEH654 TS14I. Regardless of the size, the gully plug provides grade control and retains sediment. Larger structures will have a control structure providing overflow protection and dispensing flow to a more stable section of the gully channel (fig. TS14P–16). These structures are used to stabilize gullies with design similar to a road stream crossing with a culvert. The goal of these structures is to reduce the grade above the gully plug by storing sediment. Excess runoff is delivered to the downstream channel by a drop inlet structure typically made from a corrugated metal culvert with a spillway.

Gully plugs are normally successful when constructed below active gully networks with small drainages in gentle to moderately sloping terrain. Fill materials for gully plugs should be free of woody debris and have adequate moisture and clay components to be compacted, particularly around the drop inlet structure.

Debris dams

Debris dams were used on some of the early efforts to help stabilize gullies (fig. TS14P–17). Structures were often made of available materials such as small cedar trees piled between posts in the gully. Others consisted of chicken wire fences with cedar and other brush placed across small channels and barren lands. Straw bales with rebar support have been used with limited success. They work best when installed with the lowest areas of the installation emptying into the channel thalweg and when used as a gradient control to spread flow across a channel, rather than to act as a dam structure. If straw bales are not installed correctly, they can cause water to be diverted around them, which negates any benefit. Debris structures generally provide some short-term stability success by increasing roughness and slowing or dispersing concentrated water flow, which encourages sediment deposition. Debris dams break down after a few years and lose their effectiveness.

Figure TS14P–16 Active gully area reshaped with the flow from terraces going into a waterway, rather than adjacent gully areas

Figure TS14P–17 Early efforts to stabilize gully enlargement in the SC Piedmont used log check dams

Debris dams from rootwads or large trees provide for increased grade control and sediment retention, unless concentrated flow finds a path around or under the debris (fig. TS14P–18). Once the gully treatments are stabilized with vegetation, limited surface flows occur in these hillslope channels, and most of this sediment would be retained in place. Woody debris dams are sometimes used in gullies below land reshaping activities to remove debris from the treatment area and for added sediment retention, until erosion control measures become effective. Debris dams are not appropriate as a primary control measure on severe erosion problems. A series of small dams are more practical and stable than a few larger structures.

Commercial coir logs made from coconut fibers can provide many of the benefits of debris dams when they are installed in small gullies where flow velocities and volume are low (fig. TS14P–19). Water and finer particle sizes pass through the coir logs, but coarser materials are retained in the channel, as the flow velocities are reduced and spread across the channel surface. The limited height of these porous structures prevents plunge development.

Where sediment is stored behind them and revegetation occurs, they can be stable for extended periods. Where debris dams can be used to provide some temporary improvement, costs are usually reasonable. When used concurrently with other techniques, debris dams may provide the needed short-term stabilization until other measures become effective. Regular maintenance checks are necessary after the first major storms after construction to ensure that they are functioning properly.

Living brush barriers or hedges have been used to stabilize gullies. They are effective in increasing roughness, reducing velocities, providing grade control and capturing sediments from gully channels of low to moderate gradient. Vetivergrass (*Vetiveria zizanioides*) and switchgrass (*Panicuum virgatum*) hedges have proven useful in protecting slopes and small channels in many areas from entrenchment and in accumulating sediments (National Research Council 1993; Dabney et al. 2004).

Rock check dams

Rock check dams help stabilize eroding channels or waterways and can provide permanent channel protection (grade control); water detention, rather than retention; and allow for high water overflow. Guidance for these and other grade control structures is provided in NEH654 TS14G, Grade Stabilization Techniques. Costs, proper sizing of materials, downstream splash and plunge pool control, and the frequency or number of structures are the most critical considerations. Downstream structural controls at elevation drops are needed to dissipate flow energy and prevent plunge pool undercutting of the structure. The porous nature

Figure TS14P–18 Woody debris dam used for sediment retention in SC. Sediment was produced by a severe storm.

Figure TS14P–19 Coir logs used to complement surface stabilization and revegetation measures used in background gully channel

of the rock helps to delay flow without completely restricting, as is common with concrete or other impervious structures. This is why rock check dams function as water detention structures, as they temporarily store and attenuate the runoff water. Where materials are readily available and channel access is not a problem, costs of rock dam construction can be reasonable. Due to the extent that dams modify channel function, several small, frequent dams that act as gradient control or steps are preferable to a few larger structures. In reducing gully erosion, gradient control from these structures is probably more important than sediment storage. However, in arid locations, sediment storage and moisture retention are also important to treatment goals.

The gradient associated with stable channel conditions should be considered in the location and placement of structures. Information on calculating stable slopes is provided in NEH654.08. However, reducing grade in confined gully channels does not necessarily assure stability. In small intermittent gully channels, placing a series of low rock check dams with downstream splash aprons will allow the channel to dissipate the energy by spreading out the flow. Figure TS14P–20 shows a heavily grassed waterway where a series of loose rock check dams were used to reduce channel erosion. The rock checks provide frequent grade control and flow dispersal. The woody materials in the figure are the remains of failed log check dams that floated in onto the rocks.

Dam stability is inversely proportional to the height of the dam. The primary means to reduce gully expansion is to control the grade, provide flow dispersal at the dam, and velocity and grade loss below the dam. The next rock dam in the system should be placed just before flows concentrate, velocity accelerates, and channel degradation begins.

Numerous publications exist on rock dam construction in the West for gullies and entrenched channels (Heede 1976). Many include design and installation guidelines. When properly designed and installed, rock dams have proven effective in a variety of conditions and can withstand higher flows associated with larger channels. Their advantage in arid conditions is that they are helped by additional watershed improvements such as revegetation, but do not necessarily rely on them to be functional. Materials used in rock check dams should be well graded. In small gullies, the loose rock fills appear more natural and effective in conforming to the channel dimensions. They are structurally limited in their stable height, so plunge effects from stormflow can be more easily mitigated. Natural or planted vegetation will often add to the structural integrity.

Figure TS14P–21 shows a loose rock check dam providing sediment retention and grade control in stabilized gully channel. Reducing grazing pressure helped to restore vegetation in this area.

Figure TS14P–20 A series of loose rock check dams used to reduce channel erosion, Union County, SC

Figure TS14P–21 Retention and grade control in stabilized gully channel, Alkali Creek, CO

All check dams need a control section and an energy dissipation section below it. They need to be spaced appropriately and have flanking protection as part of the design. In practice, most all are keyed into the bed and bank. Figure TS14P–22 illustrates an approach that is simple and small scale, but has at least partially failed because of the lack of a control section and energy dissipation. The result can be that the gully erodes around or underneath the structure(s). Figure TS14P–23 shows a check dam with a clearly defined control section, energy dissipation section, and appropriate bank key-in. This structure has succeeded in stopping and controlling gully formation.

Figure TS14P–22 Gully control measure failed due to lack of control section and no energy dissipation

Figure TS14P–23 Gabion check dam provides sediment retention and grade control in stabilized gully channel, Afghanistan

Other materials are sometimes used for grade control dams when large rock materials are not available. These materials include soilcrete, aggregate-filled geotextiles, and gabions.

Rock and brush grade stabilization—Small dams constructed of alternating layers of rock and brush have been used extensively with good success in arid areas. These rock and brush structures usually are relatively short structures (less than 4 feet) and typically applied on drainage areas less than 200 acres. One of these structures under construction is shown in figure TS14P–24. As a type, rock and brush dams fall between the categories of rock dams and debris dams. Like these structures, rock and brush dams flatten the gradient and promote deposition. One of these structures that has been effectively working for some time is shown in figure TS14P–25. While relatively inexpensive, they do require design. NRCS–AZ has general design details available for the design of these structures (fig. 26).

Figure TS14P–24 Rock and brush dam under construction in AZ (*Photo courtesy of Pete Bautista*)

Soilcrete—This practice involves a mixture of approximately 50 percent soil and 50 percent concrete and is placed in cloth bags. The bags are stacked in layers across the channel, with some overlap. In figure TS14P–27, the soil and concrete were dry mixed with a cement mixer, placed in cloth bags, and eventually hardened from rain and flow. The bags are placed higher on the edges of the channel, and lower in the channel thalweg to allow for overflow. Some of these structures can remain in place in gully channels for decades, after successful revegetation controls surface flow. Higher flow rates and freeze-thaw eventually contribute to soilcrete wear and failure. Poor mixing of the materials can cause weak areas, cracking, and breakup.

Aggregate-filled geotextiles—Aggregate-filled geotextiles such as geoweb material can also be used where grade control is needed and where flows are not excessive. Some gradation in the materials is needed for proper compaction. These are best installed across the channel at the channel elevation, with no plunge downstream. They require some energy dispersal to prevent plunge development below the structure.

Figure TS14P–25 Rock and brush dam in AZ that has been working effectively (*Photo courtesy of Pete Bautista*)

Gabions—Gabions are welded wire or twisted wire baskets filled with rock. They can be used to form rock material into structural members (fig. TS14P–23). These can be very effective treatments especially in low precipitation areas. They can also be constructed relatively cheaply and can withstand high stresses. Drawbacks of gabions include that they often need extra care in installation. Settling of materials, plunge pool development, soil piping, and flow diversion around or under structures can also cause failure. NEH654 TS14K provides guidance for the design of gabion structures.

Grade control structure

A rock grade control structure can be used in channels or waterways to help disperse flow and provide grade control. Figure TS14P–28 shows a reshaped gully in Abbeville County, South Carolina, with a terrace failure that resulted in severe erosion within two waterways within a tributary of Curtail Creek. Flows from outside road and trails also contributed to the failure. The grade control structure is lined with filter cloth and filled with rock. It provides grade control and flow dissipation in each waterway, but no storage of sediment. Several of the structures were installed in each waterway. The rocked gully plug uses a culvert to protect against overflow. Filter cloth check dams were installed downstream to capture sediment. This structure was constructed with a 3-foot deep ditch across the channel lined with filter fabric and filled with materials that will not easily erode. A variety of materials can be used to construct grade control structures. These may include logs, concrete, bricks, and weathered asphalt waste.

Another example of a grade control structure is shown in figure TS14P–29. More information is available in, NRCS National Practice Standard 410, Grade Stabilization Structure. This standard applies to all types of grade stabilization structures, including a combination of earth embankments and mechanical spillways and full-flow or detention-type structures. This standard also applies to channel side-inlet structures installed to lower the water from a field elevation, a surface drain, or a waterway, to a deeper outlet channel.

Figure TS14P–26 Detail for rock and brush grade stabilization

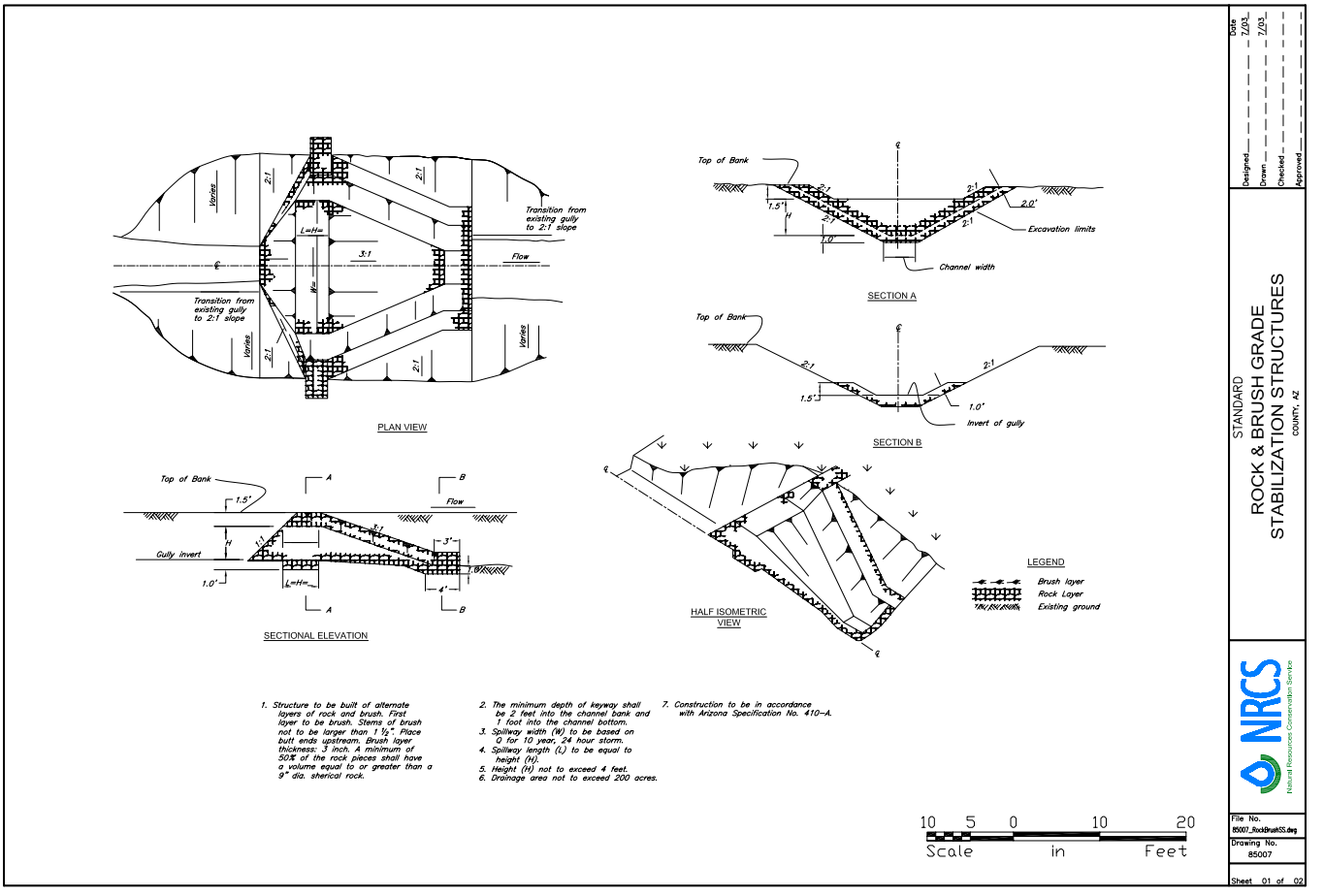


Figure TS14P–27 Soilcrete used to construct a small grade control structure

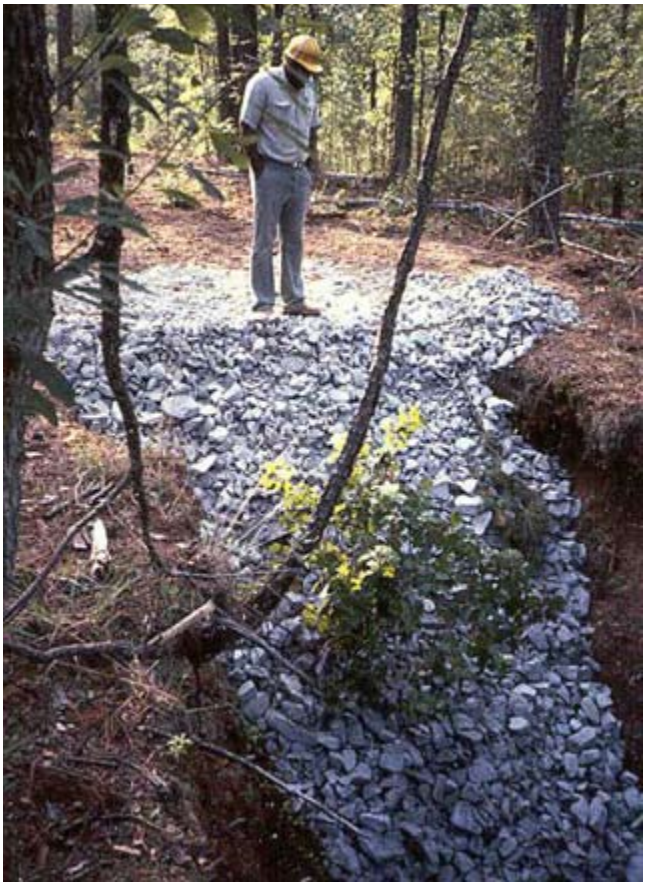
Figure TS14P–28 Reshaped gully with terrace failure, Abbeville County, SC

Figure TS14P–29 Grade stabilization structure

Rock treatments

Rock materials can effectively treat some of the moderate to severe gully erosion. Crushed rock and gravel and cobble-sized rock placed in gully nickpoints provides effective channel cover and erosion control, while allowing surface water flow to be dissipated (fig. TS14–30). The gravel allows water to disperse and lose energy as it moves past nickpoints, controlling erosion. This treatment is generally applicable to gullies with small drainage areas. Within a few months, the vegetation and other treatment measures will help control erosion and reduce the concentrated flow from the hillslopes. In humid climates, grass and tree

Figure TS14P–30 Gravel treatment in active gully head armors the surface and conveys stormflow from failed terrace and lowers it to the more stable main gully channel

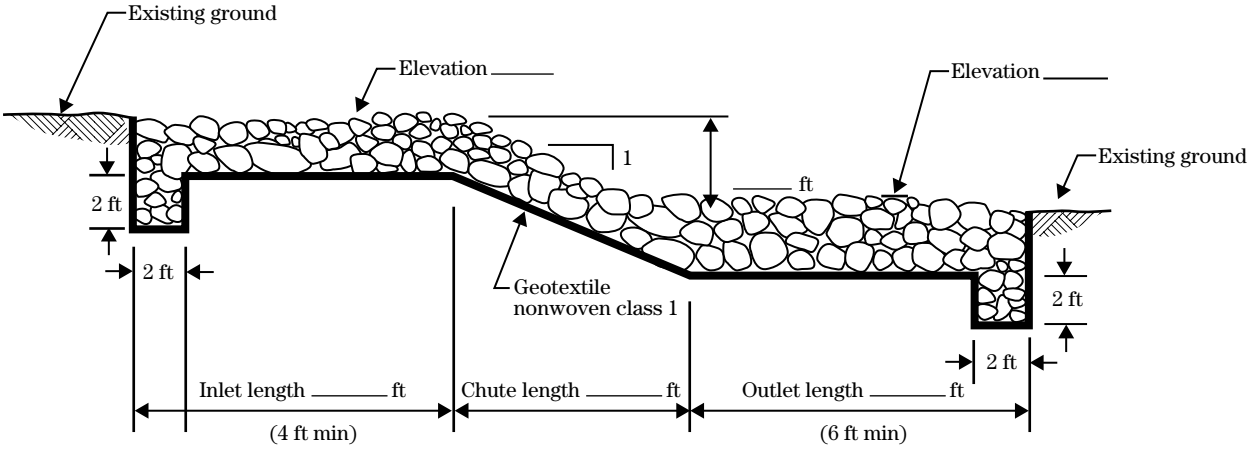


growth will develop in 3 to 5 years to control runoff. Rock placement provides immediate benefits, but can be costly when materials are not readily available and where hand labor is required.

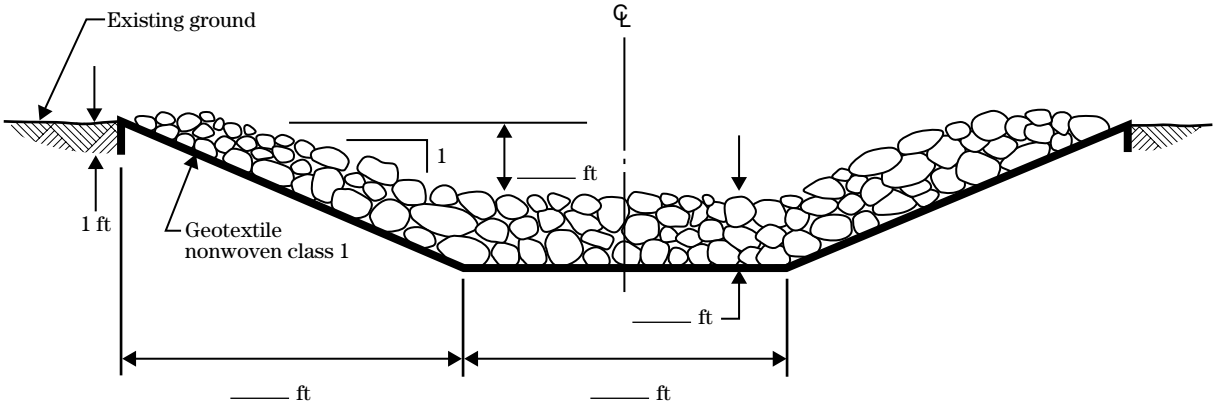
Rock chutes are grade control structures that are designed to reduce instream gradients or provide stable entry for surface water drainage to a stream. An example of a rock chute is shown in figure TS14P–31, and example design drawings for a rock chute are shown in figure TS14P–32 (NRCS–MI). Rock chutes are addressed in more detail in NEH654 TS14C.

Figure TS14P–31 Rock chute constructed to prevent gully erosion at surface water inlet to stream, SD

Figure TS14P–32 Design drawings for rock chute grade control structure



Profile along centerline



Typical section of chute

Prior to the establishment of vegetative cover, rock is occasionally used to stabilize gullies, waterways, terraces, contour trenches, or diversion ditches where excessive rainfall and erosion occur. While easily implemented, the approach shown in figure TS14P–33 may not be appropriate in an actively farmed field. In addition, it may not prove to be successful in the long term without land treatments.

Water diversion

Water diversions capture and transmit stormwater away from the gully. They are especially appropriate when roads, terraces, parking lots, urban and housing developments, or water transmission or other activities have captured or diverted flow into the gully channel. Removing other contributing flow sources can help stabilize active gullies and are used when there is a place to divert the water for dispersal and infiltration without accelerating erosion on an adjacent area. The larger the contributing drainage, the less likely this technique will be effective. Costs are generally low; however, the ability to divert water is often limited in severely gullied terrain. The diversion outlet should be on relatively flat to moderately sloping terrain with good cover and infiltration capability. Field review of soils, landforms, and bedrock will help prevent activating other types of mass erosion instability in the process of diverting flow. Stormwater storage of diverted waters in retention ponds may be appropriate for some applications.

Figure TS14P–33 Rock placed in gullies armors the flow path on steeper slopes to prevent gully advance

Terraces are a type of water diversion placed systematically along a slope to remove stormwater from a gully treatment area before surface erosion and severe rilling occur. Terraces need stable gradients (preferably 1.5% to 2%), so that runoff water will move effectively from the area without aggradation or degradation. However, gullies can be caused by abandoned or unmaintained farm terraces. Terraces and other water conveyance structures may eventually fill in, settle, or weaken from buried debris, plant roots, rodents, and other burrowing animals that can contribute to soil piping. For these reasons, hillslope terraces need periodic maintenance to function correctly. Increasing terrace size to an effective depth of 2 to 3 feet and adding some soil compaction from equipment are more costly, but will provide added insurance that minor settlement or sedimentation will not alter their function. Conversion of contributing areas to permanent forest and grasslands will often increase soil infiltration and plant transpiration to the extent that terraces are no longer needed, as surface flow is no longer present.

Waterways

Waterways are sometimes constructed to prevent gully erosion when water and associated erosive forces cannot be diverted, defused, or contained. Special treatments or designs are often necessary when dealing with highly erosive soils to ensure rapid stability through vegetation, channel armor (gravel, rock placement, erosion blankets, and hedges), geotechnical, or soil bioengineering materials. Surface water energy is dissipated through a channel-type system. Incorporating channel features consistent with other stable streams in the vicinity with similar watershed size and conditions would approach restoration goals. More information on the design of a waterway is provided in NEH654.07, in the design of threshold channels.

Rock treatments have been used in small gullies to facilitate immediate repair. Figure TS14P–34 shows a reshaped gully with rock-filled concentrated flow channel. Stormflow from the contributing 40-acre drainage area above the treatment area resulted in severe erosion of the waterway before erosion control measures were established. Soilcrete check dams were used for immediate stabilization, but the severity of the erosion made long-term stability a major concern. However, the erosion resulted in a relatively natural looking channel that entrenched several feet

into the landscape. Filling much of the storm channel with the crushed rock provided the desired immediate stability and a waterway with some sinuosity and natural function.

Geotechnical materials, fiber logs, and erosion mats may provide waterway protection and aid vegetative recovery, but costs can be excessive on large project areas. Careful attention to manufacturers' recommendations is needed when using prefabricated materials, and to ensure that the materials' performance criteria meet the needs of the application. Grass sod may be useful when placed at frequent intervals across the waterway, to cover much of the waterway to provide an immediate increase in flow resistance and channel stability. Vetivergrass and switchgrass hedges and plantings have also been reported to protect and stabilize the soil on steep banks and in concentrated flow channels (National Research Council 1993; Dabney et al. 2004). On larger channels, soil bioengineering or similar channel design techniques as described elsewhere in this handbook can help supply short and long-term stability needs.

Fertilization

The application of fertilizer to treat gullies can be an effective method to increase plant diversity and density (McKee and Law 1985; McKee, Gartner, and Law 1995). Soil nutrient testing should be conducted to verify nutrient needs. For example, in the South Carolina Piedmont, nitrogen and phosphorus have been depleted on severely eroded sites, and applications of 400 pounds per acre of pelletized 35–17–0 (%N–%P–%K) have proven invaluable to increase survival, growth, and enhancement of vegetation. Where gully and watershed conditions are beginning to stabilize, fertilization may be the only treatment needed to accelerate recovery. Marked benefits in plant growth and density have been documented for more than 5 years from a single application of slow-release fertilizer onto problem soils in the South Carolina piedmont. Other soil conditions may benefit from fertilizer application; however, the type of nutrient needs will dictate the amount and type of fertilizer used.

Figure TS14P–34 Reshaped gully with rock-filled concentrated flow channel, Union County, SC, 1987

Land reshaping

Land smoothing or reshaping has been a useful approach for active complex gully systems, but this method provides the best long-term rehabilitation or restoration, when all resources and benefits are considered. Land reshaping to smooth the surface to less than 25 percent slope is typically done with dozers, but a pan scraper with one or more dozers can efficiently move soil on larger treatment areas. Reshaped gully slopes over 25 percent are extremely difficult to stabilize with standard erosion control practices. Soil from adjacent areas is typically borrowed for use to fill in and reshape the treatment area. Adjacent soils may be highly erodible, and intensive erosion control measures may be needed to prevent gully formation.

Practices associated with land reshaping may include diversion ditches, waterways, contour trenches, subsoiling or ripping the soil (18 to 24 in deep and sometimes in two directions), liming, fertilizing, mulching, seeding, planting trees, and several years of maintenance. Primary costs are for equipment use. This method relies on several interdependent steps and timing for success. Land reshaping and smoothing may be questionable where conditions will not favor vegetative regrowth needed for erosion control. Land reshaping should not be attempted without aggressive erosion control to address the exposed soils and concentrated flow issues.

Examples of using another type of land reshaping in arid areas on a smaller scale provides individual tree catchments or scallops in the surface by digging small contour trenches, potholes, or pits for each tree (Cohen 1994; Fayang 2004). The catchments intercept much of the sparse rainfall/runoff, capturing the rainwater for each tree. As the trees develop and flourish, leaf fall and other vegetation begin to develop a soil cover. With time, infiltration improves, runoff declines, and nutrients recycle. Gradually, additional benefits accrue in enhanced water tables, reduced flooding, and increased baseflow. This treatment has also been used in the American Southwest (Schmidt 1994).

Example: Gully erosion control guidelines

Reshaping gullies can be a wasteful and ineffective experience if the measures and operations are not designed, implemented, checked, and maintained until they are stable. The following are suggestions or guidelines that have produced benefits from humid South Carolina conditions (Hansen 1995). The user is cautioned that modifications to these guidelines may be needed to account for conditions in different areas. However, many of these recommendations are broadly applicable.

- Avoid land reshaping during dry periods (hardened soils) to reduce costs.
- Treat active rill or gully erosion before it erodes into the saprolite (or other extremely erodible material) to avoid gully expansion and added costs.
- Avoid exposing or using saprolite as fill material, when possible.
- When moving large amounts of soil on relatively flat or moderate slopes, a pan earthmover is almost twice as effective as a dozer, resulting in reduced costs.
- Design drainage terraces with uniform slope gradients of 1.5 to 2 percent.
- Space terraces to control erosion and disperse surface flow into filter areas.
- Compact fill materials when needed to avoid settling or failure.
- Plant trees in soil rips to capture and retain moisture to improve survival.
- Construct terraces (about 3 ft deep) to minimize maintenance and prevent failure due to settling of soil.
- Tree survival and growth increase when planted the same year as the grasses.
- Inoculating seedlings with mycorrhizae may improve survival and growth.
- Maintain the investment by checking after major storms or other disturbances.
- Without project design and contract, equipment with operator rental may pose less risk resulting in lower cost, but with higher technical administration fees.
- Maintain access for equipment until the area is fully stabilized.
- Check terrace outlets, waterways, structures, channels, and other flow conveyances for stability, especially after major events.

Example: Conceptual treatment design—rock check dam

The following describes the site assessment and concept design for gully control at a site in Sumter National Forest of South Carolina.

Site selection—From problem site visits, gully channels with 2 percent or less gradient are generally not actively entrenching and expanding into the surrounding area. A gully section with a 6 percent gradient is actively eroding and expanding. Old agricultural terraces and uncontrolled road drainage may be adding flow to the gully, and continued nickpoint migration upstream will cause further channel instability and sediment effects.

Treatment selection—Grade control and some sediment storage with rock check dams are needed, but major plunge development in the highly erodible soils must be avoided.

Treatment concept design—Dam height is limited to 2 feet with some rock dissipation below each one. The initial grade minus the desired gradient is 4 percent. Therefore, over each 100-foot length of the gully channel, a 4-foot loss in elevation must be achieved so that the effective gradient between the check dams becomes 2 percent. For each 100-foot spacing, two 2-foot check dams will reduce the gradient by 4 percent (4 ft/100 ft), so the spacing should be at 50-foot intervals.

Conclusion

Gullies can have serious negative impacts. These impacts can range from loss of agricultural production to impacts on water supply and channel conveyance to destruction of downstream habitat. However, there are numerous land management and treatment practices that have proven effective in stopping gully formation.

Technical Supplement 14Q

Abutment Design for Small Bridges

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Design of abutments for small bridges requires geotechnical analysis.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14Q

Abutment Design for Small Bridges

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Technical Supplement 14Q

Abutment Design for Small Bridges

Purpose

This technical supplement presents a procedure for determining the ultimate and allowable bearing capacity for shallow strip footings adjacent to slopes. Additional guidelines related to scour protection and layout are also included. Structural design of bridges and footings is beyond the scope of this document

NRCS use of bridges

Bridges are installed in a variety of U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) applications including farm and rural access roads, livestock crossings, Emergency Watershed Protection (EWP) work, and recreation facilities (figs. TS14Q–1 and TS14Q–2). Bridges installed under NRCS programs are generally single-span, single-lane structures and use various simple structural systems including rail cars, steel I-beams, and timber stringers. Timber decking is often used to provide the driving surface. The entire bridge structure is normally supported by simple strip footings of timber or concrete on either abutment. The procedure presented in this technical supplement is appropriate for the design of abutments for the relatively small bridges typically constructed in NRCS work.

Small bridges may also be used to replace existing culverts that act as barriers to fish passage (figs. TS14Q–3 and TS14Q–4).

Bearing capacity

Flat ground formula

The bearing capacity of strip footings adjacent to slopes is an extension of the classical theory of bearing capacity for footings on flat ground.

The bearing capacity of a soil foundation is provided by the strength of the soil to resist shearing (sliding) along induced failure zones under and adjacent to the footing. Consequently, bearing capacity is a function of the soil's shear strength, cohesion, and frictional resistance. Further information on the selection of shear strength parameters for bearing capacity determination is included later in this technical supplement.

Bearing capacity formulas for footings on flat ground have been proposed by Terzaghi (1943) and others and have the following general form (fig. TS14Q–5 for definition sketch):

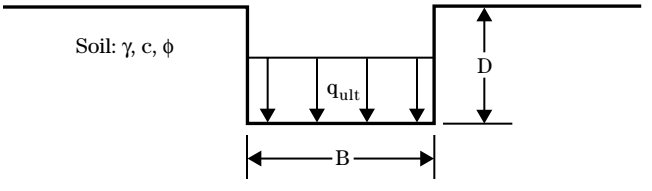
Figure TS14Q–1 Farm bridge with steel I-beam structural members and timber strip footing (*Photo courtesy of Ben Doerge*)

Figure TS14Q–2 Flood-damaged bridge in EWP program (*Photo courtesy of Ben Doerge*)

Figure TS14Q–3 Forest road culvert acting as fish passage barrier (*Photo courtesy of Christi Fisher*)

Figure TS14Q–4 Culvert replaced by bridge *(Photo courtesy of Christi Fisher)*

Figure TS14Q–5 Definition sketch—strip footing on flat ground



$$q_{ult} = c \times N_c + q \times N_q + \frac{1}{2} \times \gamma \times B \times N_\gamma$$

(eq. TS14Q–1)

where:

q_{ult} = ultimate bearing capacity (F/L^2)
c = cohesion (F/L^2)
q = γD = surcharge weight (F/L^2)
D = depth of footing below grade (L)
γ = unit weight of soil (F/L^3)
B = footing width (L)
N_c, N_q, and N_γ = bearing capacity factors related to cohesion, surcharge, and friction, respectively (unitless)

The bearing capacity factors, N_c, N_q, and N_γ, are all functions of the soil's angle of internal friction, ϕ (phi). Tables of bearing capacity factors for the flat ground case are given in tables TS14Q–1 and TS14Q–2 (Bowles 1996). In the classical treatment of bearing capacity, the foundation is assumed to consist of a single, homogeneous soil. The basic bearing capacity formula has been refined by the inclusion of modification factors to account for such variables as footing shape, inclination, eccentricity, and water table effects. See U.S. Army Corps of Engineers (USACE) EM–1110–1–1905 (1992a) and Bowles (1996) for further description of these factors.

Allowable bearing capacity

The allowable bearing capacity is determined by applying an appropriate factor of safety to the ultimate bearing capacity, q_{ult}, as determined from the bearing capacity formula. A factor of safety of 3.0 is often used with dead and live loads (USACE EM–1110–1–1905, U.S. Department of Navy Naval Facilities Engineering Command (NAVFAC) DM 7.2; American Association of State Highway and Transportation (AASHTO) Standard Specifications for Highway Bridges). Settlement of footings should also be checked to verify that excessive displacements will not occur. The treatment of bearing capacity under seismic loading is beyond the scope of this document.

Sloping ground methods

When the ground surface on one side of the footing is sloping, as in the case of a bridge abutment adjacent to a stream channel, the bearing capacity is reduced,

Table TS14Q–1 Terzaghi (1943) bearing capacity factors

ϕ, deg.	N_c	N_q	N_γ
0	5.7	1.0	0.0
5	7.3	1.6	0.5
10	9.6	2.7	1.2
15	12.9	4.4	2.5
20	17.7	7.4	5.0
25	25.1	12.7	9.7
30	37.2	22.5	19.7
35	57.8	41.4	42.4
40	95.7	81.3	100.4

Table TS14Q–2 Meyerhof (1957) and Hansen (1970) bearing capacity factors

ϕ, deg.	N_c	N_q	N_γ	
			(Meyerhof)	(Hansen)
0	5.14	1.0	0.0	0.0
5	6.49	1.6	0.1	0.1
10	8.34	2.5	0.4	0.4
15	10.97	3.9	1.1	1.2
20	14.83	6.4	2.9	2.9
25	20.71	10.7	6.8	6.8
30	30.13	18.4	15.7	15.1
35	46.10	33.3	37.2	33.9
40	40.25	64.1	93.6	79.4

compared to the flat ground case. The reduction is the result of the loss of some of the available shear resistance on the sloping side due to the shortening of the failure surface and decrease in the total weight acting on the failure surface. The bearing capacity for footings adjacent to slopes is a function of the same five variables as for footings on flat ground, with two additional variables: slope angle (β) and distance from the shoulder of the slope to the corner of the footing on the slope side (b) (fig. TS14Q–6).

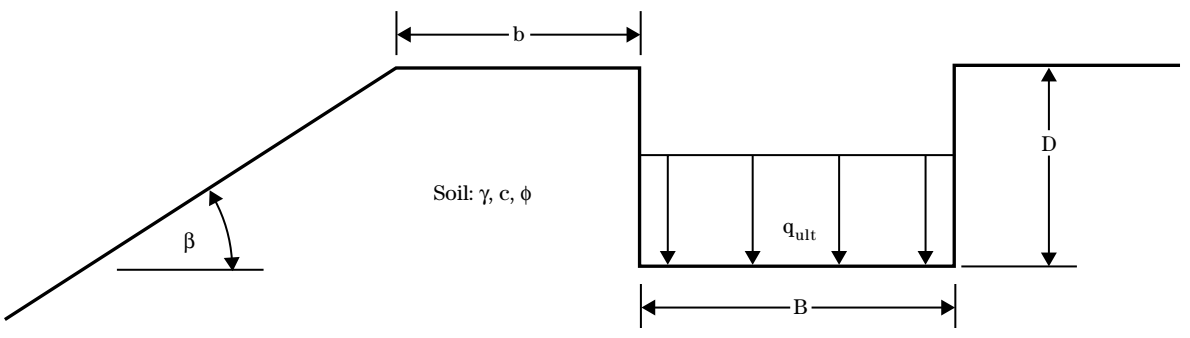
The reduction in bearing capacity for footings located adjacent to slopes has been studied extensively, using both theoretical and physical models. The effect of the slope is typically accounted for by modifying the bearing capacity factors in the traditional bearing capacity equation for flat ground. As in the classical analysis for the flat ground case, all methods for sloping ground are based on the assumption of a homogeneous foundation. A surprisingly wide variation in results may be observed between the various methods. The findings of tests on actual physical models are useful in assessing the validity of the various theoretical models.

The Meyerhof method is selected for the purposes of this technical supplement due to its long-term acceptance within the profession and its relative simplicity. Example calculations are given later.

Meyerhof method

The Meyerhof (1957) method is based on the theory of plastic equilibrium. It is the oldest method for analyzing footings adjacent to slopes and has enjoyed wide use. For example, this method is cited in NAVFAC DM 7.2

Figure TS14Q–6 Definition sketch—strip footing adjacent to a slope



and in the AASHTO Standard Specification for Highway Bridges.

In the Meyerhof (1957) method, the bearing capacity formula takes the following form:

$$q_{ult} = c \times N_{cq} + \frac{1}{2} \times \gamma \times B \times N_{\gamma q} \qquad \text{(eq. TS14Q–2)}$$

The terms N_{cq} and $N_{\gamma q}$ are modified bearing capacity factors in which the effect of surcharge is included. These factors are determined from design charts reproduced in NAVFAC DM 7.2 (fig. TS14Q–7). Procedures for analyzing rectangular, square, and circular footings, as well as water table effects, are also presented in these charts. With the Meyerhof (1957) method, foundations of both cohesive and cohesionless soils may be analyzed, and the footing may be located either on or at the top of the slope. Values of $N_{\gamma q}$ for cohesionless soils are also given in tabular form in table TS14Q–3 and for cohesive soils in table TS14Q–4.

Other methods

Many other methods exist for estimating the bearing capacity of strip footings adjacent to slopes. Bearing capacity may also be estimated using the principles of limit equilibrium or finite element analysis. The designer may wish to use more than one method and compare the results before making a final design decision. It is recommended that the bearing capacity for the flat ground case also be computed for reference as an upper bound.

Selection of shear strength parameters

Soil shear strength parameters (ϕ and c) are selected based on the assumption that footing loads may be applied rapidly. If the foundation soils may become saturated at any time during the life of the structure, then the design shear strength parameters should be selected accordingly.

For cohesive soils that develop excess pore pressure when loaded rapidly (CH, MH, and CL soils), undrained shear parameters are used. The cohesion (c) is taken to be the undrained shear strength (s_u) of the soil in a saturated condition, and an angle of internal friction (ϕ) of zero is used. The undrained shear strength may be determined from the unconsolidated-undrained (UU) shear test, the unconfined compression (q_u) test, or the vane shear test. The undrained shear strength may also be estimated from field tests or index properties. See table TS14Q–5 to estimate the undrained shear strength of saturated fine-grained soils from consistency, standard penetration test (blow count), and liquidity index.

For cohesionless, free-draining soils that are able to dissipate excess pore pressure rapidly (SW, SP, GW, GP, SM, GM, and nonplastic ML soils), the effective shear strength parameters (ϕ' and c') are used. The effective cohesion parameter (c') for cohesionless soils is either zero or very small and is normally neglected. The effective angle of internal friction (ϕ') may be determined from shear tests or may be estimated from generalized charts and tables. See table TS14Q–6 (USACE 1992a) and figure TS14Q–8 (NAVFAC 1982b) for charts to estimate the effective angle of internal friction for sands and gravels.

A qualified soils engineer should be consulted when selecting shear strength parameters for design.

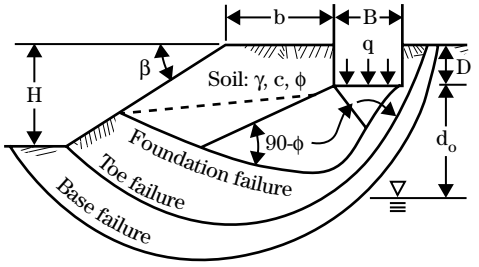
Other considerations

Bridge abutments consisting of soil or other erodible material must be protected against the effects of scour. Either the footing should be embedded below the maximum anticipated scour depth or adequate scour protection must be provided. The embedment approach is generally not applicable with the shallow strip footings normally used with NRCS-designed bridges. Therefore, scour protection, such as rock riprap or gabions, should be provided, as necessary, to prevent erosion of the slope and possible undermining of the footings.

The U.S. Forest Service recommends that bridges with shallow strip footings be used primarily in stream channels that are straight and stable, have low scour potential, and will not accumulate significant debris or ice. Any additional antiscour measures needed to ensure the long-term integrity of the bridge abutments should be incorporated into the design (McClelland 1999).

Figure TS14Q–7 Meyerhof method design charts: ultimate bearing capacity for shallow footing placed on or near a slope

Case I: Continuous footing at top of slope



Water at $d_o \geq B$

$$q_{ult} = cN_{cq} + \gamma_T \frac{B}{2} N_{\gamma q} \quad (a)$$

Water at ground surface

$$q_{ult} = cN_{cq} + \gamma_{sub} \frac{B}{2} N_{\gamma q} \quad (b)$$

If $B \leq H$:
Obtain N_{cq} from figure 4b for case I with N_o=0.
Interpolate for values of $0 < D/B < 1$
Interpolate q_{u1t} between eq (a) and (b) for water at intermediate level between ground surface and d_o=B.

If $B > H$:
Obtain N_{cq} from figure 4b for case I with stability number

$$N_o = \frac{\gamma H}{c}$$

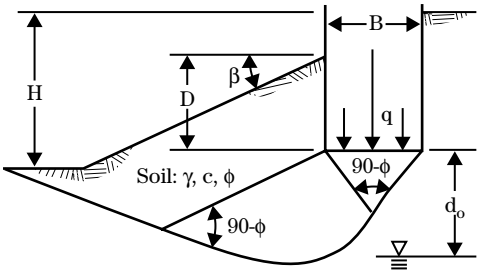
Interpolate for values $0 < D/B < 1$ for $0 < N_o < 1$. If $N_o \geq 1$, stability of slope controls ultimate bearing pressure.
Interpolate q_{u1t} between eq (a) and (b) for water intermediate level between ground surface and d_o = B. for water at ground surface and sudden drawdown: Substitute ϕ' for ϕ in eq (b)

$$\phi' = \tan^{-1}\left(\frac{\gamma_{sub}}{\gamma_T}\tan\phi\right)$$

Cohesive soil ($\phi = 0$)
Substitute in eq (a) and (b) D for B/2 and $N_{\gamma q} = 1$.
Rectangular, square, or circular footing:

$$q_{u1t} = \left[q_{u1t} \begin{array}{l}\text{for continuous footing}\\ \text{as given above}\end{array} \right] \times \left[\frac{q_{u1t}\text{ for finite footing}}{q_{u1t}\text{ for continuous footing}} \right]$$

Case II: Continuous footings on slope



Same criteria as for case I except that N_{cq} and $N_{\gamma q}$ are obtained from diagrams for case II

Figure TS14Q–7 Meyerhof method design charts: bearing capacity for shallow footing placed on or near a slope—Continued

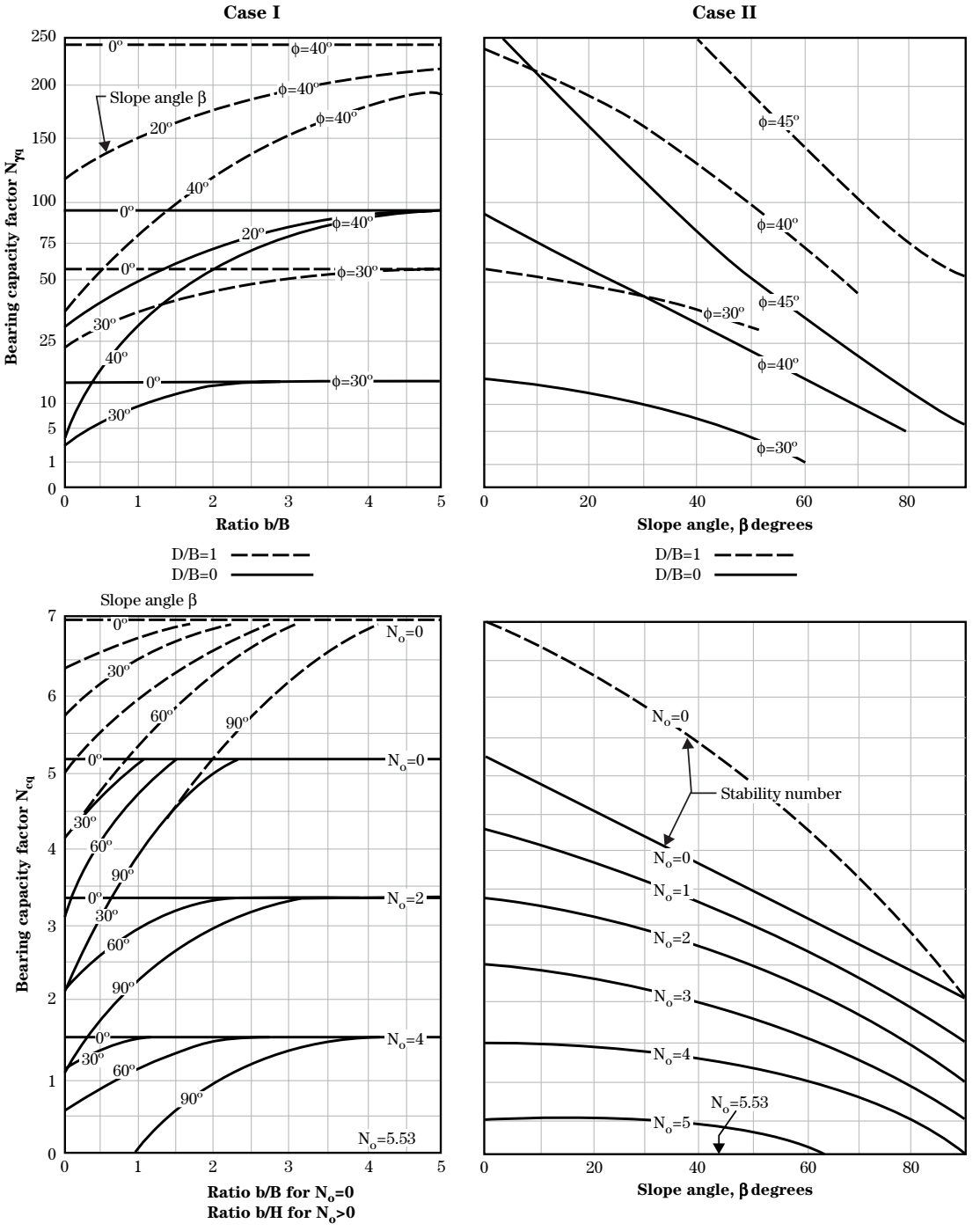


Table TS14Q–3 Meyerhof method design table (cohesionless soils)

Meyerhof method—cohesionless soils				Bearing capacity factors near slopes									
				$N_{\gamma q}$									
				b/B									
ϕ, deg	**D/B**	**β, deg**	**Z**	**0.00**	**0.25**	**0.50**	**0.75**	**1.00**	**1.50**	**2.00**	**3.00**	**4.00**	**6.00**
30	0	**0**		15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0
		18.4	3H:1V	7.0	8.3	9.8	11.3	12.2	13.8	14.4	15.0	15.0	15.0
		21.8	2.5H:1V	5.6	7.0	8.8	10.6	11.7	13.5	14.3	15.0	15.0	15.0
		26.6	2H:1V	3.5	5.2	7.5	9.7	11.0	13.2	14.1	15.0	15.0	15.0
		30		2.0	4.0	6.5	9.0	10.5	13.0	14.0	15.0	15.0	15.0
	1	**0**		57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0	57.0
		18.4	3H:1V	36.1	39.2	41.7	43.5	46.0	49.0	52.1	54.5	56.4	57.0
		21.8	2.5H:1V	32.3	35.9	38.8	41.0	43.9	47.6	51.2	54.1	56.3	57.0
		26.6	2H:1V	26.9	31.3	34.8	37.5	41.0	45.5	49.9	53.5	56.1	57.0
		30		23.0	28.0	32.0	35.0	39.0	44.0	49.0	53.0	56.0	57.0
40	0	**0**		92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0	92.0
		18.4	3H:1V	36.8	41.4	46.0	51.5	56.1	64.4	71.8	82.8	87.4	92.0
		20		32.0	37.0	42.0	48.0	53.0	62.0	70.0	82.0	87.0	92.0
		21.8	2.5H:1V	29.4	34.7	39.9	46.0	51.2	60.7	68.8	81.4	86.8	92.0
		26.6	2H:1V	22.4	28.4	34.4	40.7	46.4	57.1	65.7	79.7	86.3	92.0
		33.7	1.5H:1V	12.0	19.2	26.2	32.9	39.3	51.7	61.1	77.2	85.6	92.0
		40		2.8	11.0	19.0	26.0	33.0	47.0	57.0	75.0	85.0	92.0
	1	**0**		240.0	240.0	240.0	240.0	240.0	240.0	240.0	240.0	240.0	240.0
		18.4	3H:1V	133.3	140.6	148.0	154.4	159.0	171.9	182.0	200.4	213.3	230.8
		20		124.0	132.0	140.0	147.0	152.0	166.0	177.0	197.0	211.0	230.0
		21.8	2.5H:1V	116.2	124.4	132.7	140.1	145.7	160.5	172.2	193.4	208.6	228.6
		26.6	2H:1V	95.3	104.3	113.3	121.6	128.9	145.9	159.5	183.8	202.1	224.7
		33.7	1.5H:1V	64.4	74.5	84.5	94.3	104.1	124.2	140.7	169.6	192.5	219.0
		40		37.0	48.0	59.0	70.0	82.0	105.0	124.0	157.0	184.0	214.0

Notes:
1. Bold values of β and the associated $N_{\gamma q}$ values are read directly from the Meyerhof charts. Other values are interpolated.
2. Intermediate values of β, $N_{\gamma q}$ values may be determined by linear interpolation.
3. To calculate ultimate bearing capacity: $q_{ult} = 0.5\phi B N_{\gamma q}$

Table TS14Q–4 Meyerhof method design table (cohesive soils)

Meyerhof method—cohesive soils (ϕ = 0)				Bearing capacity factors near slopes									
				N_{cq}									
				b/B or b/H									
D/B	**N_s**	**β, deg**	**Z**	**0**	**0.5**	**1**	**1.5**	**2**	**2.5**	**3**	**3.5**	**4**	**4.5**
0	0	**0**		5.14	5.14	5.14	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		18.4	3H:1V	4.55	4.90	5.12	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		21.8	2.5H:1V	4.44	4.86	5.12	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		26.6	2H:1V	4.29	4.79	5.11	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		30		4.18	4.75	5.11	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		33.7	1.5H:1V	4.04	4.66	5.07	5.14	5.14	5.14	5.14	5.14	5.14	5.14
		60		3.08	4.06	4.82	5.12	5.14	5.14	5.14	5.14	5.14	5.14
		90		1.93	3.00	3.90	4.58	5.00	5.14	5.14	5.14	5.14	5.14
	2	**0**		3.33	3.33	3.33	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		18.4	3H:1V	3.08	3.23	3.32	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		21.8	2.5H:1V	3.03	3.21	3.32	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		26.6	2H:1V	2.97	3.18	3.32	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		30		2.92	3.16	3.32	3.33	3.33	3.33	3.33	3.33	3.33	3.33
		33.7	1.5H:1V	2.83	3.09	3.28	3.32	3.33	3.33	3.33	3.33	3.33	3.33
		60		2.16	2.62	3.00	3.22	3.32	3.33	3.33	3.33	3.33	3.33
		90		1.04	1.71	2.28	2.65	2.97	3.14	3.27	3.33	3.33	3.33
	4	**0**		1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		18.4	3H:1V	1.32	1.43	1.49	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		21.8	2.5H:1V	1.28	1.42	1.49	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		26.6	2H:1V	1.23	1.40	1.48	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		30		1.20	1.39	1.48	1.50	1.50	1.50	1.50	1.50	1.50	1.50
		33.7	1.5H:1V	1.12	1.32	1.43	1.48	1.49	1.50	1.50	1.50	1.50	1.50
		60		0.52	0.83	1.10	1.30	1.42	1.50	1.50	1.50	1.50	1.50
		90				0.03	0.60	0.98	1.21	1.33	1.41	1.48	1.50
1	0	**0**		7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00	7.00
		15		6.50	6.68	6.82	6.94	7.00	7.00	7.00	7.00	7.00	7.00
		18.4	3H:1V	6.35	6.57	6.75	6.90	6.98	7.00	7.00	7.00	7.00	7.00
		21.8	2.5H:1V	6.21	6.46	6.68	6.86	6.96	7.00	7.00	7.00	7.00	7.00
		26.6	2H:1V	6.01	6.31	6.59	6.81	6.94	7.00	7.00	7.00	7.00	7.00
		30		5.86	6.20	6.52	6.77	6.92	7.00	7.00	7.00	7.00	7.00
		33.7	1.5H:1V	5.68	6.06	6.40	6.69	6.87	6.98	7.00	7.00	7.00	7.00
		45	1H:1V	5.14	5.62	6.05	6.43	6.73	6.93	7.00	7.00	7.00	7.00
		60		4.11	4.80	5.44	5.95	6.41	6.74	6.98	7.00	7.00	7.00
		90				4.00	4.67	5.27	5.75	6.25	6.63	6.88	7.00

Notes:
1. Bold values of β and the associated N_{cq} values are read directly from the Meyerhof charts. Other values are interpolated.
2. Intermediate values of β, N_{cq} values may be determined by linear interpolation.
3. N_s = stability factor of slope = $\gamma H/c$
   where:
   γ = unit weight of soil (lb/ft^3)
   H = vertical height of slope (ft)
   c = cohesion (or undrained shear strength) of soil (lb/ft^2)
4. To calculate ultimate bearing capacity: $q_{ult} = cN_{cq}$

Table TS14Q–5 Undrained shear strength values for saturated cohesive soils

Consistency description	s_u 1/ (lb/ft²)	Thumb penetration/consistency	LI 2/	N_{60} (SPT) 3/ (blows/ft)
Very soft	0–250	Thumb penetrates >1 in, extruded between fingers	≥1.0	<2
Soft	250–500	Thumb penetrates 1 in, molded by light finger pressure	1.0–0.67	2–4
Medium	500–1,000	Thumb penetrates ¼ in, molded by strong finger pressure	0.67–0.33	4–8
Stiff	1,000–2,000	Indented by thumb, but not penetrated	0.33–0	8–15
Very stiff	2,000–4,000	Not indented by thumb, but indented by thumbnail	≤0	15–30
Hard	>4,000	Not indented by thumbnail	<0	>30

1/ s_u = undrained shear strength of soil

2/ LI = liquidity index = $\frac{w_{sat} - PL}{PI} = \frac{w_{sat} - LL + PI}{PI}$

where:

w_{sat}, % = saturated water content at in situ density = $\left[\left(\frac{\gamma_w}{\gamma_d}\right) - \left(\frac{1}{G_s}\right)\right] \times 100\%$

γ_w = unit weight of water = 62.4 lb/ft³

γ_d = unit weight of soil at *in situ* density, lb/ft³

G_s = specific gravity of soil solids, unitless

3/ N_{60} = blows per foot by standard penetration test (SPT), corrected for overburden pressure

Table TS14Q–6 ϕ' values for sands

Angle of internal friction of sands, ϕ'

a. Relative density and gradation (Schmertmann 1978)

Relative density D_r, percent	Fine grained		Medium grained		Coarse grained	
	Uniform	Well graded	Uniform	Well graded	Uniform	Well graded
40	34	36	36	38	38	41
60	36	38	38	41	41	43
80	39	41	41	43	43	44
100	42	43	43	44	44	46

b. Relative density and initial *in situ* soil tests

Soil type	Relative density D_r percent	Standard penetration resistance N_{60} (Terzaghi and Peck 1967)	Cone penetration resistance q_c, ksf (Meyerhof 1974a)	Friction angle ϕ', deg		
				Meyerhof (1974b)	Peck, Hanson, and Thornburn (1974)	Meyerhof (1974b)
Very loose	<20	<4	—	<30	<29	<30
Loose	20–40	4–10	0–100	30–35	29–30	30–35
Medium	40–60	10–30	100–300	35–38	30–36	35–40
Dense	60–80	30–50	300–500	38–41	36–41	40–45
Very dense	>80	>50	500–800	41–44	>41	>45

(a) ASTM D653 defines relative density as the ratio of the difference in void ratio of a cohesionless soil in the loosest state at any given void ratio to the difference between the void ratios in the loosest and in the densest states. A very loose sand has a relative density of 0 percent and 100 percent in the densest possible state. Extremely loose honeycombed sands may have a negative relative density.

(b) Relative density may be calculated using standard test methods ASTM D4254 and the void ratio of the *in situ* cohensionless soil,

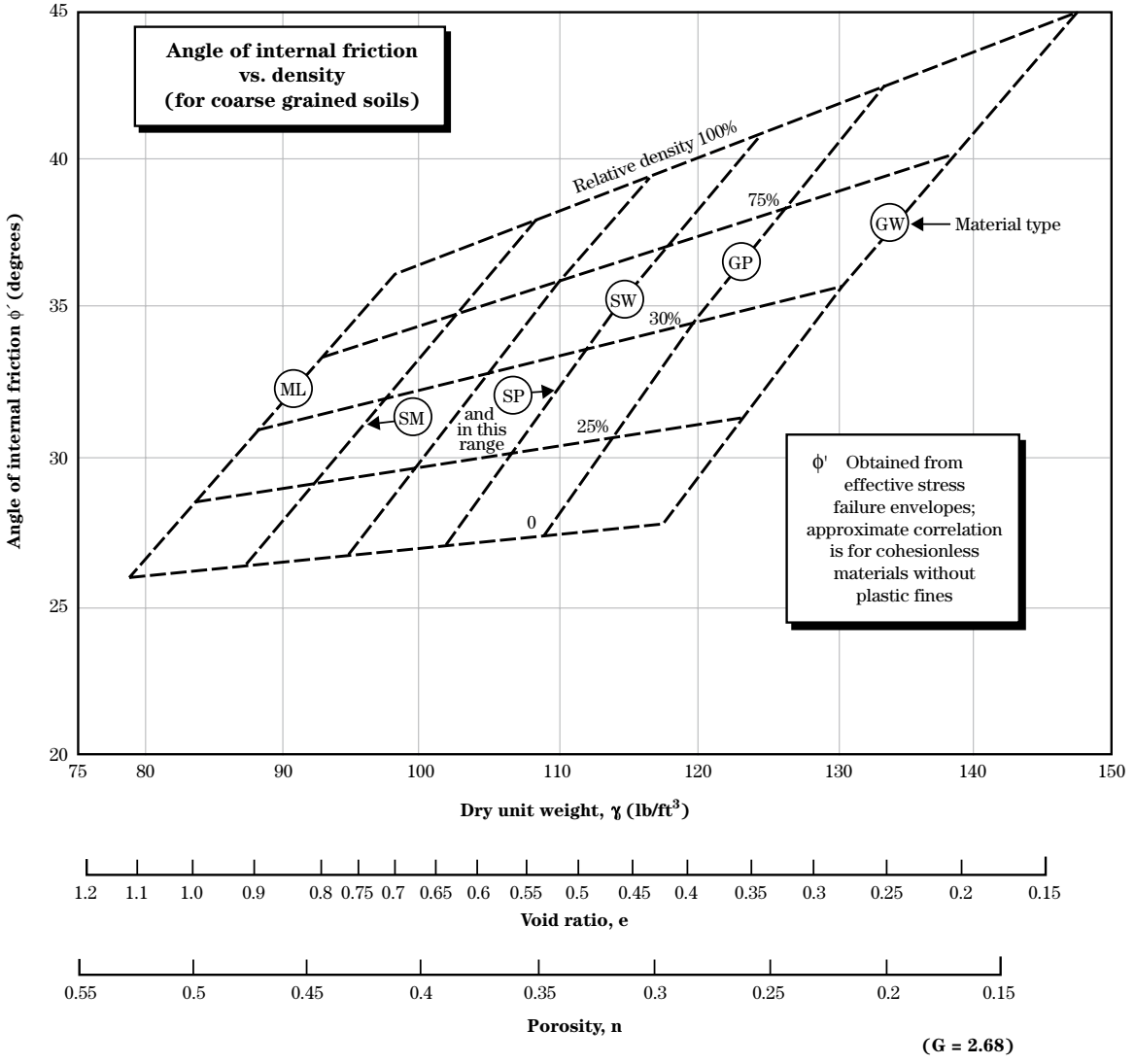
$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

$$e = \frac{G}{\gamma_d}\gamma_w - 1$$

where:

e_{min} = reference void ratio of a soil at the maximum density
e_{max} = reference void ratio of a soil at the minimum density
G = specific gravity
γ_d = dry density, kips/ft^3
γ_w = unit weight of water, 0.0625 kips/ft^3

Figure TS14Q–8 ϕ´ values for coarse grained soils



Example problem 1: Strip footing adjacent to a slope

Given: The abutment shown in figure TS14Q–9.

Find: Ultimate and allowable bearing capacity, q_{ult} and $q_{allowable}$, respectively.

Solution: Use the Meyerhof (1957) method to estimate the ultimate and allowable bearing capacity.

Compute $\frac{b}{B} = \frac{D}{B} = \frac{3\text{ ft}}{3\text{ ft}} = 1$

Since the soil is cohesionless (c = 0), the N_{cq} term in equation TS14Q–2 may be neglected.

Determine $N_{\gamma q}$ by interpolation from table TS14Q–3.

For $\phi = 30^\circ$, $N_{\gamma q} = 41.0$, and for $\phi = 40^\circ$, $N_{\gamma q} = 128.9$

By interpolation, for $\phi = 35^\circ$, $N_{\gamma q} = 85$

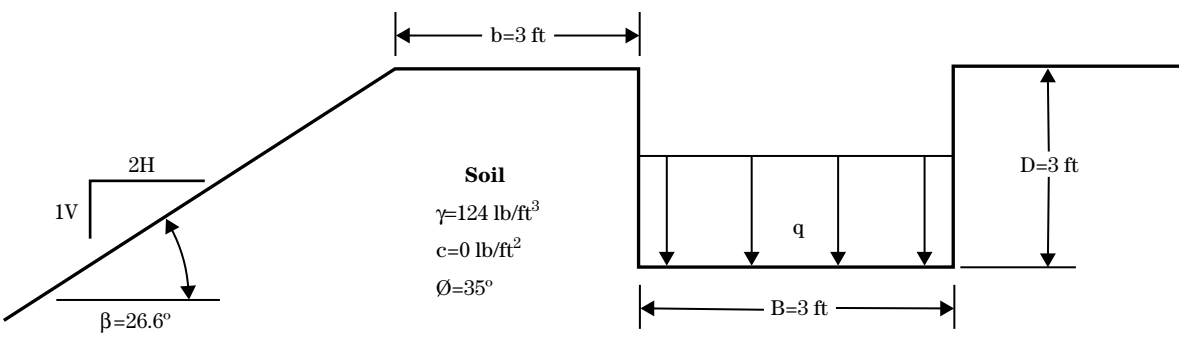
Solve for ultimate bearing capacity, q_{ult}, using equation TS14Q–2:

$$\begin{aligned} q_{ult} &= \frac{1}{2} \times \gamma \times B \times N_{\gamma q} \\ &= (0.5) \times \left(124\text{ lb/ft}^3\right) \times (3\text{ ft}) \times (85) \\ &= 15{,}810\text{ lb/ft}^2 \\ &= 15.8\text{ k/ft}^2 \end{aligned}$$

Applying a factor of safety (FS) of 3.0 to determine the allowable bearing capacity:

$$\begin{aligned} q_{allowable} &= \frac{q_{ult}}{FS} \\ &= \frac{\left(15.8\text{ k/ft}^2\right)}{3} \\ &= 5.3\text{ k/ft}^2 \end{aligned}$$

Figure TS14Q–9 Problem schematic for example problem 1—strip footing adjacent to a slope



Example problem 2: Surface load adjacent to slope

Given: The abutment shown in figure TS14Q–10.

Find: Ultimate and allowable bearing capacity, q_{ult} and $q_{allowable}$, respectively.

Solution: Use the Meyerhof (1957) method to estimate the ultimate and allowable bearing capacity.

Compute

$$\frac{b}{B} = \frac{3\text{ ft}}{3\text{ ft}} = 1$$

and

$$\frac{D}{B} = \frac{0\text{ ft}}{3\text{ ft}} = 0$$

Since the soil is purely cohesive ($\phi = 0$), the $N_{\gamma q}$ term in equation TS14Q–2 may be neglected.

$$N_s = \left(100\text{ lb/ft}^3\right) \times \frac{\left(10\text{ ft}\right)}{\left(500\text{ lb/ft}^2\right)}$$

$$= 3.0$$

Since $N_s > 0$, compute b/H = (3 ft)/(10 ft) = 0.33

Determine N_{cq} by interpolation from table TS14Q–4.

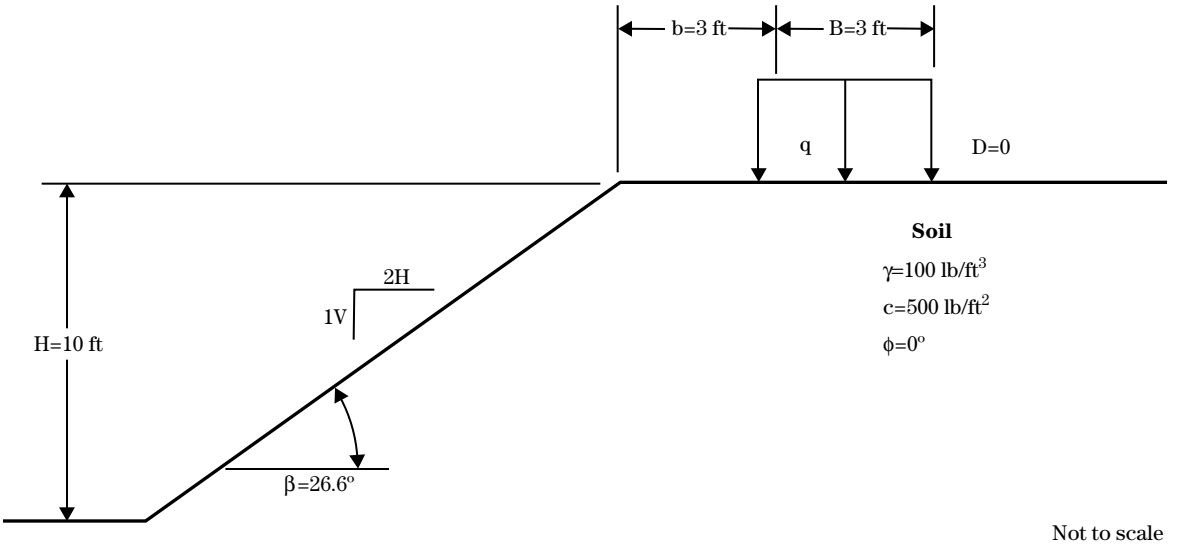
b/H	N_{cq}
0.50	3.18
0.33	3.11
0.00	2.97

So, by equation TS14Q–2, $q_{ult} = cN_c = (500\text{ lb/ft}^2)\times(3.11) = 1{,}560\text{ lb/ft}^2$

and

$q_{allowable} = q_{ult}/FS = (1{,}560\text{ lb/ft}^2)\div(3.0) = 520\text{ lb/ft}^2$

Figure TS14Q–10 Problem schematic for example problem 2—surface load adjacent to a slope



Technical Supplement 14R

Design and Use of Sheet Pile Walls in Stream Restoration and Stabilization Projects

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: Piles made of wood, steel, or concrete may be needed to stabilize streambanks where bridges, roads, and property boundaries are restrictive.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14R

Design and Use of Sheet Pile Walls in Stream Restoration and Stabilization Projects

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Technical Supplement 14R

Design and Use of Sheet Pile Walls in Stream Restoration and Stabilization Projects

Purpose

This technical supplement provides an introduction to the use of sheet pile, types of walls, sheet pile materials, classical method of design for wall stability, structural design, specification, and installation of sheet pile for stream restoration and stabilization projects.

This technical supplement describes typical applications for cantilever sheet pile wall in stream restoration and stabilization projects, types of sheet pile material, loads applied to the sheet pile, failure modes, design for cantilever wall stability, structural design of the piles, and some construction considerations. It does not address stream stability; hydraulic analyses of the streamflow; geotechnical analyses and slope stability of the stream slopes; or the ecological, aesthetic, or geomorphic consequences of the use of sheet pile.

Introduction

Sheet pile may be used in a variety of applications for stream restoration and stabilization. It is typically used to provide stability to a stream, stream slopes, or other constructed structures in high risk situations. Typical applications of sheet pile include toe walls, flanking and undermining protection, grade stabilization structures, slope stabilization, and earth retaining walls. While sheet pile can be combined with soil bioengineering techniques, it does have some ecologic and geomorphic disadvantages. Stream restoration and stabilization may require the use of structural measures to provide lateral or vertical stability to the stream. Structural measures include concrete retaining walls, drop structures, and sheet pile walls. These measures result in a statically stable stream within the stabilized area and are useful in arresting unacceptable lateral stream migration and local vertical instability. Structural measures may be used when vegetation and other soil bioengineering practices are not stable under the stress or duration of the design event or wherever the consequences of any movement of the banks are unacceptable.

Applications

Sheet pile walls may be used in a variety of applications for stream bank stabilization and restoration projects such as:

- toewall for scour protection (fig. TS14R–1)
- flanking and undermining protection (fig. TS14R–2)
- streambed grade stabilization (fig. TS14R–3)
- bank slope stability (fig. TS14R–4)
- bank retaining walls (fig. TS14R–5)

Figure TS14R–1 Toewall for scour protection cross section

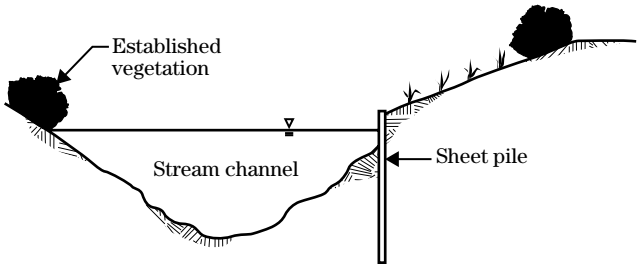


Figure TS14R–2 Flanking and undermining protection profile

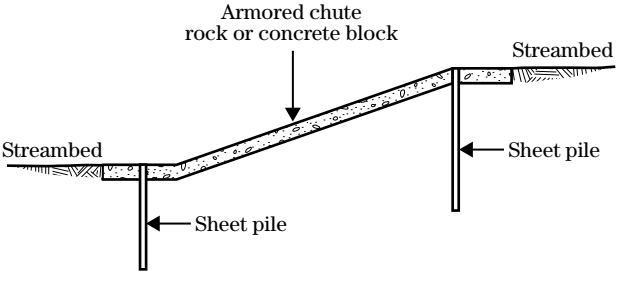


Figure TS14R–3 Streambed grade stabilization profile

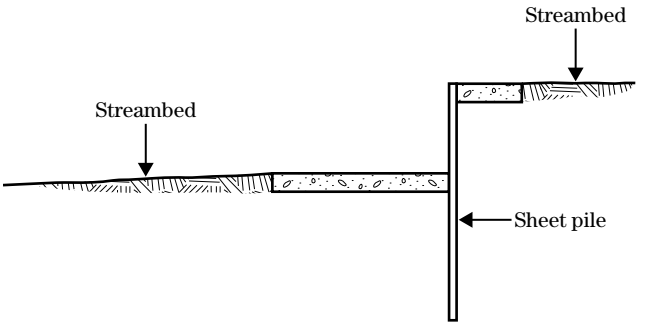


Figure TS14R–4 Bank slope stability cross section

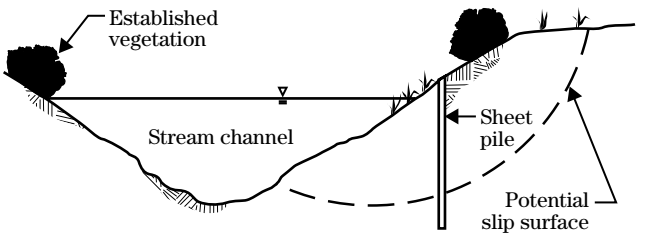
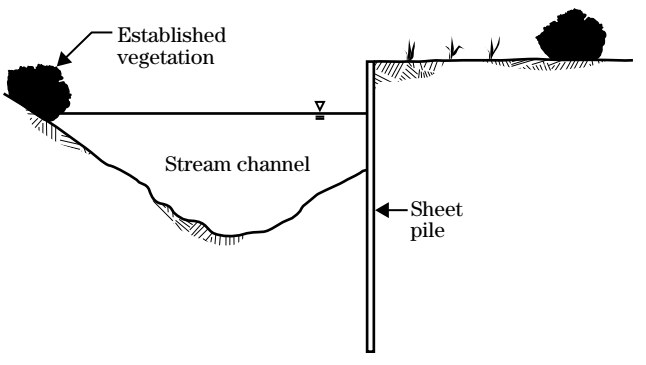


Figure TS14R–5 Bank retaining wall cross section



Types of sheet pile walls

Sheet pile walls may be cantilever or anchored walls. Figure TS14R–6 illustrates both a cantilever sheet pile wall and an anchored sheet pile wall. Cantilever walls derive support from adequate embedment below the stream channel. Steel cantilever walls are limited to wall heights of 15 to 20 feet, while vinyl cantilever walls are limited to 6 to 10 feet. An anchored wall is typically required when the wall height exceeds that suitable for a cantilever wall. Anchored sheet pile walls derive support from embedment in the soil and the anchor force(s) applied to the piling wall.

Materials

Steel

Steel sheet pile is available in various shapes (types), sizes, weights, and steel grades. Z-type piles and American Society for Testing and Materials International (ASTM) A572, Grade 50 are the most common.

Sheet pile may be hot-rolled or cold-rolled piles (fig. TS14R–7 (U.S. Army Corps of Engineers (USACE), 1994c). Hot-rolled piles are formed into the final shape with the interlocks, while the steel is in the molten state. Cold-rolled piles are fabricated into the final shape from flat plate steel. The interlocks for hot-rolled piles are stronger than cold-formed pile interlocks and may allow easier and straighter driving in hard driving conditions and allow less soil migration though the interlocks. Typically, less seepage and material loss are allowed through hot-rolled interlocks. Cold-rolled piles are usually acceptable for stream restoration and stabilization projects. Material specifications often allow either hot-rolled or cold-rolled piles.

Concrete

Concrete piles are precast and often prestressed, with a wall thickness of 6 to 12 inches and widths of 30 to 48 inches. The joints may be tongue-and-groove or grouted (fig. TS14R–8 (USACE 1994c)).

Figure TS14R–6 Sheet pile

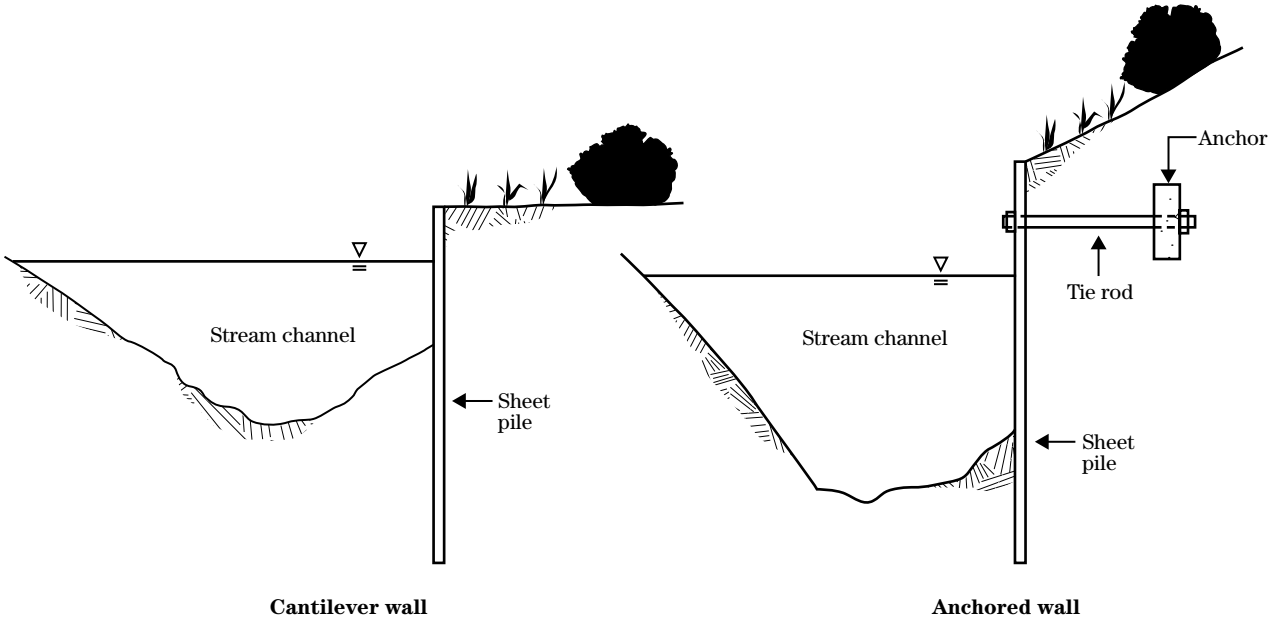


Figure TS14R–7 Sheet pile section

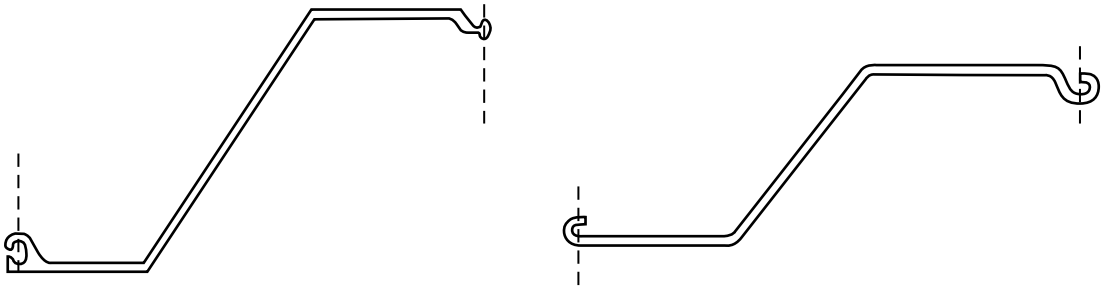
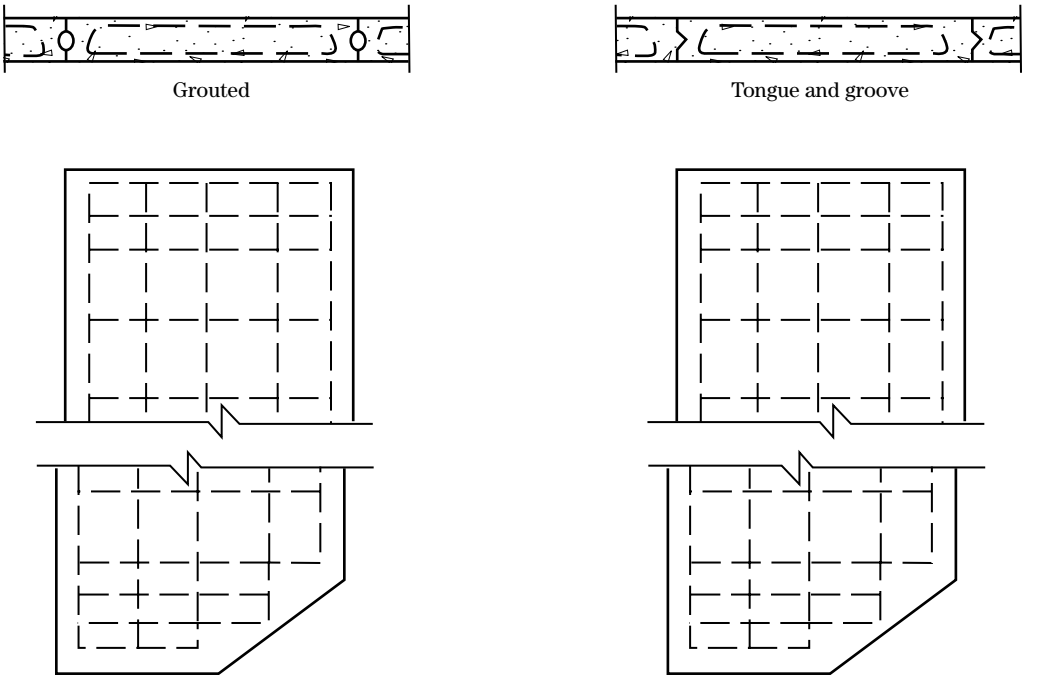


Figure TS14R–8 Concrete piles



Wood

Wood piles may be either independent sheets or tongue-and-groove interlocking sheets (fig. TS14R–9 (USACE 1994c)). Wood piles are used on short wall heights and often anchored with wood walers and vertical wood piles. All wood components should be treated to reduce rot or damage due to wood-destroying insects or water borers.

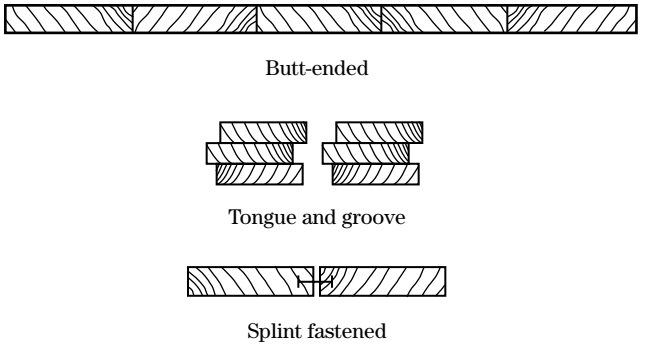
Vinyl

Vinyl sheet pile is available in sections of similar shape to Z-shaped steel sheet pile. Vinyl has lower strength and modulus of elasticity than steel and is, therefore, limited to lower wall heights or anchored walls. Vinyl sheet pile may be manufactured by monoextrusion of all virgin material or coextrusion of recycled material, coextruded with a virgin material coating to provide resistance to ultraviolet light.

Fiber reinforced polymer

Sheet piling may also be manufactured of a synthetic fiber-reinforced polymer (FRP). This type of sheet pile is also referred to as fiberglass or composite sheet pile. A FRP product consists of fiber reinforcement and a polymer resin matrix. The fiber reinforcement typically consists of glass reinforcing fibers. Because of the method of manufacturing, the mechanical properties (strength, modulus of elasticity) may vary with orientation. Due to the potential strength of FRP sheet pile and its resistance to corrosion, FRP could be considered for applications requiring wall heights higher than allowed for vinyl sheet pile or for areas with high corrosion potential for steel. The required strength, modulus of elasticity, and anisotropic nature of the material must be considered in the design.

Figure TS14R–9 Wood pile sheets



Soil properties

The shear strength of soils may vary depending on the rate that load is added to the soil, duration of the load, whether a previous load has been exerted on the soil (in particular for overconsolidated clays), and the permeability of the soil. Shear strength parameters are often characterized as undrained and drained parameters. The terms undrained and drained are not a description of the water level in the soils, but rather a description of the pore pressure condition in the soil when it is loaded. An undrained condition (also called short-term, quick, total stress, or unconsolidated-undrained) assumes that pore pressures will develop due to a change in load. The assumption is that the pore pressures that develop are not known and must be implicitly considered in the methods used to analyze soils for this condition.

A drained condition (also called long term, slow, effective stress, or consolidated-drained) implies that either no significant pore pressures are generated from the applied load or that the load is applied so slowly that the pressure dissipates during the slowly applied loading.

Coarse-grain soils

Coarse-grain soils include sands, gravels, and nonplastic silts of high enough permeability that excess pore pressures do not develop as a result of a change in loading. Soils with a permeability of $1x10^{-4}$ centimeters per second or greater are often assumed to have permeability rates high enough so that excess pore pressures do not develop. The shear strength of coarse-grain soils is estimated from a consolidated-

drained (CD) or consolidated-undrained condition with pore pressure measurements (CU´) shear tests. The shear strength may also be estimated from *in situ* tests such as standard penetration tests or cone penetration tests. The drained shear strength applies to both short-term and long-term load conditions. Typical soil properties for coarse-grain materials are shown in table TS14R–1.

Fine-grain soils

Fine-grain soils such as clays and plastic silts are more complex. They have a low permeability, and shear strength of these materials varies with duration of load. They have the potential to develop excess pore pressure due to changes in loading. If a soil has low permeability and experiences a fast change in load, it will exhibit undrained shear strength parameters. If the load is maintained for a sufficient period of time, the soil will exhibit drained shear stress parameters. Analyses in fine-grain soils should consider both undrained and drained conditions, with the most critical condition governing the design.

For overconsolidated clay soils that contain fissures and slickensides, the design of a sheet pile wall should consider the fully softened shear strength. If the wall is being placed to stabilize a recent slide, the residual shear strength should be considered. Typical soil properties for fine-grain materials are shown in table TS14R–1. Tables TS14R–2 and TS14R–3 provide the description of coarse-grain soil density and fine-grain soil consistency. Figure TS14R–10 illustrates the empirical correlation between effective phi angle and PI (USACE 1994c) A more detailed treatment of soil properties is provided in NEH654 TS14A.

Table TS14R–1 Estimated soil properties

Soil type 1/	Moist unit weight lb/ft^3	Sat. unit weight lb/ft^3	Undrained shear strength properties		Drained shear strength properties		Angle of wall friction (steel pile) δ	Wall/soil adhesion 3/ lb/ft^2
			Cohesion lb/ft^2	Angle of internal friction ϕ	Cohesion lb/ft^2	Angle of internal friction ϕ		
Loose sand	95–125	120–130	0	28	0	28	$0.5 \times \phi$	0
Medium dense sand	110–130	125–135	0	32	0	32	$0.5 \times \phi$	0
Dense sand	110–140	130–140	0	38	0	38	$0.5 \times \phi$	0
Very soft clay	85–100	85–100	0–250	0	0	2/	$0.5 \times \phi$	0–250
Soft clay	100–120	100–120	250–500	0	0	2/	$0.5 \times \phi$	250–500
Medium clay	110–125	110–125	500–1,000	0	0	2/	$0.5 \times \phi$	500–750
Stiff clay	115–130	115–130	1,000–2,000	0	50–100	2/	$0.5 \times \phi$	750–950
Very stiff clay	120–140	120–140	2,000–4,000	0	100	2/	$0.5 \times \phi$	950
Hard clay	>130	>130	>4,000	0	100	2/	$0.5 \times \phi$	950

Compiled from USACE, EM 1110–2–2504, Design of Sheet Pile Walls; Pile Buck, Inc. Steel Sheet Piling Design Manual; and NAVFAC DM –7.2, Foundations and Earth Structures

1/ See tables TS14R–2 and TS14R–3 for qualitative descriptions of soil types.

2/ See figure TS14R–10 (USACE 1994c).

3/ Wall/soil adhesion is typically 0 for drained (long-term) conditions.

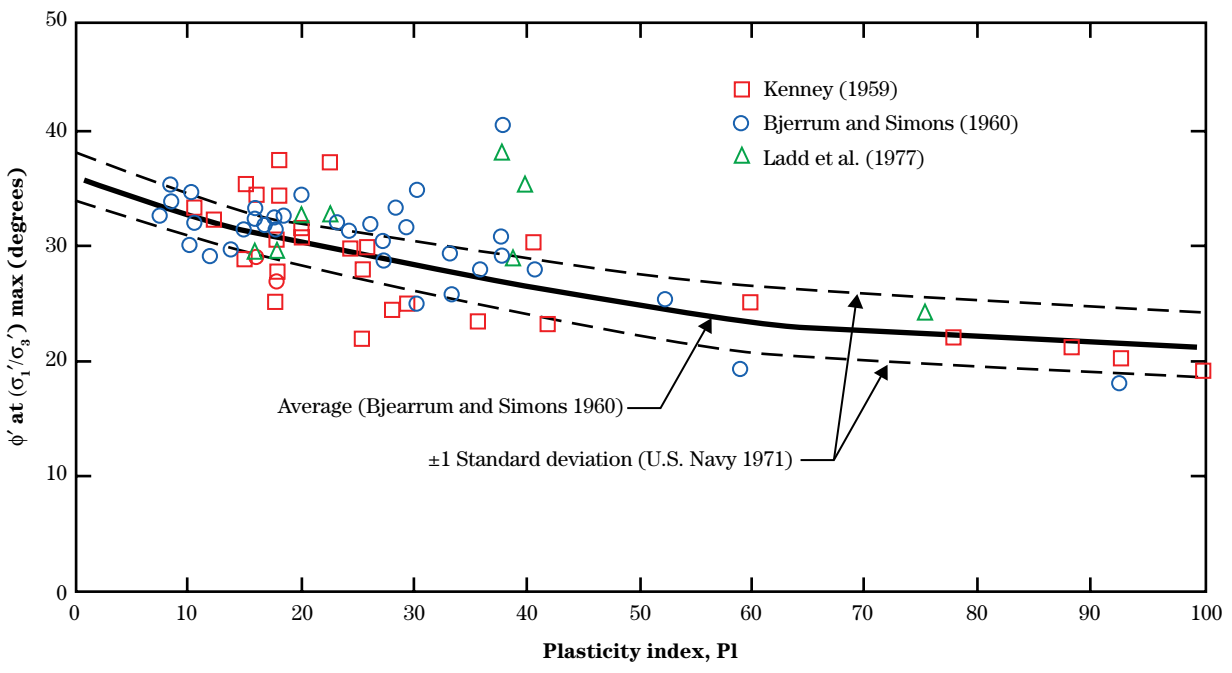
Table TS14R–2 Description of coarse-grain soil density

Density description	Evaluation/description
Very loose	A 1/2-in rod can be pushed easily by hand into soil
Loose	Soil can be excavated with a spade. A 2-in square wooden peg can easily be driven to a depth of 6 in
Medium dense	Soil is easily penetrated with a 1/2-in rod driven with a 5-lb hammer
Dense	Soil requires a pick for excavation A 2-in square wooden peg is hard to drive to a depth of 6 in
Very dense	Soil is penetrated only a few centimeters with a 1/2-in rod driven with a 5-lb hammer

Table TS14R–3 Description of fine-grain soil consistency

Saturated consistency	Evaluation/description
Very soft	Thumb will penetrate greater than 1 in. Soil is extruded between fingers
Soft	Thumb will penetrate about 1 in. Soil molded by light finger pressure
Medium	Thumb will penetrate about 1/4 in. Soil molded by strong finger pressure
Stiff	Indented with thumb
Very stiff	Indented by thumb nail
Hard	Thumbnail will not indent

Figure TS14R–10 Empirical correlation between effective phi angle and PI



Loads

Lateral earth pressure

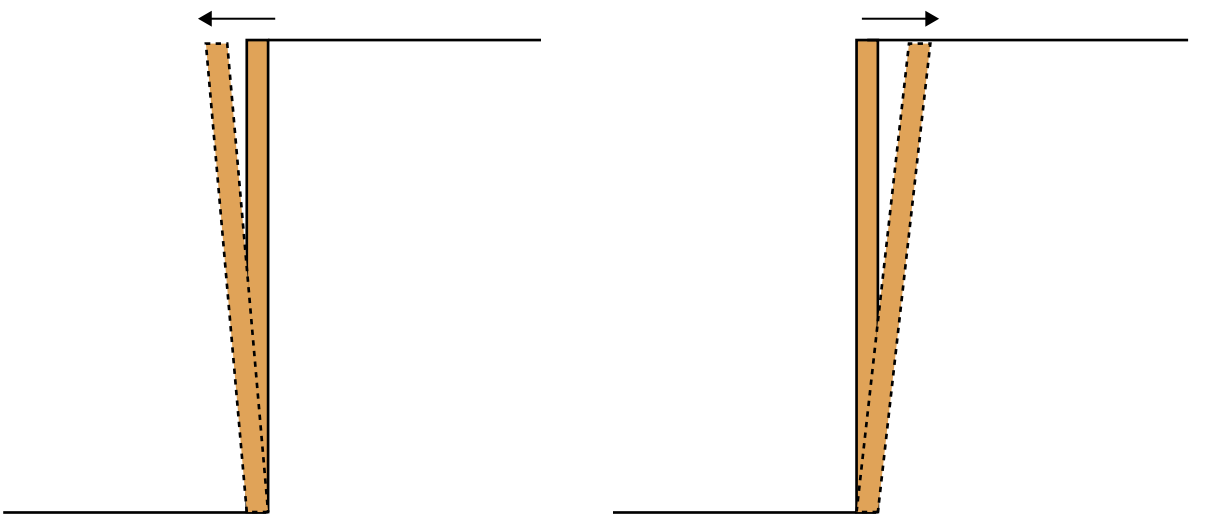
The lateral (horizontal) earth pressure is a function of the soil properties (cohesion, phi angle, and unit weight), height of overburden, and the elevation of the water table. Earth pressures varies from an initial state referred to as at-rest, K_o, to a minimum limit state referred to as active, K_A, to a maximum limit state referred to as passive, K_P. The classical method of sheet pile design assumes development of active and passive lateral earth pressures.

Active earth pressure develops when the pile moves or rotates away from the soil allowing the soil to expand laterally (horizontally) in the direction of the pile movement (fig. TS14R–11). Active earth pressure is the driving force in sheet pile stability analysis. In general, a lateral movement of approximately 1 inch is required to fully mobilize the shear resistance for each 20 feet of wall height.

Passive earth pressure develops when the pile moves or rotate towards the soil, tending to compress the soil laterally (horizontally) in the direction of the pile movement (fig. TS14R–11). Passive earth pressure is the resisting force in sheet pile stability analysis. In general, a lateral movement of approximately 1 inch is required to fully mobilize the shear resistance for each 2 feet of wall height.

More rigorous analysis may be conducted, assuming that the soil behaves as a spring, with the maximum resistance equal to the active or passive lateral earth pressure, as appropriate.

Figure TS14R–11 Active and passive rotation



Active state | Passive state

Water loads

A difference in water level on either side of the wall creates unbalanced hydrostatic pressure, adding to the pressure forcing the wall outward (fig. TS14R–12). The difference in water level may be because of a ground water table, which is higher in the bank than in the stream, a higher water level upstream of an inchannel sheet pile, or from a high streamflow which saturates the bank, followed by rapid drawdown when the water level in the stream drops faster than the water can drain from the bank.

Surcharge

Surface surcharge (fig. TS14R–13) also exerts lateral pressure on the wall, forcing the wall outward. Typical surcharge loadings may be due to equipment (parked or traveling), storage areas, construction materials, vehicles, and others. Surcharge loads are often estimated to be 100 to 200 pounds per square foot. Other surcharge loads include spoil, snow, or ice.

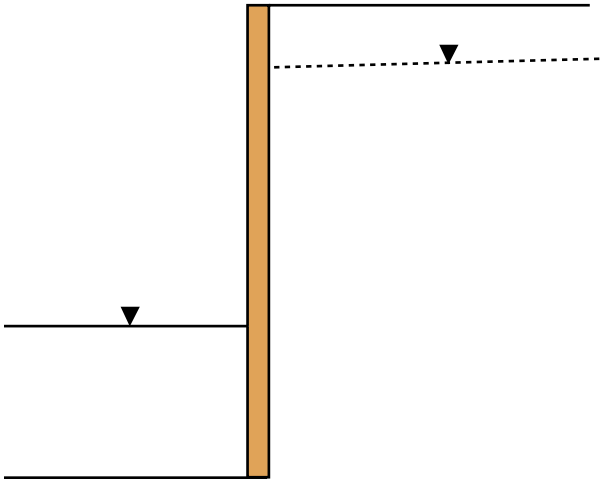
Wall stability

Both anchored and cantilever sheet pile must be analyzed against overturning. Wall penetration must be great enough to prevent deep-seated failure (fig. TS14R–14 (USACE 1994c)) or rotational failure (fig. TS14R–15 (USACE 1994c)). Deep-seated failure should be assessed by a slope stability analysis conducted by a geotechnical engineer.

The rotational stability of a cantilever wall or an anchored wall may be evaluated using methods presented in the Retaining Wall Design Guide (U.S. Department of Agriculture Forest Service (USDA FS) 1994) or the Steel Sheet Piling Design Manual (Pile Buck, Inc. 1987), with some simplifying assumptions, or the USACE computer program CWALSHT (USACE 1994b).

Penetration depths determined by the Retaining Wall Design Guide or Steel Sheet Piling Design Manual are typically increased by 30 percent to provide a factor of safety against overturning. An example calculation is provided later in this technical supplement.

Figure TS14R–12 Differential water load



Active state

Figure TS14R–13 Surface surcharge loading

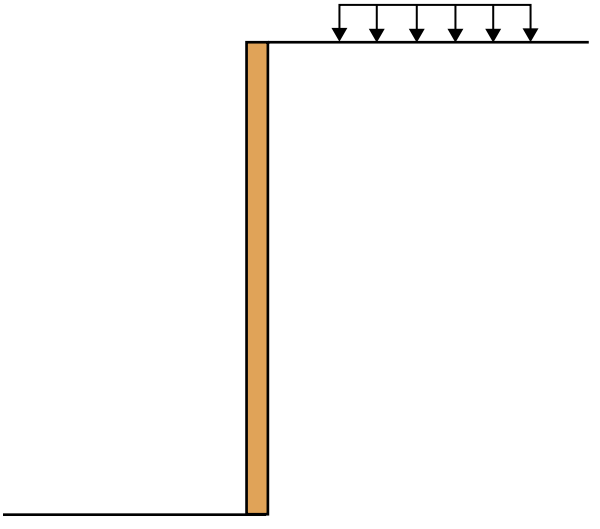
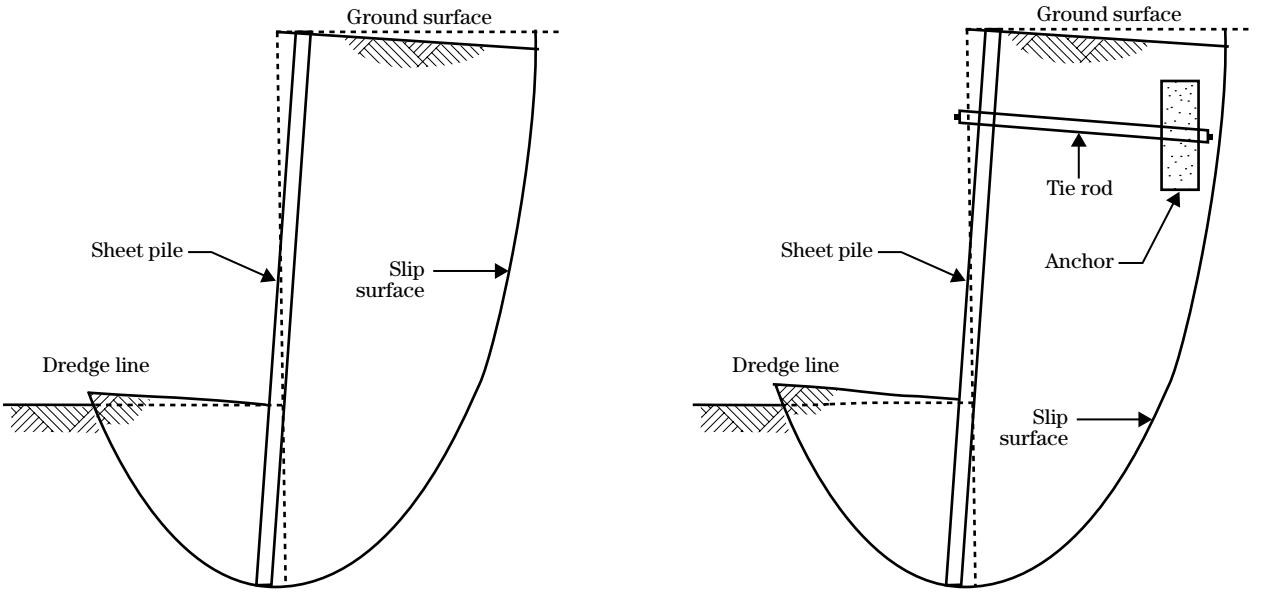
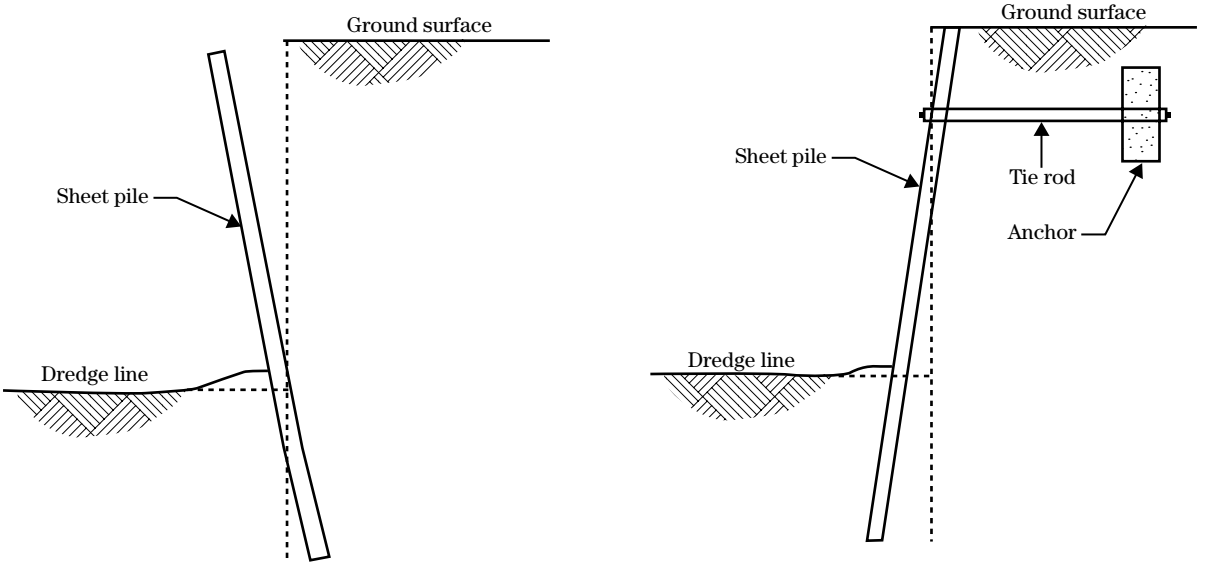


Figure TS14R–14 Deep-seated failure



a. Cantilever wall **b. Anchored wall**

Figure TS14R–15 Wall rotational failure



a. Cantilever wall **b. Anchored wall**

CWALSHT calculates the required depth of penetration based on the acceptable factor of safety for passive soil pressures. Factors of safety are applied to both active and passive pressures. A factory of safety for active pressures may be applied; however, it is considered sufficient to use a value of 1.0 unless wall deformations are restricted. The following factors of safety are recommended by the USACE for retaining walls:

- Usual loads—2.0 for undrained (short-term) conditions and 1.5 for drained (long-term) conditions
- Unusual loads—1.75 for undrained (short-term) conditions and 1.25 for drained (long-term) conditions
- Extreme loads—1.5 for undrained (short-term) conditions and 1.10 for drained (long-term) conditions

Usual loads are considered to be loads frequently experienced by the system in performing its primary function such as retaining soil or a differential water load at the design storm. Usual loads may be long term or intermittent. Unusual loads may be construction or operational loads of an infrequent and short-term nature such as surcharge from construction equipment adjacent to the wall. Extreme loads are worst case loads such as water loads above the design storm. The system should be designed to withstand extreme loads without failure.

CWALSHT will compute both the active and passive lateral earth pressures from the input listed below and conduct a sheet pile design or analysis:

- wall height
- water table elevations
- anchor location (if anchored wall)
- soil properties
- moist unit weight
- saturated unit weight
- undrained shear strength parameters (as applicable)
- drained shear strength parameters (as applicable)
- soil/pile properties
- angle of wall friction
- wall/soil adhesion

An example using CWALSHT is provided at the end of this technical supplement.

Structural design

Sheet pile failure may also be caused by overstressing the pile (fig. TS14R–16 (USACE 1994c)).

To avoid compounding factors of safety, the sheet piling and wales are designed to resist forces produced by soil pressures calculated using a factor of safety of 1.0 for both active and passive pressures (USACE 1994c). Therefore, the design bending moment, shear, and associated deflection for the sheet pile are based on a factor of safety of 1.0 for both active and passive soil pressures.

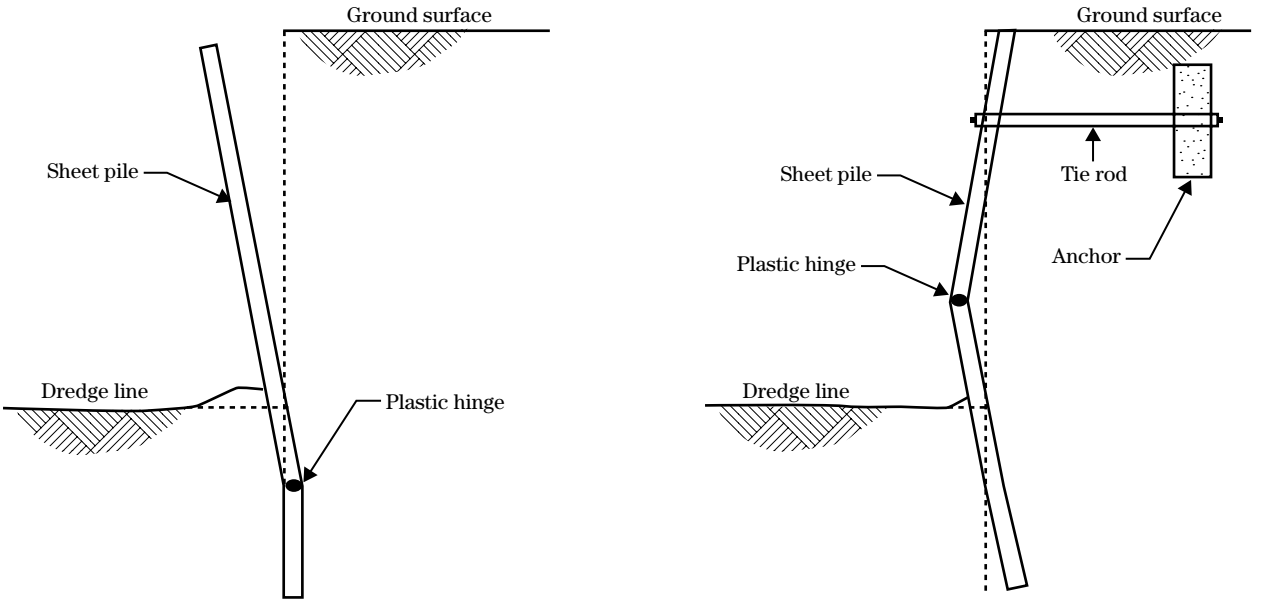
Moment—The maximum moment may be evaluated using methods presented in the Retaining Wall Design Guide or Steel Sheet Piling Design Manual. The moment along the length of the pile may be evaluated with CWALSHT.

Shear—The shear along the length of the pile may be evaluated with CWALSHT.

Deflection—A scaled deflection is calculated in the CWALSHT design mode. An estimate of the actual deflection may be determined by dividing the scaled deflection by the modulus of elasticity, E, of the pile material and the moment of inertia, I, of the pile section. The deflection along the length of a particular pile section may be evaluated with the CWALSHT analysis mode.

No firm guidelines exist for acceptable deflection, and values ranging from 1 to 5 inches are typically considered acceptable. It is recommended that the deflection be limited to 1 to 3 inches for stream restoration and stabilization projects.

Figure TS14R–16 Overstressed pile



a. Cantilever wall **b. Anchored wall**

Sheet design

Steel—The minimum section modulus per foot of wall is:

$$S_{min} = \frac{M_{max}}{f_b} \qquad \text{(eq. TS14R–1)}$$

The minimum shear area per foot of wall is:

$$A_{v,min} = \frac{V_{max}}{f_v} \qquad \text{(eq. TS14R–2)}$$

The allowable stress, f_b, for steel sheet piling is typically:

Bending and axial load:

$$f_b = 0.5f_y \quad \text{(usual loads)} \qquad \text{(eq. TS14R–3)}$$

$$f_b = 0.5f_y \times (1.33) \quad \text{(unusual loads)} \qquad \text{(eq. TS14R–4)}$$

$$f_b = 0.5f_y \times (1.75) \quad \text{(extreme loads)} \qquad \text{(eq. TS14R–5)}$$

Shear:

$$f_v = 0.33f_y \quad \text{(usual loads)} \qquad \text{(eq. TS14R–6)}$$

$$f_v = 0.33f_y \times (1.33) \quad \text{(unusual loads)} \qquad \text{(eq. TS14R–7)}$$

$$f_v = 0.33f_y \times (1.75) \quad \text{(extreme loads)} \qquad \text{(eq. TS14R–8)}$$

where:

f_y = yield strength of the pile material

Concrete—Reinforced concrete and prestressed concrete piles should be designed in accordance with the appropriate ACI Code.

Wood—Wood piles should be designed in accordance with the **National Design Specification for Wood Construction (NDS)** (American Wood Council 2005).

Vinyl and fiber reinforced polymer—The allowable stress should be limited to half the yield stress of the material.

Anchor design—Anchors may consist of concrete, steel member, or sheet pile deadmen attached to the pile with tie rods, tiebacks with grouted anchors, or various configurations of steel or concrete piles attached to the sheet pile by a wale or through a tie rod. Design of anchors and tie rods is described in Design of Sheet Pile Walls (USACE 1994c).

Construction considerations

Piling—Cold-rolled steel sheet pile sections have a weaker interlock than hot-rolled sections and may unlock while being driven in hard conditions, resulting in misalignment. A minimum pile thickness of a fourth inch is typically recommended for driveability. In tough driving conditions, such as dense to very dense sands, very stiff to hard clay soils, or soils containing significant amounts of gravel, a thicker pile should be considered. In areas where corrosion of the steel pile is a concern, a thicker pile than required structurally should be considered to allow for corrosion throughout the design life.

Equipment—Sheet pile is typically installed by driving, jetting, or trenching. Jetting is often not allowed for walls designed to retain soil. Hammers for driving may be steam, air, diesel-drop, single action, double action, differential action, or vibratory. Vibratory hammers work well in sand, silt, or softer clay soils. Harder driving conditions such as stiff clay may require an impact hammer.

Access for a crane is often required to operate the hammer. Short piles or piles in easier driving conditions may be installed with a backhoe or hammer attached to a back/track hoe. A temporary guide structure or template is recommended to ensure that the piles are driven in the correct alignment. Use of a protective cap is required with impact hammers. Protective shoes may be used on the tip of a pile in hard driving conditions.

When driving vinyl pile in stiff clays or dense sands, a steel mandrel is often driven with the vinyl pile and extracted upon completion of driving. The purpose of the mandrel is to support the vinyl pile only during driving.

Pile driving and installation—Piles should be driven with the proper size hammer for the size of pile, depth of penetration, and soil conditions. When impact hammers are used, the hammer should be appropriately sized and a protective cap utilized to prevent excessive damage to the pile. In some conditions, large impact hammers are not appropriate for driving smaller pile sections and have caused excessive damage to the pile. A smaller impact hammer may work better in these situations.

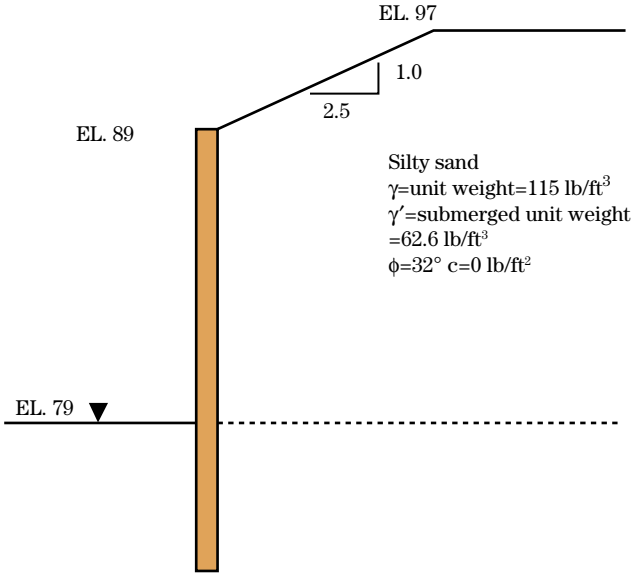
Alignment—Piles should be maintained in alignment during driving. Sheet pile should not be driven more than an eighth inch per foot out of plumb in the plane of the wall or perpendicular to the plane of the wall.

Conclusion

Sheet pile may be used in a variety of applications for stream restoration and stabilization. Typical applications of sheet pile are in high risk situations where no additional bank or bed movement is acceptable. Sheet pile applications include toe walls, flanking and undermining protection, grade stabilization structures, slope stabilization, and earth retaining walls. Sheet pile is often combined with soil bioengineering techniques to provide stability to a stream, stream slopes, or other manmade structures. It is particularly useful in open channel environments that are characterized by high velocities and shear stresses. Its use has distinct advantages because of accepted design techniques established contracting and construction procedures. However, the use of sheet pile does have certain cost, aesthetic, ecologic and geomorphic drawbacks. It is important to balance these potential drawbacks against the need to provide static protection.

Steel sheet pile wall example calculation

Given: A steel sheet pile will be installed to provide support to the lower streambank and prevent further erosion of the bank toe.



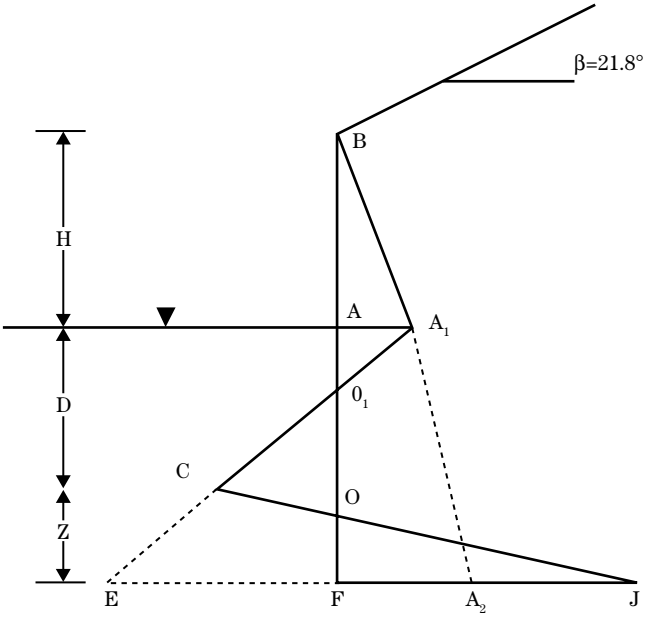
Determine:

Required embedment depth

Design embedment depth

Minimum recommended steel sheet pile wall properties

Solution:



Net soil pressure

Steel sheet pile wall example calculation—Continued

$$\beta = \tan^{-1}\frac{1}{2.5} = 21.8^{\circ} \qquad \text{(eq. TS14R–9)}$$

Using Rankine equations for lateral earth pressure coefficients.

$$K_A = \cos\beta\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \qquad \text{(eq. TS14R–10)}$$

$$= \cos 21.8\frac{\cos 21.8 - \sqrt{\cos^2 21.8 - \cos^2 32}}{\cos 21.8 + \sqrt{\cos^2 21.8 - \cos^2 32}} = 0.39$$

$$K_P = \cos\beta\frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}} \qquad \text{(eq. TS14R–11)}$$

$$= \cos 21.8\frac{\cos 21.8 + \sqrt{\cos^2 21.8 - \cos^2 32}}{\cos 21.8 - \sqrt{\cos^2 21.8 - \cos^2 32}} = 2.2$$

Less conservative methods of estimating K_A and K_P, such as Coulomb (1776) equations, are acceptable. These equations are described in the Steel Sheet Piling Design Manual (Pile Buck, Inc. 1987) and other foundation engineering references.

Determine wall pressures:

$$P_{A1} = \gamma H K_a = 115 \times 10 \times 0.39 = 448.5 \text{ lb/ft}^2 \qquad \text{(eq. TS14R–12)}$$

$$\begin{aligned} P_{A2} &= P_{A1} + \gamma' D K_A \\ &= 448.5 + (62.6)D(0.39) \\ &= 448.5 + 24.4D \end{aligned} \qquad \text{(eq. TS14R–13)}$$

$$\begin{aligned} P_E &= \gamma' D\left(K_p - K_A\right) - P_{A1} \\ &= 62.6D(2.2 - 0.39) - 448.5 \\ &= 113.3D - 448.5 \end{aligned}$$

$$\begin{aligned} P_J &= \gamma' D(K_P - K_A) + \gamma H K_P \\ &= 62.6D(2.2 - 0.39) + 115 \times 10 \times 2.2 \\ &= 113.3D + 2{,}530 \end{aligned} \qquad \text{(eq. TS14R–14)}$$

To be stable, the sum of forces must be zero, and the sum of moments at any point must be zero.

$$\sum F = 0 \text{ by areas} \qquad \text{(eq. TS14R–15)}$$

$$\text{Area}\left(BAA_1\right) + \text{Area}\left(AA_1A_2F\right) + \text{Area}\left(ECJ\right) - \text{Area}\left(EA_1A_2\right) = 0 \qquad \text{(eq. TS14R–16)}$$

or

Steel sheet pile wall example calculation—Continued

$$\frac{1}{2}HP_{A1}+\left(P_{A1}+P_{A2}\right)\frac{D}{2}+\left(P_E+P_J\right)\frac{Z}{2}-\left(P_E+P_{A2}\right)\frac{D}{2}=0 \qquad \text{(eq. TS14R–17)}$$

Solve this equation for Z:

$$Z=\frac{\left(P_E-P_{A1}\right)D-HP_{A1}}{P_E+P_J} \qquad \text{(eq. TS14R–18)}$$

$$\sum M=0 \text{ at point F} \qquad \text{(eq. TS14R–19)}$$

$$\sum M_F=\frac{1}{2}HP_{A1}\left(D+\frac{H}{3}\right)+P_{A1}\frac{D^2}{2}+\left(P_E+P_J\right)\frac{Z^2}{6}-\left(P_E+P_{A2}\right)\frac{D^2}{6}+\left(P_{A2}-P_{A1}\right)\frac{D^2}{6}=0 \qquad \text{(eq. TS14R–20)}$$

These two equations may be solved by trial and error.

Assume a depth of penetration, D.

Solve for Z.

Substitute Z into the ΣM_F. Continue adjusting D and Z until ΣM_F =0.

Try D = 20 feet

$$P_{A1}=448.5 \text{ lb/ft}^2 \qquad \text{(eq. TS14R–21)}$$

$$P_{A2}=448.5+\left(24.4\times 20\right)=936 \qquad \text{(eq. TS14R–22)}$$

$$P_E=113.3\times 20-448.5=1,817 \qquad \text{(eq. TS14R–23)}$$

$$P_J=113.3\times 20+2,530=4,796 \qquad \text{(eq. TS14R–24)}$$

$$Z=\frac{\left(P_E-P_{A1}\right)D-HP_{A1}}{P_E+P_J}=\frac{\left(1817-448\right)20-10\times 448}{1,817+4,796}=3.46 \qquad \text{(eq. TS14R–25)}$$

$$\Sigma M_F=\frac{1}{2}\times 10\times 448\left(20+\frac{10}{3}\right)+448\frac{20^2}{2}+\left(1,817+4,796\right)\frac{3.46^2}{6}-\left(1,817+936\right)\frac{20^2}{6}+\left(936-448\right)\frac{20^2}{6} \qquad \text{(eq. TS14R–26)}$$

$$\sum M_F=4,185 \qquad \text{(eq. TS14R–27)}$$

Try D = 20.5 feet

$$P_{A1}=448.5 \text{ lb/ft}^2 \qquad \text{(eq. TS14R–28)}$$

$$P_{A2}=448.5+\left(24.4\times 20.5\right)=948 \qquad \text{(eq. TS14R–29)}$$

$$P_E=113.3\times 20.5-448.5=1,874 \qquad \text{(eq. TS14R–30)}$$

$$P_J=113.3\times 20.5+2530=4852 \qquad \text{(eq. TS14R–31)}$$

Steel sheet pile wall example calculation—Continued

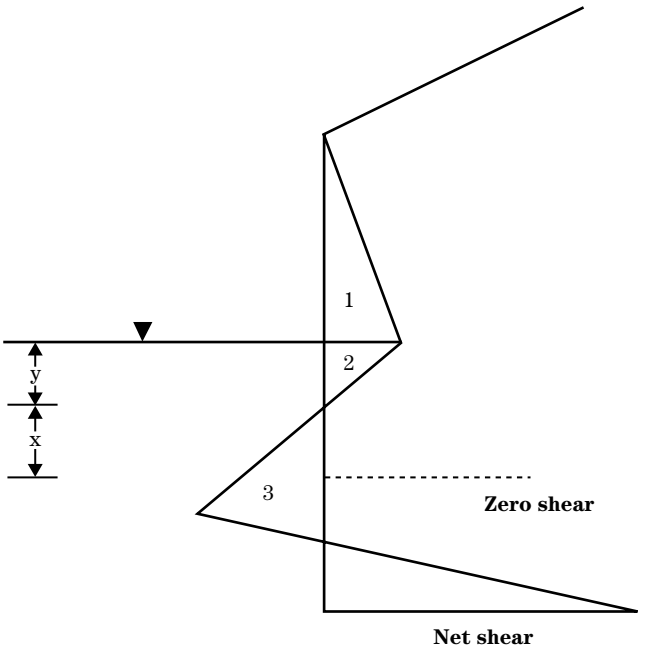
$$Z = 3.68 \quad \text{(eq. TS14R–32)}$$

$$\sum M_F = 164 \text{ ft-lb} \quad \text{(eq. TS14R–33)}$$

The required depth is 20.5 feet.

To determine the design embedment depth, increase D by 30 percent for a factor of safety.

$$D = 20.5 \times 1.30 = 26.65 \text{ ft} \quad \text{(eq. TS14R–34)}$$



The design embedment depth is 26.5 feet.

Locate the point of zero shear.

$$y = \frac{P_{A1}}{\gamma'(K_P - K_A)} = \frac{448.5}{62.6(2.2 - 0.39)} = 3.96 \quad \text{(eq. TS14R–35)}$$

$$P_1 = \frac{1}{2} P_{A1} H = \frac{1}{2} \times 448 \times 10 = 2{,}240 \text{ lb/ft of wall} \quad \text{(eq. TS14R–36)}$$

Steel sheet pile wall example calculation—Continued

$$P_2 = \frac{1}{2} P_{A1} y = \frac{1}{2} \times 448 \times 3.96 = 887 \text{ lb/ft of wall} \quad \text{(eq. TS14R–37)}$$

Solve for X

$$\frac{1}{2} \gamma' \left(K_P - K_A\right) x^2 = P_1 + P_2 \quad \text{(eq. TS14R–38)}$$

$$x^2 = \frac{2\left(P_1 + P_2\right)}{\gamma'\left(K_P - K_A\right)} = \frac{2\left(2{,}240 + 887\right)}{62.6\left(2.2 - 0.39\right)} = 55 \text{ ft} \quad \text{(eq. TS14R–39)}$$

$$x = 7.42 \text{ ft} \quad \text{(eq. TS14R–40)}$$

Solve for the maximum moment (occurs at the point of zero shear)

$$P_3 = \frac{1}{2} \gamma' \left(K_P - K_A\right) x^2 = P_1 + P_2 = 2{,}240 + 887 = 3{,}127 \quad \text{(eq. TS14R–41)}$$

$$M_{max} = P_1 l_1 + P_2 l_2 - P_3 l_3 \quad \text{(eq. TS14R–42)}$$

$$l_1 = \frac{H}{3} + y + x = \frac{10}{3} + 3.96 + 7.42 = 14.71 \quad \text{(eq. TS14R–43)}$$

$$l_2 = \frac{2y}{3} + x = \frac{2\left(3.96\right)}{3} + 7.42 = 10.06 \quad \text{(eq. TS14R–44)}$$

$$l_3 = \frac{x}{3} = \frac{7.42}{3} = 2.47 \quad \text{(eq. TS14R–45)}$$

$$M_{max} = 2{,}240\left(14.71\right) + 887\left(10.06\right) - 3{,}127\left(2.47\right) = 34{,}149 \text{ ft-lb} \quad \text{(eq. TS14R–46)}$$

The minimum section modulus of 50 kilopounds per square inch steel per foot of wall is:

$$S_{min} = \frac{M_{max}}{f_b} \quad \text{(eq. TS14R–47)}$$

$$f_b = 0.5 f_y = 0.5 \times 50{,}000 = 25{,}000 \text{ lb/in}^2 \quad \text{(eq. TS14R–48)}$$

$$S_{min} = \frac{34{,}149 \text{ ft-lbs} \times 12 \text{ in/ft}}{25{,}000 \text{ lb/in}^2} = 16.39 \text{ in}^3\text{/ft of wall} \quad \text{(eq. TS14R–49)}$$

Steel sheet pile wall CWALSHT program

The steel sheet pile should be Grade 50 steel with a minimum section modulus of 16.4 cubic inches per foot of wall.

Given: The steel sheet pile is described in the previous example.

Determine:

Design embedment depth

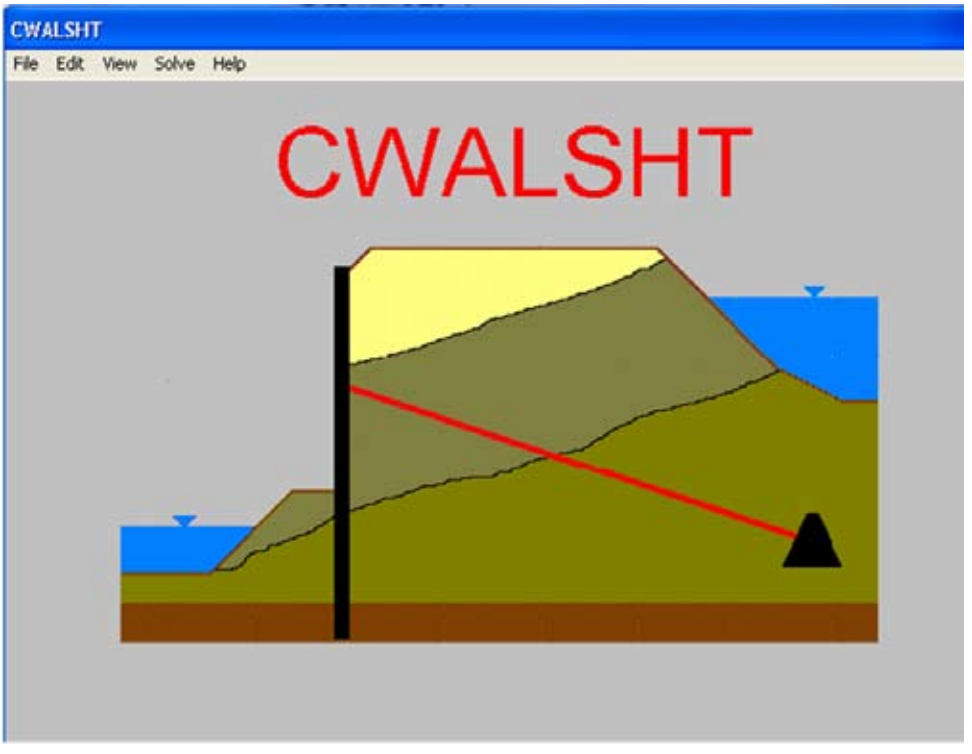
Maximum moment and shear in the pile

Minimum recommended steel sheet pile wall properties

Estimate the deflection of the top of the pile

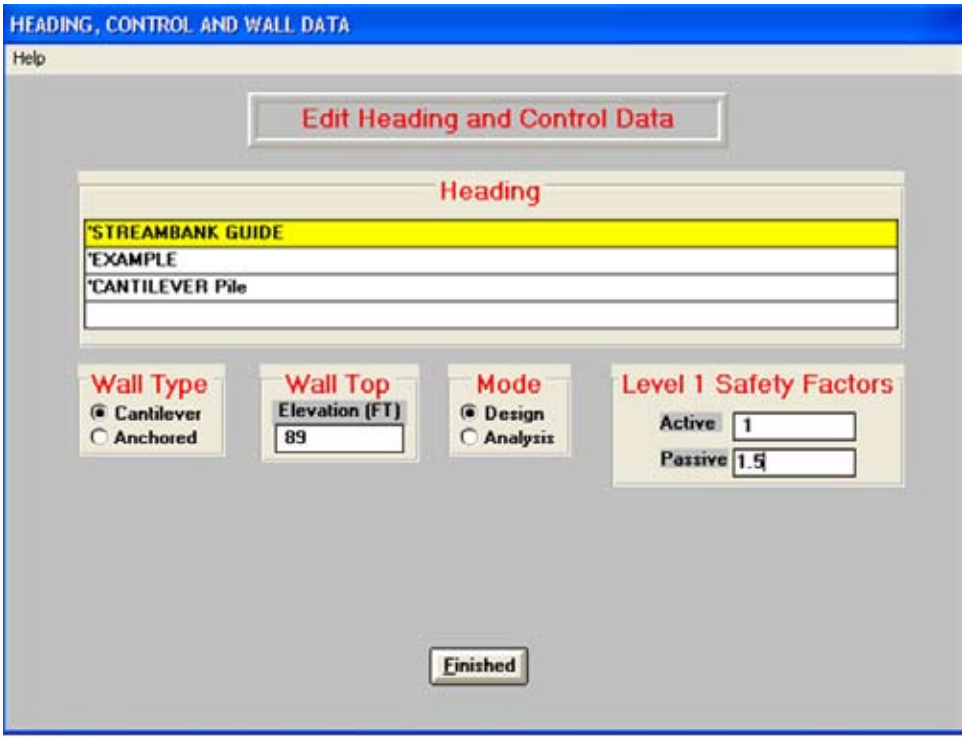
Solution:

Using CWALSHT, enter the data for the analysis.

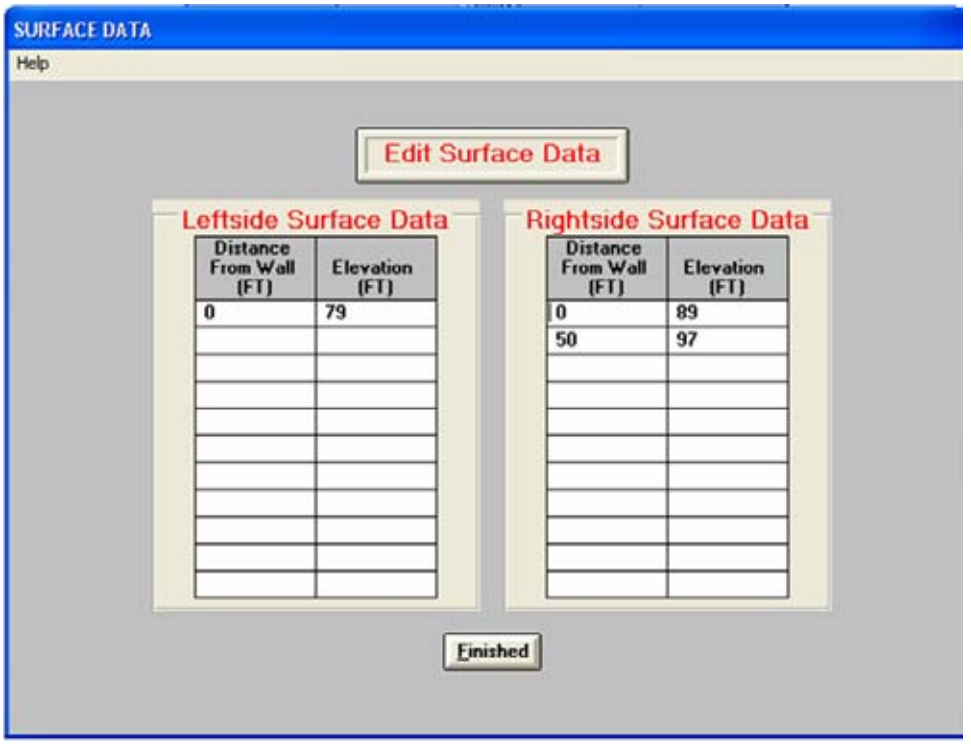


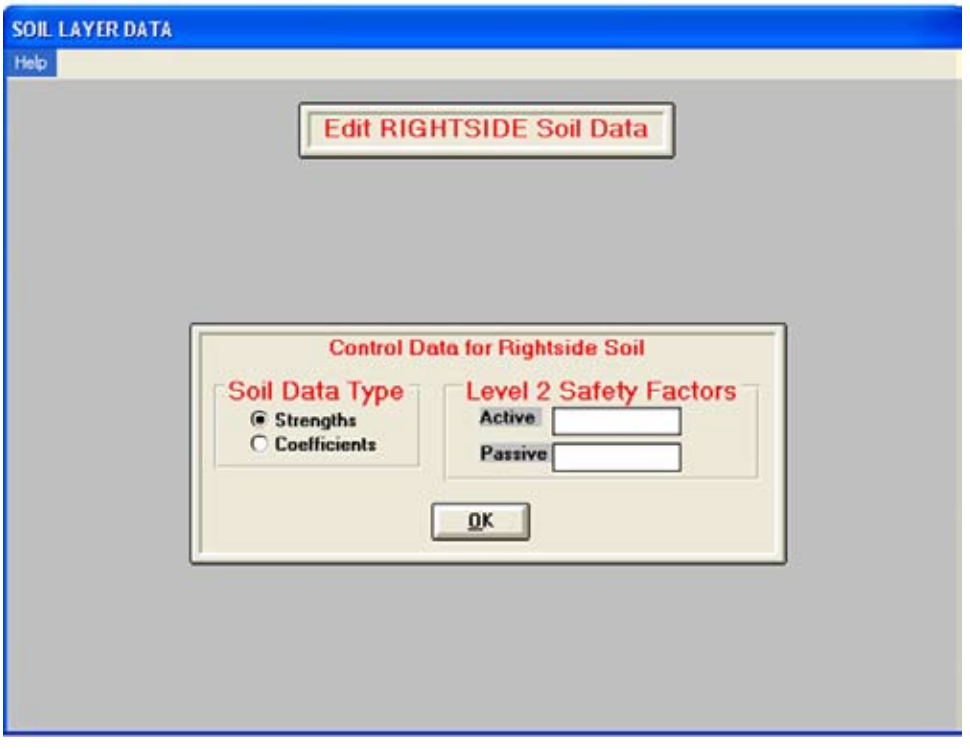
A factor of safety of 1.5 on the passive soil pressure is recommended for this usual load condition. Since the soils are silty sand with a moderate permeability, they will exhibit drained (long-term, effective) behavior.

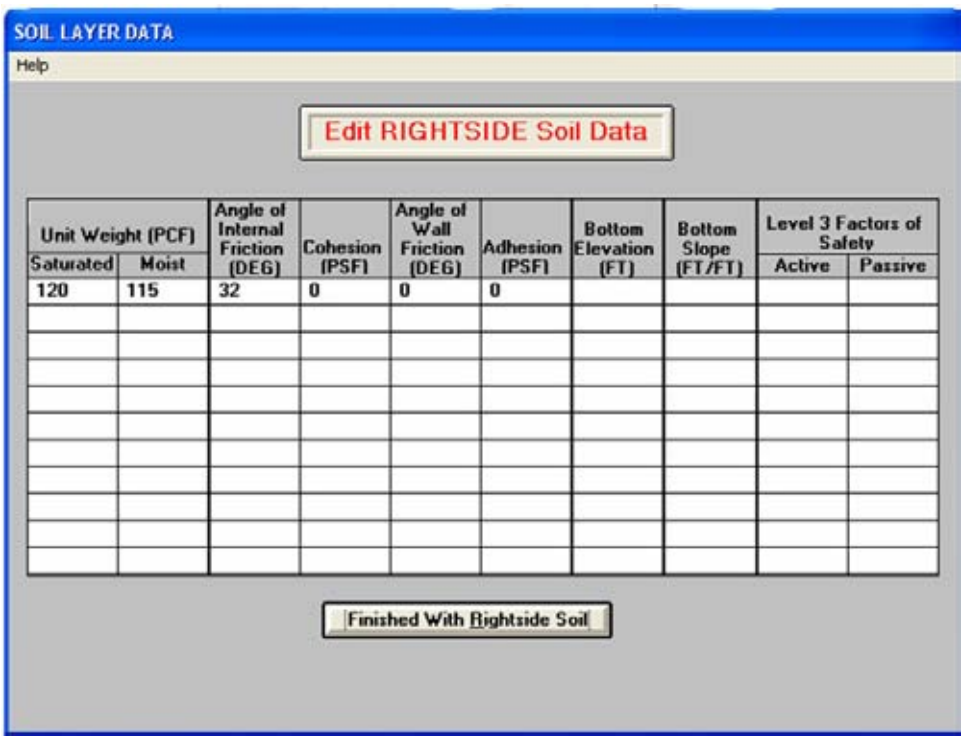
Enter the surface data for both the left side and right side. CWALSHT requires the pile to be loaded from the right side.



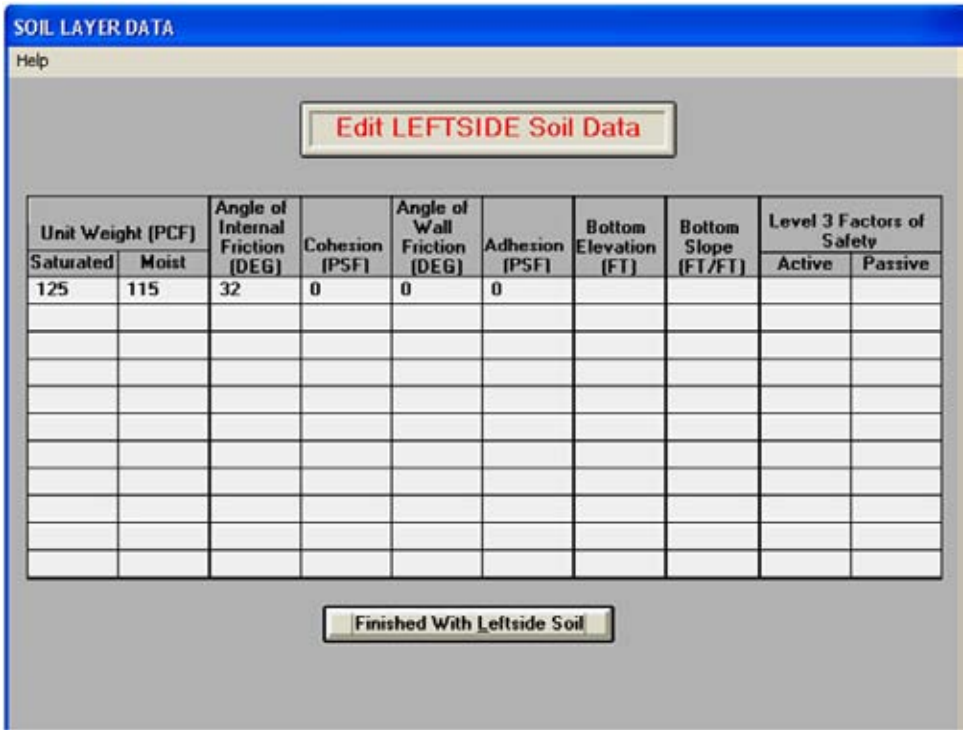
Enter the soils data for both the left and right side. The level 1 factors of safety input previously will apply to the soils data unless level 2 factors are input to override level 1.





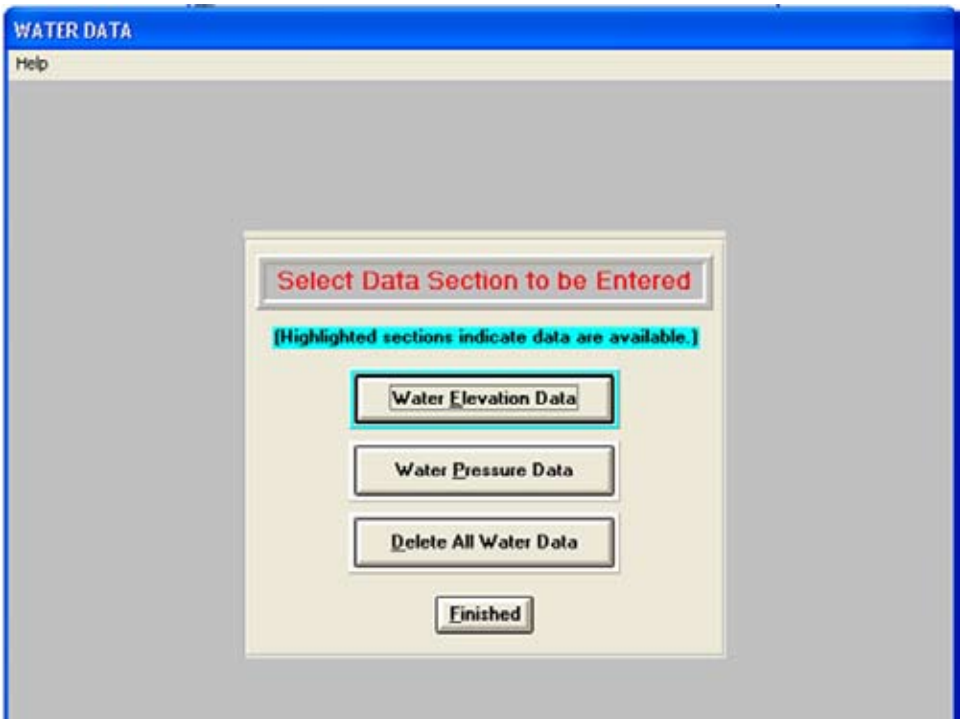


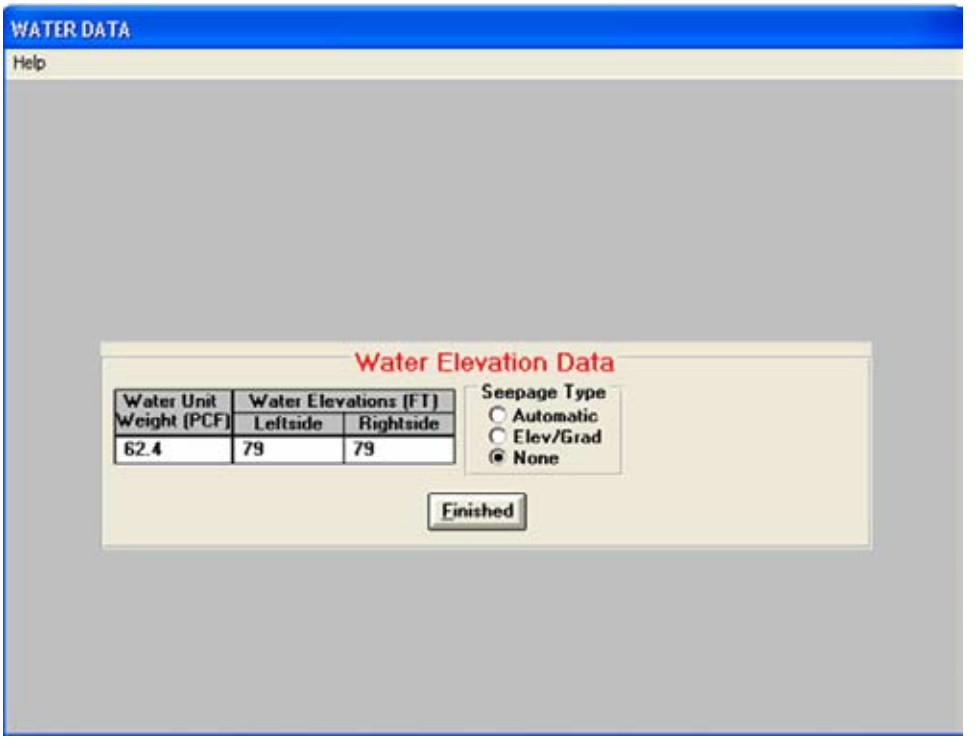
Unit Weight (PCF)		Angle of Internal Friction (DEG)	Cohesion (PSF)	Angle of Wall Friction (DEG)	Adhesion (PSF)	Bottom Elevation (FT)	Bottom Slope (FT/FT)	Level 3 Factors of Safety	
Saturated	Moist							Active	Passive
120	115	32	0	0	0				



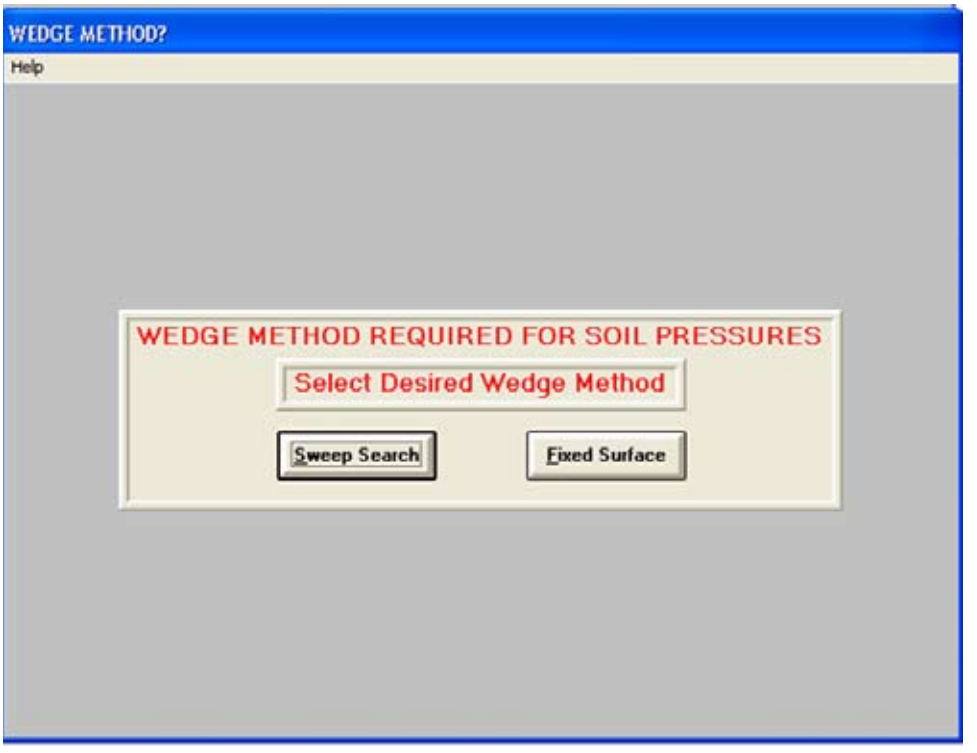
Unit Weight (PCF)		Angle of Internal Friction (DEG)	Cohesion (PSF)	Angle of Wall Friction (DEG)	Adhesion (PSF)	Bottom Elevation (FT)	Bottom Slope (FT/FT)	Level 3 Factors of Safety	
Saturated	Moist							Active	Passive
125	115	32	0	0	0				

Enter the water elevation data.





The data input for this problem is complete. The problem may be solved with the ***sweep*** search, fixed wedge method, or both may be run separately.



The fixed surface method will be used and the solution continued and completed. The output may be viewed to determine the required depth of the pile (fig. TS14R–17).

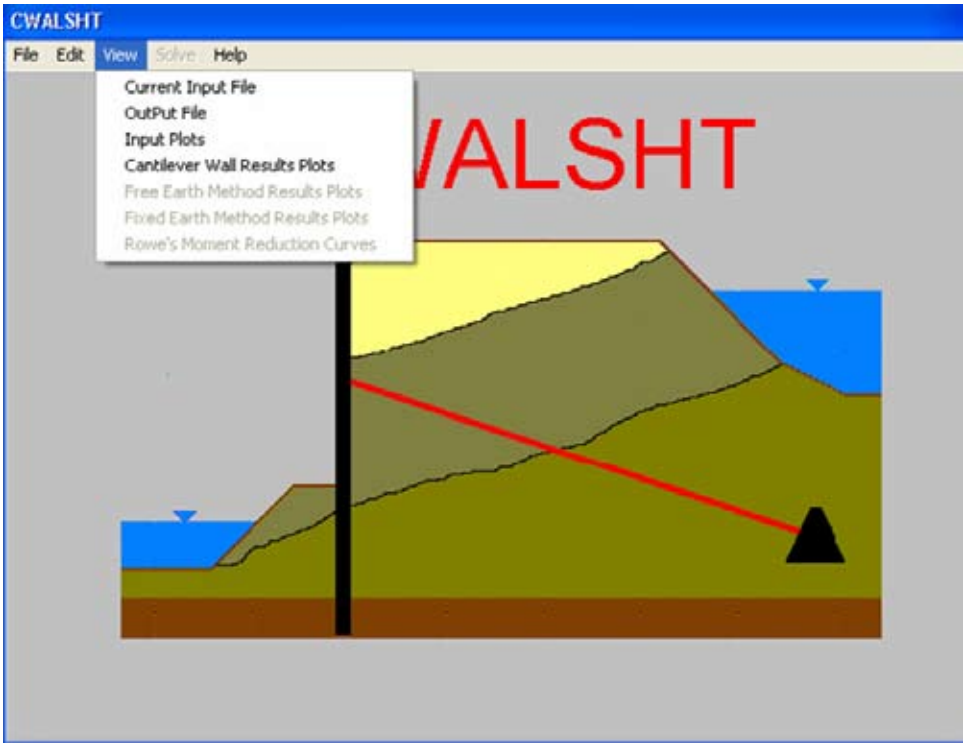


Figure TS14R–17 CWALSHT output file

```
II.--SUMMARY
           RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.
           LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
           AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.
      WALL BOTTOM ELEV. (FT)      : 61.47
           PENETRATION (FT)       : 17.53
      MAX. BEND. MOMENT (LB-FT)   : 2.5262E+04
           AT ELEVATION (FT)      : 69.22
      MAX. SCALED DEFL. (LB-IN3)  : 9.6124E+09
           AT ELEVATION (FT)      : 89.00
                 NOTE:  DIVIDE SCALED DEFLECTION MODULUS OF
                        ELASTICITY IN PSI TIMES PILE MOMENT
                        OF INERTIA IN IN4 TO OBTAIN DEFLECTION
                        IN INCHES.
```

So the design embedment depth using a factor of safety on passive soil pressure is 17.53 feet.

The maximum moment and shear should be determined using a factor of safety of 1.0 in the CWALSHT program. The results are:

```
II.--SUMMARY
           RIGHTSIDE SOIL PRESSURES DETERMINED BY FIXED SURFACE WEDGE METHOD.
           LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
           AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.
      WALL BOTTOM ELEV. (FT)      : 65.72
           PENETRATION (FT)       : 13.28
      MAX. BEND. MOMENT (LB-FT)   : 1.9080E+04
           AT ELEVATION (FT)      : 71.81
      MAX. SCALED DEFL. (LB-IN3)  : 5.1147E+09
           AT ELEVATION (FT)      : 89.00
                 NOTE:  DIVIDE SCALED DEFLECTION MODULUS OF
                        ELASTICITY IN PSI TIMES PILE MOMENT
                        OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
                        IN INCHES.

      PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
                                  BY CLASSICAL METHODS
DATE: 14-DECEMBER-2004TIME: 14:08:44
                            ****************************
                            *  COMPLETE OF RESULTS FOR *
                            *   CANTILEVER WALL DESIGN *
                            ****************************
           I.--HEADING
           ,STREAMBANK GUIDE
           ,EXAMPLE
           ,CANTILEVER Pile
```

Figure TS14R–17 CWALSHT output file—Continued

```
II.--RESULTS

              BENDING       SCALED        NET
ELEVATION     MOMENT        SHEAR      DEFLECTION     PRESSURE
   (FT)       (LB-FT)        (LB)       (LB-IN^3)      (LB/FT^2)
89.00      0.0000E+00            0.  5.1147E+09           0.00
88.00      6.4623E+00           19.  4.7876E+09          38.77
87.00      5.1698E+01           78.  4.4606E+09          77.55
86.00      1.7448E+02          174.  4.1336E+09         116.32
85.00      4.1358E+02          310.  3.8070E+09         155.09
84.00      8.0778E+02          485.  3.4811E+09         193.87
83.00      1.3958E+03          698.  3.1567E+09         232.64
82.00      2.2166E+03          950.  2.8347E+09         271.41
81.00      3.3087E+03         1241.  2.5165E+09         310.26
80.00      4.7111E+03         1570.  2.2041E+09         348.96
79.00      6.4616E+03         1936.  1.8999E+09         383.19
78.00      8.5600E+03         2231.  1.6069E+09         205.14
77.00      1.0863E+04         2345.  1.3288E+09          22.60
76.88      1.1153E+04         2346.  1.2956E+09           0.00
76.00      1.3188E+04         2276.  1.0694E+09   -160.07
75.00      1.5353E+04         2024.  8.3278E+08   -342.67
74.00      1.7176E+04         1590.  6.2264E+08   -525.27
73.00      1.8473E+04          974.  4.4211E+08   -707.87
72.00      1.9063E+04          175.  2.9340E+08   -890.47
71.00      1.8762E+04   -807.        1.7750E+08   -1073.07
70.00      1.7388E+04   -1971.       9.3872E+07   -1255.67
69.00      1.4758E+04   -3318.       4.0106E+07   -1438.27
68.00      1.0690E+04   -4848.       1.1635E+07   -1620.87
67.36      7.2451E+03   -5923.       3.5387E+06   -1737.81
67.00      5.0536E+03   -6126.       1.4035E+06         612.96
66.00      3.2402E+02   -2244.       3.7195E+03        7150.96
65.72      0.0000E+00   0.           0.0000E+00        8970.81

NOTE:  DIVIDE SCALED DEFLECTION MODULUS OF
       ELASTICITY IN PSI TIMES PILE MOMENT
       OF INERTIA IN IN^4 TO OBTAIN DEFLECTION
       IN INCHES.
```

The maximum moment estimated by the CWALSHT program is 19,080 feet-pounds and maximum shear of 6,126 pounds at elevation 77.

The difference in the analysis completed by hand calculations, and the CWALSHT analysis is due to the method of estimating the active and passive earth pressure.

The minimum section modulus of 50 kilopounds per square inch steel per foot of wall is:

$$S_{min} = \frac{M_{max}}{f_b} \qquad \text{(eq. TS14R–50)}$$

$$f_b = 0.5 f_y = 0.5 \times 50{,}000 = 25{,}000 \text{ lb/in}^2 \qquad \text{(eq. TS14R–51)}$$

$$S_{min} = \frac{19{,}080 \text{ ft-lb/ft of wall} \times 12 \text{ in/ft}}{25{,}000 \text{ lb/in}^2} = 9.15 \text{ in}^3\text{/ft of wall} \qquad \text{(eq. TS14R–52)}$$

A CZ–67 pile will provide a section modulus of 10.69 cubic inches per foot of wall. The thickness of a CZ–67 is 0.217 inches. A minimum thickness of 0.25 inches is recommended. A CZ–84 provides a thickness of 0.276 inches and section modulus of 13.62 cubic inches per foot of wall.

The minimum shear area per foot of wall is:

$$A_{v,min} = \frac{V_{max}}{f_v} \qquad \text{(eq. TS14R–53)}$$

$$f_v = 0.33 f_y = 0.33 \times 50{,}000 = 16{,}500 \text{ lb/in}^2 \qquad \text{(eq. TS14R–54)}$$

$$A_{v,min} = \frac{6{,}126 \text{ lb/ft of wall}}{16{,}500 \text{ lb/in}^2} = 0.37 \text{ in}^2\text{/ft of wall} \qquad \text{(eq. TS14R–55)}$$

$$A_v \text{ of Z-shaped piles} = t_w \times \frac{h}{w}$$

$$A_v \text{ of u-shaped piles} = 2t_w \times \frac{h}{w}$$

$$\begin{aligned} A_v \text{ of CZ-84} &= t_w \times \frac{h}{w} \\ &= 0.276 \times \frac{7.88}{\left(\frac{21.65}{12}\right)} \\ &= 1.2 \text{ in}^2\text{/ foot of wall} > 0.37 \end{aligned} \qquad \text{(eq. TS14R–56)}$$

The steel sheet pile should be Grade 50 steel with a minimum section modulus of 13.6 cubic inches per foot of wall.

The deflection of the top of the pile may be determined by using the analysis mode of CWALSHT and inputting the actual pile properties or by dividing the scaled deflection provided by the design mode by the modulus of elasticity of the steel and the moment of inertia per foot of pile.

$$\begin{aligned} \text{Deflection} &= \frac{\text{Scaled deflection}}{(E \times I)} \\ &= 5.1147E + \frac{9}{(29{,}000{,}000 \times 53.63)} \\ &= 3.28 \text{ in} \end{aligned} \qquad \text{(eq. TS14R–57)}$$

Technical Supplement 14S

Sizing Stream Setbacks to Help Maintain Stream Stability

(210–VI–NEH, August 2007)

Issued August 2007

Cover photo: In some cases, stream restoration may be achieved by allowing the stream to adjust itself within the riparian area and flood plain. What are the migration boundaries, and where should development be restricted to allow this adjustment are questions that need to be tempered by local requirements, land use and ownership, and the community's restoration goals and objectives.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Technical Supplement 14S

Sizing Stream Setbacks to Help Maintain Stream Stability

Contents

Purpose	**TS14S–1**
Introduction	**TS14S–1**
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The need for setbacks	**TS14S–1**
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Conclusion	**TS14S–3**

Tables

Table TS14S–1 Recommended width for vegetated riparian zones TS14S–2

Technical Supplement 14S

Sizing Stream Setbacks to Help Maintain Stream Stability

Purpose

Many local communities, watershed groups, counties, and states are developing setback ordinances to help protect stream systems. Several guidelines are outlined in this technical supplement. This technical supplement also presents an empirically based equation that calculates the streamway width required to allow a stream to self-adjust its meander pattern. This technical supplement does not cover stream setbacks that are required by local or state laws, nor does it address conservation program requirements.

Introduction

Although many local communities, watershed groups, counties, and states are developing setback ordinances to help protect stream systems, existing guidelines were developed on the basis of different, often nebulous, objectives and are highly variable. In an effort to provide maximum protection or to establish easily understood ordinances, setbacks have ranged from mandating no development in the 100-year flood plain to a fixed setback width (such as 100 ft) that may be unrelated to the stream size or drainage area. As these approaches are only loosely related to stream morphology, if at all, they will provide widely variable levels of effectiveness, underestimating or overestimating the area most vital to maintaining the integrity of streams.

Flood plain function

Flood plains of ecologically healthy streams are characterized by frequent, extensive over-bank flow. Fluvial processes size the main channel to convey the effective (bankfull) discharge, and larger flows spread out onto the flood plain. Abandoned channels or adjacent terraces may have been the active flood plain in the past. Flood plains and adjacent terraces are a complex system of soil, bedrock, vegetation, and subsurface water that affect water quality, wildlife habitat, instream habitat enhancement, recreation, and aesthetics (Large and Petts 1994).

While the flood plain provides important ecologic and pollutant filtration purposes, stream stability depends on the flood plain for the following:

- dissipation of energy of flows exceeding the effective discharge (bankfull)
- sediment transport, storage, and supply—most importantly bedload sediment
- ability of the main channel to adjust its dimension, pattern, and profile, maintaining a dynamic equilibrium

The need for setbacks

Land use change on the landscape often increases the magnitude and volume of discharge, encroaches on the flood plain, and increases stream conveyance by channel lowering, widening, and straightening. The impact of these changes on the stability, ecological function, and general health of the river system is very site specific. Unfortunately, efforts to establish simple, but universal, river corridor protection guidelines or requirements are often arbitrary (table TS14S–1). A useful review of the literature on riparian zone characteristics is presented by Wenger (1999).

Calculating the streamway width

Many empirical equations have been developed to describe bankfull (effective discharge) channel geometry. One such equation by Williams (1986) relates meander belt width (B, ft) and the bankfull width (W, ft) (eq. TS14S–1) (converted from the metric form):

$$B = 5.0\ W^{1.12} \qquad \text{(eq. TS14S–1)}$$

where:
B = belt width (ft)
W = bankfull width (ft)

An equation for the relationship between bankfull channel width and drainage area (DA, square miles) for rivers in the eastern United States (Dunne and Leopold 1978) gives:

$$W = 14.6\ DA^{0.38} \qquad \text{(eq. TS14S–2)}$$

Substituting equation TS14S–2 into equation TS14S–1 provides a relationship for the belt width as a function of drainage area:

$$B = 100\ DA^{0.43} \qquad \text{(eq. TS14S–3)}$$

In developing an equation that might be used to define stream setbacks, it is also important to provide a:

- flood plain that is wide enough to accommodate the existing meander pattern
- flood plain that would accommodate future meander migration that might occur
- factor of safety to account for uncertainty since the equation is based on empirical equations that do not account for all the variability in data used in their development
- minimum level of protection on both banks of the stream

Equation TS14S–1 is based on 153 data points from rivers around the world. The correlation coefficient (r) for the equation is 0.96. Belt width and bankfull width data for 47 of the locations are presented by Williams (1986). The data have been analyzed, and the regression equation that was obtained is very similar to equation TS14S–1. This equation underpredicted the belt width by a mean amount of 24 percent for 24 of the sites and overpredicted the belt width by a mean amount of 36 percent for 23 of the sites.

Overprediction is not a major concern since the method does not attempt to account for meander migration or riparian zone protection. However, without modification, the equation fails the setback requirements at least half the time, so the calculated belt widths have been evaluated in increasing increments of 10 percent. A 10 percent increase reduced the number of sites where the belt width was underpredicted from 24 to 17, while a 20 percent increase reduced the number of sites where the belt width was underpredicted from 24 to 12. Additional increases up to 50 percent only reduced the number of underpredicted sites from 24 to 8. However, an increase of this magnitude resulted in a mean overprediction of 74 percent in the belt width size at the 39 sites where the equation overpredicted the belt width. Based on this analysis, the following equation is obtained that increases the estimated beltway obtained from equation TS14S–4 by 20 percent:

$$\text{Streamway} = 120\ DA^{0.43} \qquad \text{(eq. TS14S–4)}$$

Table TS14S–1 Recommended widths for vegetated riparian zones

Function	Study	Relevant details	Width (ft)
Riparian habitat areas	Washington Department of Fish and Wildlife (2001)	Fish and wildlife-based review of 1,500 articles	150- to 250- or 100-yr flood plain
Wildlife protection	Rabeni (1991)[1/]	Fish, amphibians, birds	25–200
	Cross (1985)[1/]	Small mammals	30–60
	Brown, Schafer, and Brandt (1990)[1/]	Provide food, water, cover	300–600
Water quality	Ahola (1989)[1/]	General improvements	160
	Pinay and Descamps (1988)[1/]	As above	3–6
	Correll and Weller (1989)[1/]	Nitrate control	About 60
Sediment control	Peterjohn and Correll (1984)[1/]	Nutrient control	About 60
Bank stabilization	Ontario Ministry of Agricultural, Food, and Rural Affairs (1998)	Agricultural ditch bank stabilization	10
Urban stream buffer	Schueler (1995)	Survey of 36 buffer programs	20–200

1/ As cited by Large and Petts (1994)

Equations TS14S–3 and TS14S–4 only apply to the eastern United States. The uncertainty associated with equation TS14S–2 was not evaluated, and it was not possible to evaluate equations TS14S–3 and TS14S–4 with the data published by Williams (1986) because drainage area data were not presented.

Conclusion

Successful stream stewardship requires combining knowledge of natural stream concepts with sound engineering and scientific principles and an understanding and appreciation of the ecology of the stream and its interaction with the landscape. A stream stability protection setback should be based on stream geomorphology concepts and specifically the ability of the stream to self-adjust and maintain itself in a state of dynamic equilibrium.

For the setback to accomplish the goal of the impacted stream to sustain dynamic-equilibrium also requires the incorporation of:

- landscape measures that reduce runoff such as reduction in paved surface area and practices to maintain or enhanced infiltration
- detention/retention management strategies that result in similar post and predevelopment bedload and sediment transport amounts

Case Studies

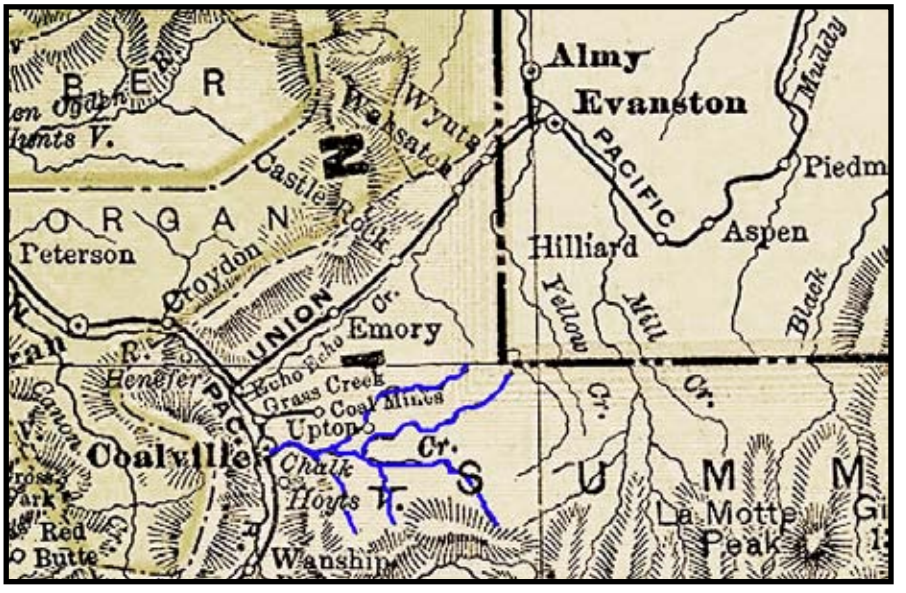
Issued August 2007

Cover photo: When selecting a design approach, it is often informative to examine how different practitioners have approached project design in similar conditions.

Introduction

Numerous case studies are presented that represent a wide assortment of approaches and instream treatments. Project goals also vary widely and, in some cases, these goals and objectives were not completely achieved. These case studies represent locations all over the United States in an attempt to represent some of the regionally sensitive challenges. Physical, chemical, and biological variations are expected. The authors of each case study are identified to allow the reader to follow up with an individual or organization if more information is needed. The opinions expressed are those of the authors.

Case Study 1 Chalk Creek, Summit County, Utah



Issued August 2007

Cover photo: Stream restoration practices were used to prevent streambank erosion and improve fish habitat in this stream.

Advisory Note

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Case Study 1 Chalk Creek, Summit County, Utah

By Shane Green, Area Range/Riparian Specialist, U.S. Department of Agriculture, Natural Resources Conservation Service, Utah

Overview

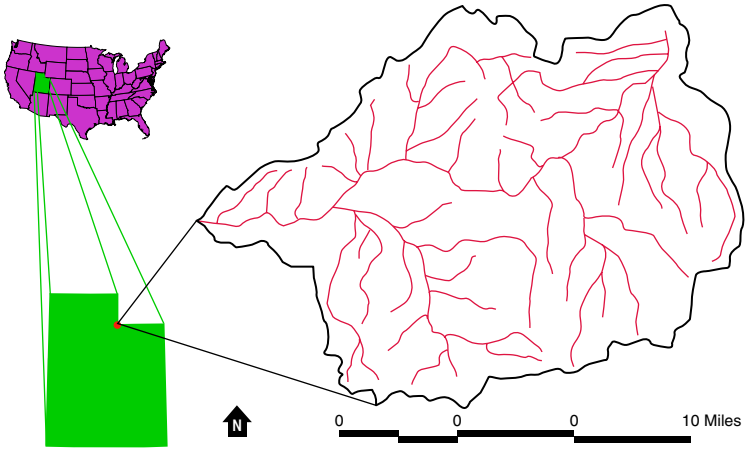
The Chalk Creek Watershed (fig. CS1–1) encompasses 172,000 acres in Summit County, Utah, 95 percent of which is rangeland. The Chalk Creek Nonpoint Source Water Quality Project began in 1991, when the Summit Soil Conservation District (SCD) organized a local steering committee to provide planning guidance decisions. This effort was in response to Chalk Creek being listed on the state 303(d) list (Clean Water Act) with sediment being the primary impairment to water quality. The committee consists of elected officials, landowners, wildlife groups, irrigation companies, and state and Federal agency personnel. The Coordinated Resource Management Planning (CRMP) process was followed and a Coordinated Resource Management plan (CRM) was published in 1994. Since that time, about $3.2 million have been spent in improvements involving 90 landowners and 84,092 acres within the Chalk Creek Watershed.

Trout habitat is one of the key resource concerns in Chalk Creek. The watershed holds the largest documented population of Bonneville cutthroat trout (fig. CS1–2) yet discovered at that time.

Landowners installed practices that have improved Chalk Creek's water quality and the overall health of the watershed. They voluntarily adopted conservation practices such as sprinkler irrigation systems, streambank protection, grazing management, riparian fences, and mine reclamation to control erosion and reduce runoff of sediments into Chalk Creek. This case study focuses on one of the streambank protection projects undertaken in this watershed.

Figure CS1–2 Bonneville cutthroat trout

Figure CS1–1 Chalk Creek Watershed



Chalk Creek

Example streambank protection project

In this case study, the landowner had recently acquired the property and was concerned about the amount of irrigated pastureland he was losing annually to bank erosion adjacent to Chalk Creek. In one instance, the stream channel encroached approximately 30 feet into the pasture during a single snowmelt runoff (fig. CS1–3, treatment section #4). Resource inventory revealed that past practices had removed woody riparian vegetation from many of the banks. Also, some apparent nick points or overfalls in the channel indicated active downcutting. Evidence of past channel dredging was apparent on a few reaches. At one location, a new bridge constricted the flow in the channel.

Design

In 1995, the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) designed streambank protection for this landowner. This project involved installing rock riprap barbs, juniper revetments, willow plantings, and low rock/vortex grade control structures. The project was an NRCS job class VI due to drainage area size. Figure CS1–4 shows typical project details. Streambank soil bioengineering practices are addressed in NEH654 TS14I.

The project involved stabilizing a 3,840-foot reach of Chalk Creek. Rock barbs were designed at places where active bank erosion was occurring. Willow plantings and juniper revetments were placed between rock barbs. These structures were needed to prevent bank erosion. On three of the treatment areas, berms were constructed to function as new streambanks protected with rock barbs, juniper revetments, and willow plantings. The borrow pits for the berm construction were adjacent to the stream and function as ponds. Existing vegetation was maintained on remaining areas.

The Chalk Creek drainage area at this location is approximately 156 square miles. The 25-year, 24-hour stormflow is approximately 1,450 cubic feet per second. Design bankfull flow is approximately 405 cubic feet per second, and bankfull width is 35 feet. This watershed typically has one annual channel-forming flow during snowmelt runoff.

The project was designed in accordance with the following NRCS Conservation Practice Standards in the Field Office Technical Guide: Streambank and Shoreline Protection (580), Channel Stabilization (584), and Channel Bank Vegetation (322).

The slope of the creek, channel width, alignment, and cross section were determined by field surveys. Utah Engineering Technical Note #7 was used to size and field-locate sites for the rock barbs. Rock was sized using criteria in Far West Design Standards. Rock gradation was selected based on criteria that the D_{100} is two to three times the median D_{50} rock size. The streambed materials at this site are a mixture of cobble, gravel, and sand. The streambank materials are a mixture of loam, sand, and gravels.

The low rock/vortex grade control structures were located in channel crossovers where active downcutting was occurring. Seven structures were located, based on field evaluation and an analysis of the survey. Typical criteria are to limit the drop per structure to 1 foot. The rock size was evaluated using criteria in Engineering Field Manual, chapter 16, Streambank and Shoreline Protection, allowing for debris and an impact factor.

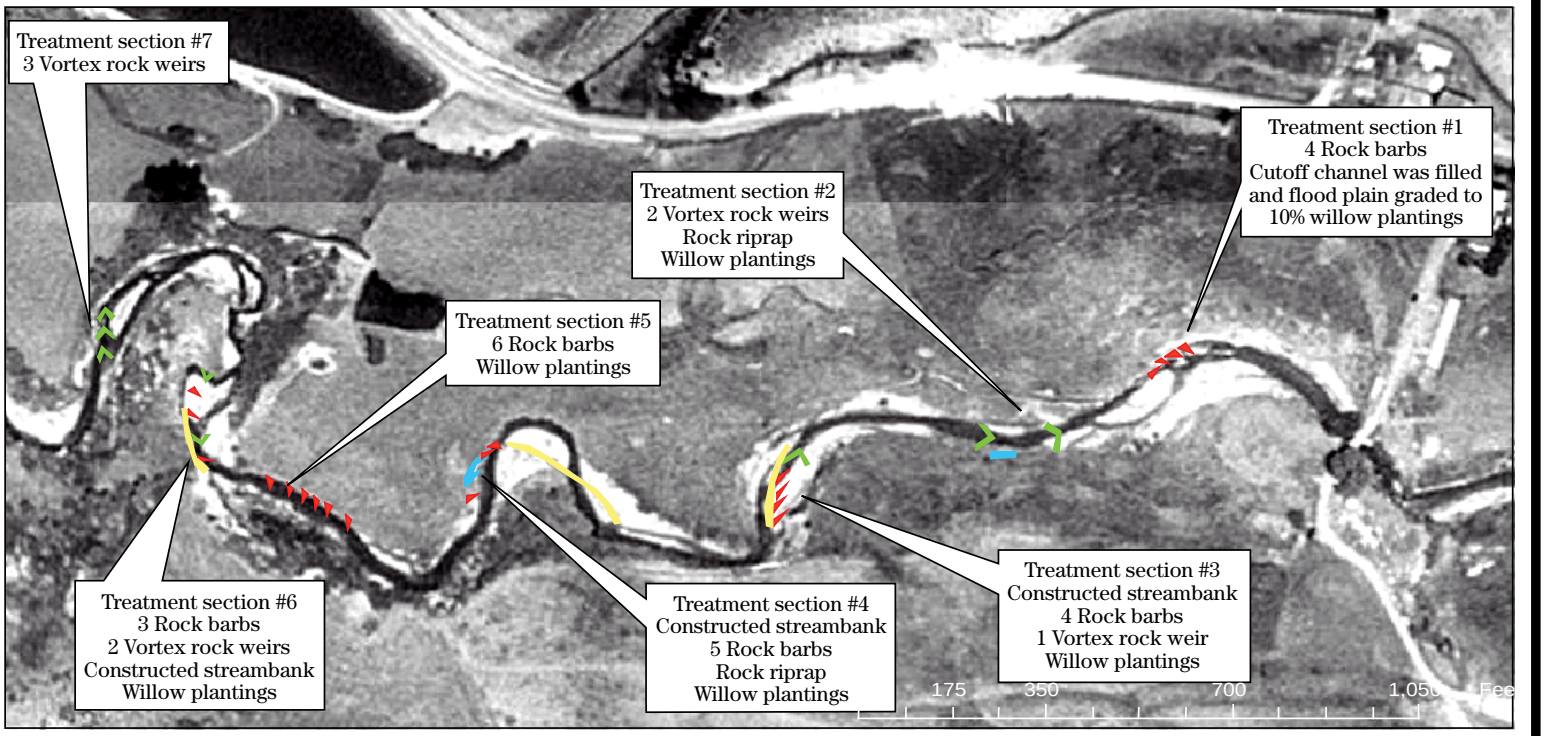
Chalk Creek Case Study design data

Rock gradation was determined using Utah Engineering Tech Note #7.

D_{100}= max 36 in, D_{75} = min 12 in
D_{100}= $2\times D_{50}$ = 24 in to $3\times D_{50}$ = 36 in
D_{75} = $1.5\times D_{50}$ = 18 in to $2.5\times D_{50}$ = 30 in
D_{50} = $1\times D_{50}$ = 12 in to $1.75\times D_{50}$ = 21 in
D_{25} = $.5\times D_{50}$ = 6 in to $1.15\times D_{50}$ = 14 in

% Passing	
100	36 in
100–75	24 in
75–50	18 in
50–25	12 in
25–0	6 in

Figure CS1–3 Chalk Creek example project, pretreatment view, 1989



N

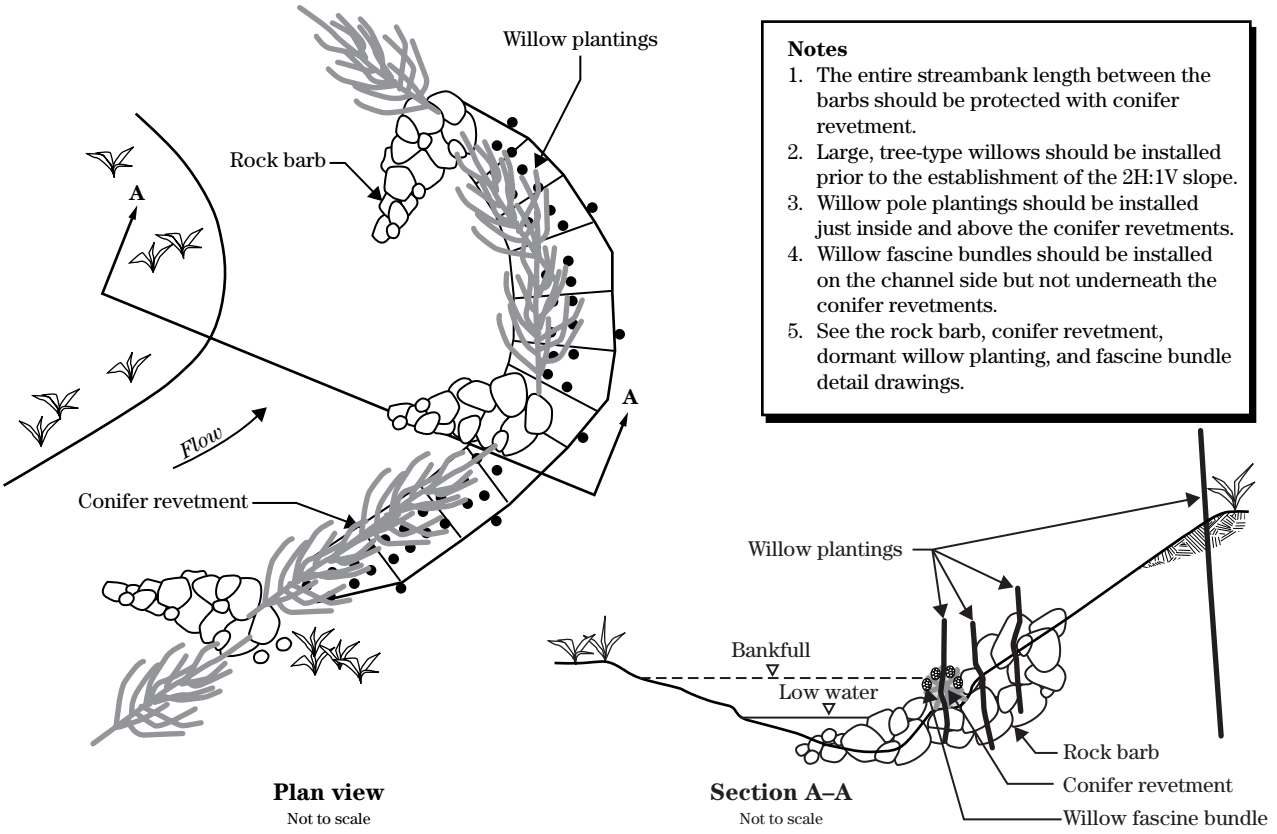
Treatment section #1 - the actively eroding bank was protected with a combination of rock barbs and willow plantings. A braided channel condition apparent in the photo was corrected by filling the cutoff channel and grading the flood plain to 10%.
Treatment section #2 - vortex rock weirs were installed to prevent downcutting following the installation of a bridge and the resulting constriction of the flood plain (see 2001 photo). A small section of rock riprap was installed adjacent to a vulnerable bridge abutment structure.
Treatment section #3 - an overwidened and braided channel condition was corrected by installing a berm, or constructed streambank. This structure was protected by rock barbs, and a vortex weir was installed to prevent downcutting.
Treatment section #4 - an oversized meander condition was corrected by installing a berm, or constructed streambank. Actively eroding banks protected with rock barbs, and later rock riprap was installed between two of the barbs (see 2001 photo).
Treatment section #5 - an actively eroding streambank was protected with rock barbs.
Treatment section #6 - a berm was constructed at a site on the streambank that was a risk for cutting off a very large meander shown in the photo. In 1995, a large runoff event began cutting a new channel that threatened to cut off this large meander. The eroding streambanks were protected with rock barbs, and a widened and braided channel was corrected by grading the flood plain to 10%. Two vortex rock weirs were installed to stop an active nick point from further downcutting.
Treatment section #7 - active nick points were prevented from further downcutting and upstream movement with three vortex rock weirs.

Legend

Practices

- Rock barbs
- Vortex rock weirs
- Constructed streambank
- Riprap

Figure CS1–4 Typical reach detail



Notes

1. The entire streambank length between the barbs should be protected with conifer revetment.
2. Large, tree-type willows should be installed prior to the establishment of the 2H:1V slope.
3. Willow pole plantings should be installed just inside and above the conifer revetments.
4. Willow fascine bundles should be installed on the channel side but not underneath the conifer revetments.
5. See the rock barb, conifer revetment, dormant willow planting, and fascine bundle detail drawings.

From the field survey:

- channel slope: 0.01 ft/ft
- bankfull width: minimum 45 ft, maximum 85 ft
- bankfull depth: minimum 2.2 ft, maximum 3.4 ft
- radius of curvature: minimum 40 ft, maximum 210 ft
- rock size calculated using the Far West Design Standards 6–13b:

$$D_{75} = \frac{3.5\gamma_w dS}{CK}$$

γ_w = specific weight of water = 62.4 lb/ft^3
d = bankfull depth = 3.4 ft
S = channel slope = 0.01 ft/ft
K = 0.72 for 2H:1V slope = 0.72
C = radius of curvature/width = 0.6

$$D_{75} = 18 \text{ in}$$

$$D_{50} = \frac{D_{75}}{1.5} = 12 \text{ in}$$

$$D_{100} = 3 \times D_{50} = 36 \text{ in}$$

All of the rock barb installations followed the typical reach detail shown in figure CS1–3 with the attendant conifer revetment and willow plantings. Willow planting techniques included pole plantings as shown on all areas, with willow brush blankets and willow fascines in selected areas (Bentrup and Hoag 1998).

The benchmark condition included livestock access to the stream from the adjacent irrigated pastures. The conservation plan included corridor fencing to facilitate livestock exclusion from the stream during the establishment and recovery of the woody riparian vegetation. Prescribed grazing is planned in the riparian area, following the recovery period for an early spring use only, to favor the woody riparian vegetation. Nutrient management, prescribed grazing, and irrigation water management were also components of the conservation plan for the adjacent irrigated pastures.

Project costs

The total cost for this case study was $41,934. A total of 1,484 feet of stream was actually treated within the 3,840-foot reach on this property. This amounts to a cost of $28 per linear foot for the treated sections of stream. Cost data taken from averages of the projects in the Chalk Creek Watershed show that a basic bank protection project as shown in the typical reach layout (rock barbs, conifer revetment, and willow pole plantings) costs about $18 per linear foot. This case study included numerous additions to the typical layout, such as vortex rock weirs, constructed streambanks, and flood plain grading, which resulted in the higher costs. It was difficult to separate the costs of some of the different components of this project because costs were combined on many of the invoices, but an approximation of the component costs is found in table CS1–1.

Project results

The landowner is pleased with the results of this project (figs. CS1–5 through CS1–7). The eminent threat of the stream changing course and cutting off a large meander at treatment section #6 has been alleviated. A few of the installed practices were not successful, however. Most notable was the failure of the barbs to stop bank erosion in treatment section #4. The stream was anticipated to follow the contour of the installed berm, but it started to curve away from the berm in subsequent years (fig. CS1–5). This caused a very tight radius of curvature on the bend where the erosion occurred between the barbs. A more careful analysis of the oversized meander and design of the placement of the berm may have prevented this. Bank erosion has ceased and woody riparian vegetation is recovering

Table CS1–1 Chalk Creek case study costs (at time of construction)

Practice	Cost per unit
Vortex rock weirs	$2,000 ea
Rock barbs	$ 500 ea
Constructed streambank (berm)	$ 11/lf
Willow plantings, dormant pole, 2 row, 3-ft spacing in each row	$ 3/lf
Conifer revetment	$ 4/lf
Rock riprap	$ 50/lf

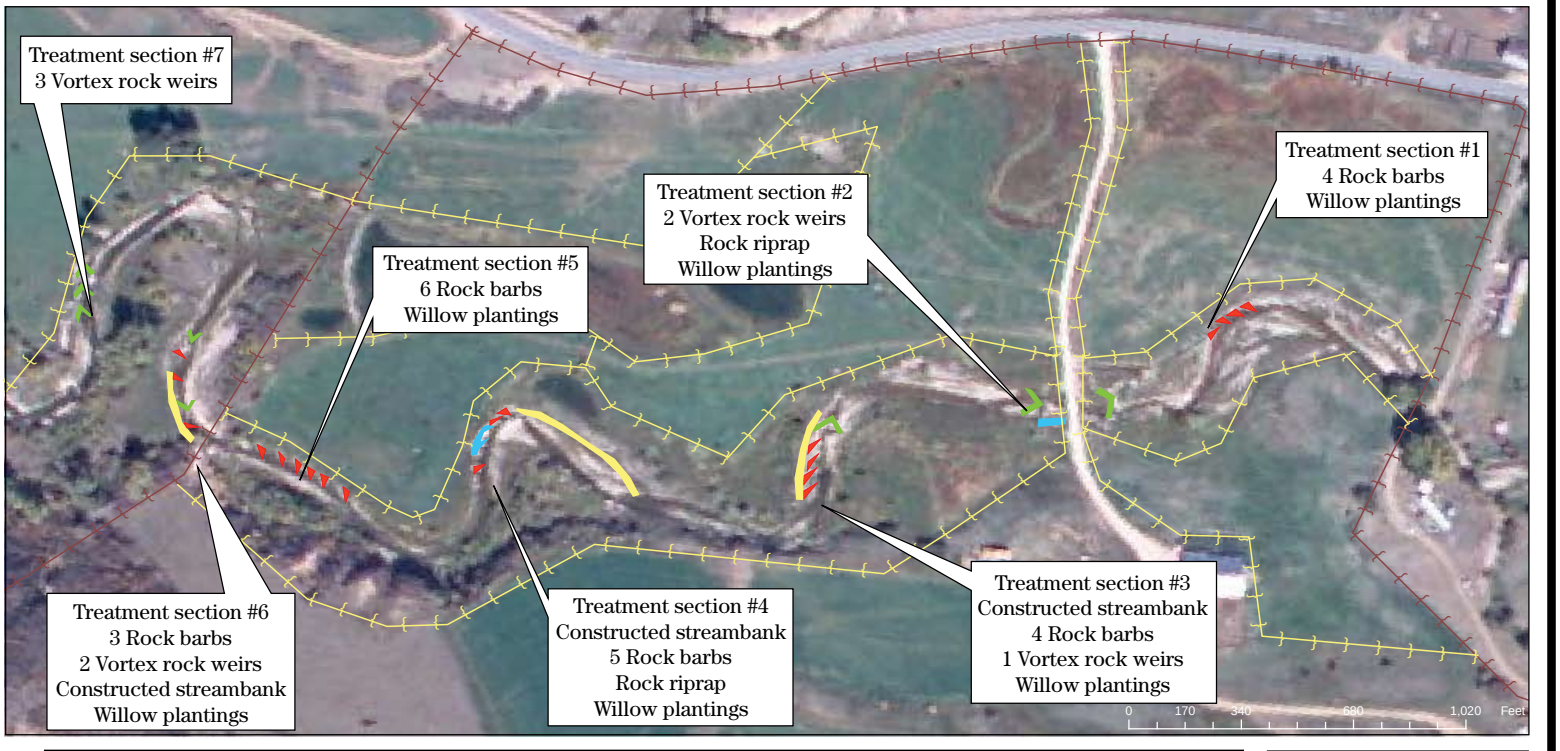
(CS1–6c). Note the function of the rock barbs as the current is focused away from the bank downstream of each barb.

A few of the willow plantings were not successful for various reasons including grazing (not getting the corridor fences built quickly enough). Also, there is some speculation that the willow brush mattress failed due to planting in the fall, rather than the spring. However, the pole plantings and fascines that were installed in the fall were all successful.

Figure CS1–7 shows treatment section #3 before, immediately following, and after treatment. Bank erosion has ceased and woody riparian vegetation is recovering. By 1998, the conifer revetments and rock barbs were necessary to provide protection for the evacuated material used to construct the new streambank. Note the improved width/depth ratio of the channel and removal of the braided condition.

The water quality impairments in Chalk Creek were due to excess sediment and phosphorus. Analysis of long-term water quality monitoring data collected in 1997 by the Utah Department of Environmental Quality shows that measurable reductions in phosphorus and sediment loads have occurred in Chalk Creek since the beginning of the project implementation in 1993. One explanation for this reduction is the implementation of many projects like the one described in this example that have occurred on Chalk Creek since the beginning of the project. Monitoring has continued, and continued improvement is anticipated.

Figure CS1–5 Chalk Creek Example Project, posttreatment view, 2001



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Treatment section #1 - the rock barbs were successful in stopping the active bank erosion; the willow plantings suffered from high mortality.
Treatment section #2 - the vortex weirs were successful in preventing downcutting of the stream. The one upstream was buried with a pulse of bedload and is no longer visible; however, the braided channel condition did not return, and the current channel is closer to the appropriate width to depth ratio. The willow plantings were unsuccessful here.
Treatment section #3 - all structures were successful and remain intact and functioning; willow plantings were also successful. The braided condition of the channel did not return, and the current channel is closer to the appropriate width to the depth ratio.
Treatment section #4 - the constructed streambank at this site remains intact; however, the channel took an unexpected move in the opposite direction (see photo). This resulted in a very small radius of curvature, and active erosion began between two of the barbs. This was corrected by installing rock riprap between these barbs. The willow plantings were successful. The borrow pit where the material was taken to build the berm is now functioning as a pond (see photo).
Treatment section #5 - the rock barbs and willow plantings were successful.
Treatment section #6 - all of the structures and plantings were successful at this site. The braided condition did not return, and the current has an appropriate width-to-depth ratio.
Treatment section #7 - all three weirs that were installed have been completely covered by a pulse of bedload and are no longer visible. The current channel has no apparent nick points and an appropriate width to depth ratio.

Legend

- Rock barbs
- Vortex rock weirs
- Constructed streambank
- Riprap
- Existing fence
- Planned fence

Figure CS1–6 Treatment section #4 (view looking upstream)

(a) 1995, before treatment

(b) 1996, immediately following treatment

(c) 1996, after treatment

Figure CS1–7 Treatment section #3 (view looking downstream)

(a) 1995, before treatment

(b) 1996, immediately following treatment

(c) 1996, after treatment

Case Study 2

Goode Road/Cottonwood Creek, Hutchins, Texas

Issued August 2007

Cover photo: Panoramic view of the completed Cottonwood Creek, Hutchins, Texas, project

Advisory Note

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Case Study 2 Goode Road/Cottonwood Creek, Hutchins, Texas

By Ming-Han Li, Ph.D., P.E., R.L.A., Assistant Professor, Department of Landscape Architecture and Urban Planning, Texas A&M University, College Station, Texas

Introduction

The Goode Road/Cottonwood Creek Project is the first streambank soil bioengineering project implemented by the Texas Department of Transportation (TxDOT). The significance of this case study is that it was implemented successfully in a hot climate area where five challenges were encountered:

- climatic constraint
- plant physiological limitation
- construction schedule conflicts
- shortage of qualified contractors
- insufficient technical guidance

Beginning in 1996, an existing TxDOT bridge on Goode Road in Hutchins, Texas, a few miles southeast of Dallas, was closed due to severe erosion on its abutment and a roadway embankment failure. The bridge, built in the 1960s, crosses Cottonwood Creek and is a single span timber plank structure (fig. CS2–1). In the 1990s, Cottonwood Creek experienced large flows after some storm events, and these flows occasionally overtopped the bridge. Because Goode Road's west-facing embankment is also a streambank of Cottonwood Creek, the roadway embankment was severely eroded by the flow (fig. CS2–2). Specifically, a sewer and a water line crossing the creek caught large amounts of debris, which indirectly led to erosive currents downstream. In 1998, TxDOT sought to reopen Goode Road by replacing the old bridge and stabilizing the streambank. The Texas Transportation Institute (TTI) worked with TxDOT engineers and designed the streambank reinforcement using both soil bioengineering methods and traditional structures. Vegetated geogrids, dormant post plantings, turf reinforcement mats, and gabion mattresses were used to stabilize roadway embankments, creek banks, and bends.

Figure CS2–1 Original timber-plank bridge structure

Design methodology

Drainage study

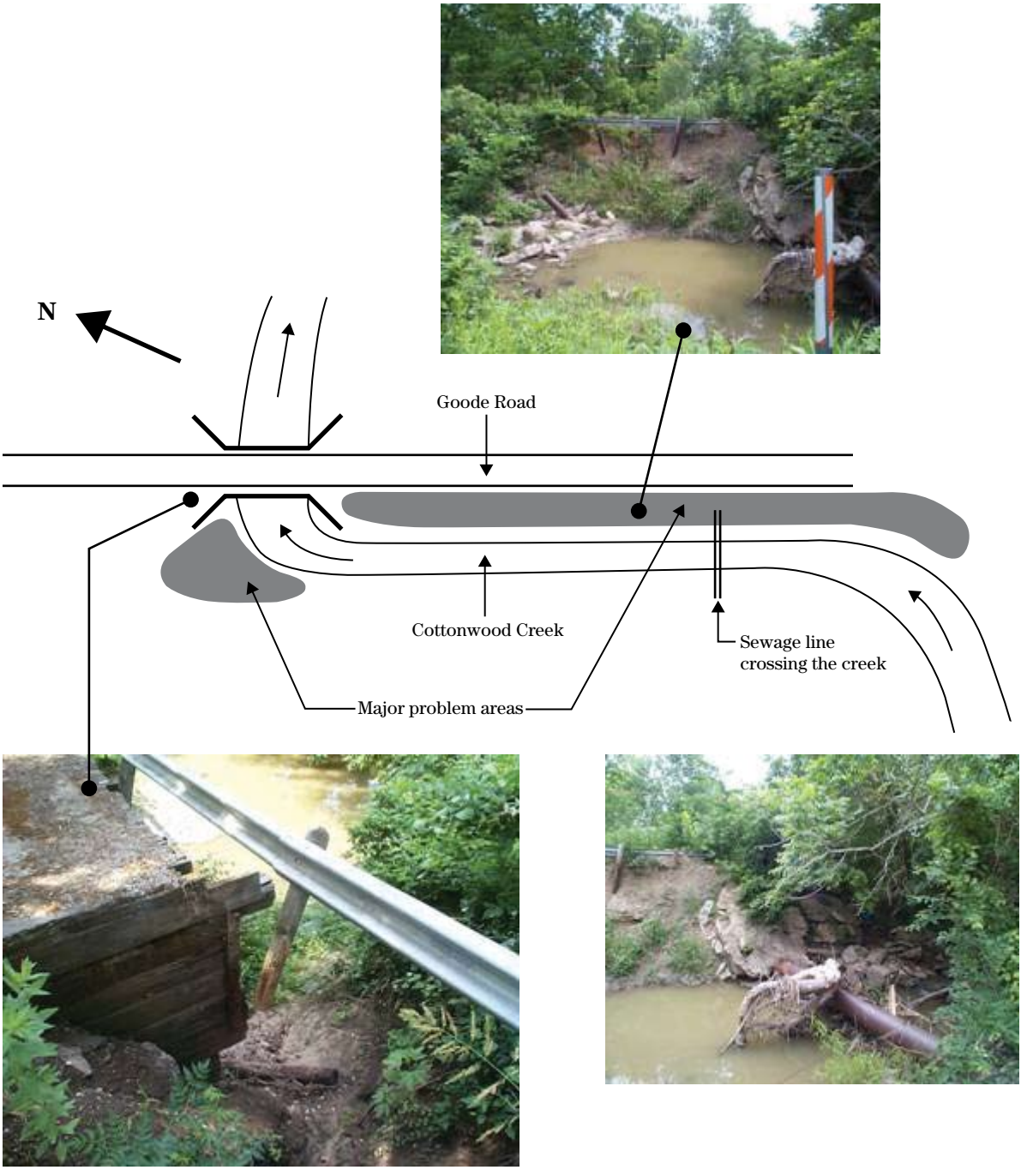
At the early design stage, TxDOT conducted a drainage study to determine the new bridge elevation. The proposed bridge was designed according to the following criteria:

- cost-effectiveness
- passing the 50-year flood with a headwater elevation that is less than the water surface elevation encountered with the existing structure
- passing the 100-year flood at a water surface elevation that is no greater than that calculated using the existing conditions, thus theoretically not encroaching on the 100-year flood plain

Cottonwood Creek at Goode Road flows in an easterly direction and drains east to the Trinity River. The drainage basin affecting Cottonwood Creek is approximately 13.1 square kilometers. The Goode Road bridge drainage basin located in a suburban section of southeast Dallas currently has single-family parcel areas and some parcels with one to several hectares.

The U.S. Army Corps of Engineers (USACE) HEC–RAS computer program (USACE 1995a) was used to calculate water surface profiles of the proposed design condition. Results of the proposed design were compared

Figure CS2–2 Goode Road was closed due to scouring of the roadway embankment and abutment areas.



with the modeling results of the original bridge design conducted by the Federal Emergency Management Agency (FEMA). It was concluded that the proposed single-span bridge structure would decrease the water surface elevation from that of existing conditions. The bridge design and bridge deck elevation were determined on this basis.

Shear stress

The project team used the water surface profile and creek channel slope data to calculate the mean shear stresses on the creekbank and bottom during flood events. The 100-year flood was used to determine the flow depth at approximately 5.3 meters. The creek channel slope was 0.3 percent. The shear stress on the creek bottom approximately equals (water specific weight)× (depth of water)×(channel slope) = 9.81 kN/m^3×5.3 m×0.003 = 156 Pa. This shear stress was considered in selecting streambank soil bioengineering techniques for eroded areas.

Soil data

Soil borings were conducted on the center of the existing roadway to investigate the soil type and profile. Clayey and sandy clay soils were under the road base and were considered *in situ* soil. Limestone was found 6 to 7 meters below the road surface and was also the creekbottom material.

Soil bioengineering measures

Gabion mattress

Streambank soil bioengineering methods are not applicable where there is little or no sunlight. Gabion was chosen to complement this weakness of soil bioengineering methods. Gabion mattress composed of wire cages and rocks was used to stabilize the bridge abutment (fig. CS2–3). The flexibility of installation on irregular areas, as well as shady spots, makes gabion mattress practical to this project.

Vegetated geogrids

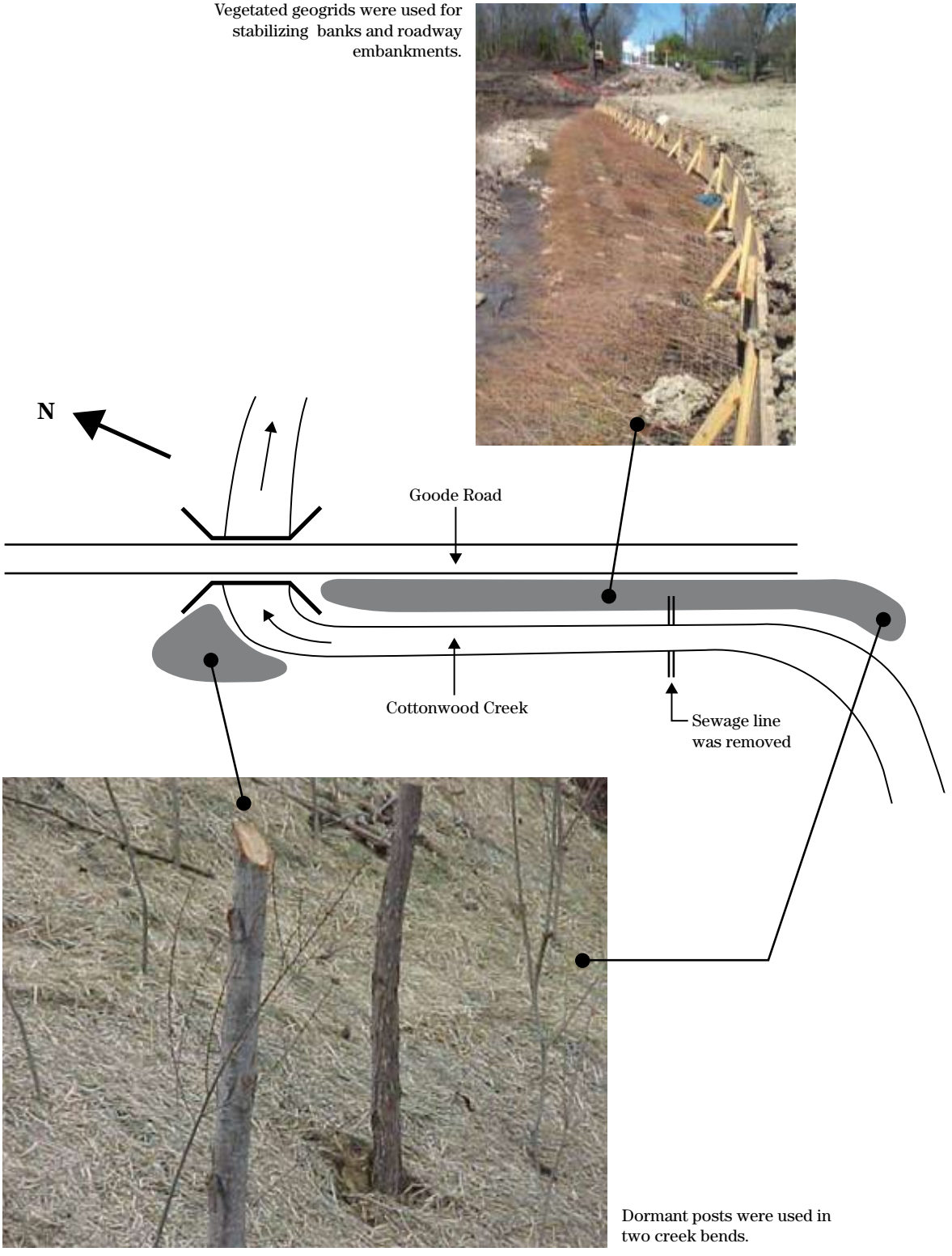
Vegetated geogrids (U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) 1996b) were the primary stabilization measure for the west-facing roadway embankment, which is also the creek bank (fig. CS2–4).

This technique was chosen because the shear stress level that can be sustained by vegetated geogrids is intermediate from 145 to 290 pascal (Li and Eddleman 2002), which brackets the calculated 156 pascal shear stress from a 100-year flood. In addition, the sandy clay onsite can be protected from erosion by geotextiles used in this technique. Rocks of 0.15 to 0.3 meters, wrapped by geogrids, were used at the bottom layer as the foundation and to protect the toe area. This stabilization method provides immediate streambank protection at the early stage after installation.

Black willow (*Salix nigra*) was used in this project because it is native and widely available in public rights-of-way. Cutting size ranged from 25 to 38 millimeters in diameter and from 1.5 to 2.7 meters in length. Approximately 33 to 40 cuttings were used per linear meter of streambank, and 3.65 meters of geogrid reinforcement were installed into the streambank for each layer. Roadway embankments and streambanks of 6 to 7 meters in height were built with this vegetative geogrid technique. No supplemental irrigation was provided after the installation.

Figure CS2–3 Gabion mattress was used for the bridge abutment area.

Figure CS2–4 Vegetated geogrids and dormant post plantings are the two primary streambank soil bioengineering measures.



Dormant post plantings

Dormant posts (USDA NRCS 1996b) of black willow (*Salix nigra*), 76 to 127 millimeters in diameter, were installed on two creek bend areas (fig. CS2–4). Boulders of 0.3 meters or larger were keyed into the toe of the bend areas to protect the streambank. In addition, turf reinforcement mats were installed along with the dormant posts to reduce the surface erosion.

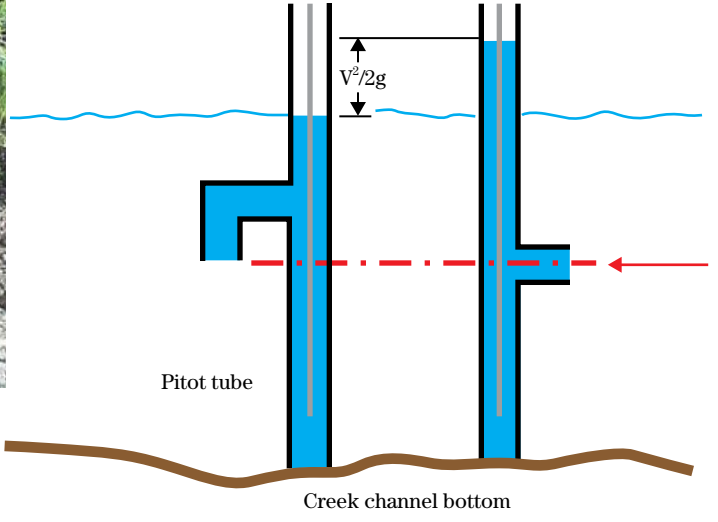
Project results

TxDOT spent approximately $466,400 to replace the bridge, repave the road, and stabilize a streambank of 160 meters. Four layers of vegetated geogrids were installed. The construction activities and timing are listed in table CS2–1. The construction of streambank stabilization was completed in March 2001, followed by the postproject evaluation. Monitored variables include creek flow velocity (point measurement (fig. CS2–5)), flow elevation (point measurement), channel

Table CS2–1 Sequence of construction activities

Activity	Time
Relocation of utility lines (sewer line, drinking water line, and power line)	Jan. 2001
Earth grading	Feb. to May 2001
Harvest of dormant black willow cuttings	Feb. and Mar. 2001
Construction of vegetated geogrids	Mar. 2001
Construction of dormant post plantings	Mar. 2001
Construction of gabion mattresses	May 2001
Placement of bridge beams and decks	June 2001
Construction of road bed and paving	July and Aug. 2001

Figure CS2–5 Water level data loggers inserted in two standpipes to monitor water elevation and velocity



profiles, and survival rate of installed cuttings. Monitored cross sections and the standpipe location for velocity monitoring are presented in figure CS2–6.

Three years after the construction, the monitoring results of this project indicate that the streambank has sustained several high-flow events, the survival rate of willow cuttings are above 90 percent even though no supplemental irrigation was provided, and channel deformation is within an acceptable range (figs. CS2–7, CS2–8, and CS2–9). In summary, this project demonstrates the successful use of soil bioengineering to complement traditional structural streambank stabilization techniques in hot climate areas.

Figure CS2–6 Project plan

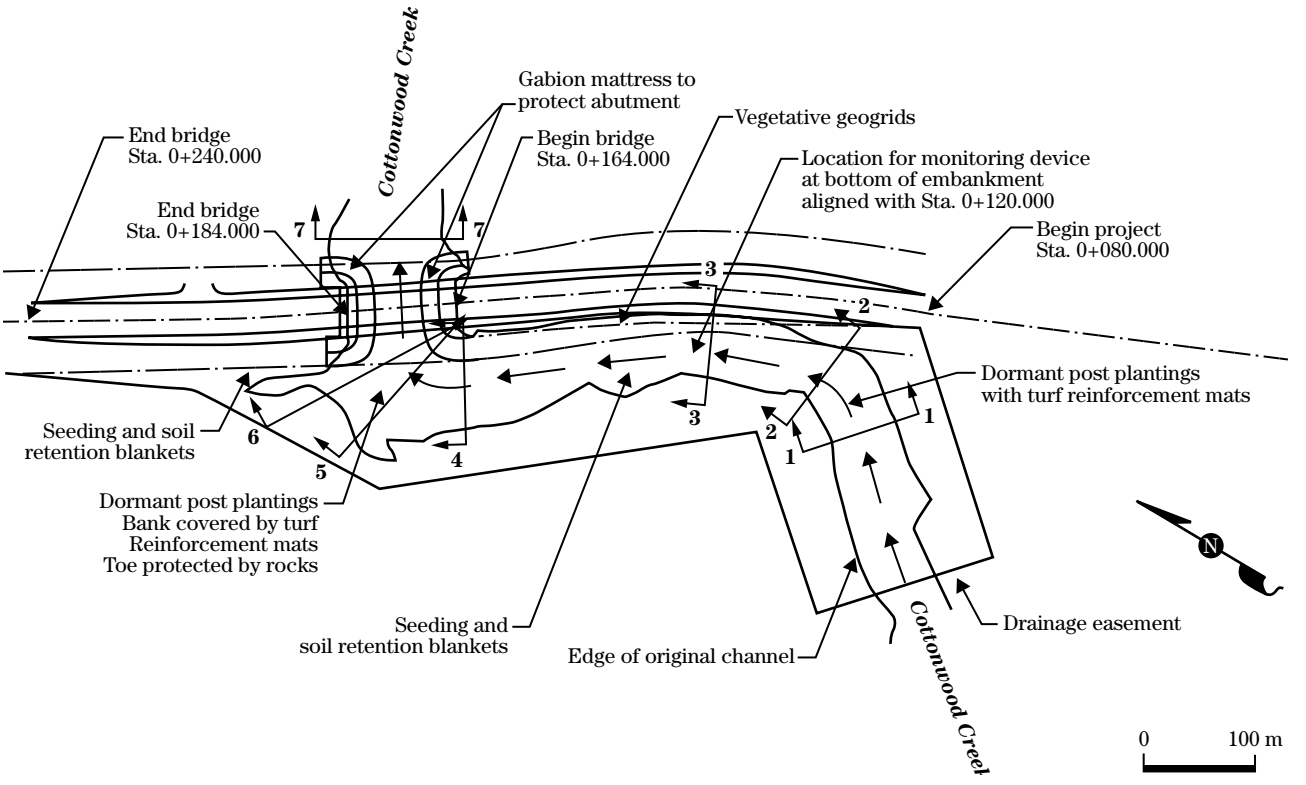


Figure CS2–7 (a) Mar. 2001—Goode Road embankment was stabilized with vegetated geogrids. The opposite bank on the left was protected with soil erosion blankets; (b) Apr. 2003—biomass increases with time; (c) May 2002—black willow covered the entire bank 1 year after construction.

(a)

(b)

(c)

Figure CS2–8 (a) May 2001—dormant black willow posts installed on the first creek bend. Rocks were used to protect the toe area; (b) Nov. 2002—black willow covered the bend 1 year after construction.

(a)

(b)

Figure CS2–9 (a) May 2002—panoramic view of the project; (b) Sept. 2003—postproject conditions (view from the top of the bridge to the creek)

(a)

(b)

Case Study 3

Little Elk River, Price County, Wisconsin

Issued August 2007

Cover photo: Completed section of Little Elk River, Price County, Wisconsin

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 3 Little Elk River, Price County, Wisconsin

By Mary K. King, Area Engineer, U.S. Department of Agriculture, Natural Resources Conservation Service, Altoona, Wisconsin. Photographs courtesy of Price County Land Conservation Department

Abstract

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS); Wisconsin Department of Natural Resources (WDNR); Wisconsin Department of Agriculture, Trade, and Consumer Protection (DATCP); U.S. Fish and Wildlife Service (USFWS); and the Price County Land Conservation Department (LCD) cooperated to improve a 1,000-foot section of trout stream that was degraded from a century of cattle access and an old rock dam crossing installed by the landowner. Rosgen's Stream Classification System (Rosgen 1996) was applied to the degraded section, as well as to an unimpacted section of river immediately upstream of the project site. Design was based on recommendations from the WDNR fish manager and recommended applications from Rosgen's book. The Rosgen approach to geomorphic channel design is provided in NEH 654.11. The design narrowed the degraded section of stream to imitate the class B4c unimpacted section of stream. Improvement to trout habitat was planned with random boulder placement in the riffles of the stream. Cross sections of the stream before construction, after construction, and the following spring were compared. The final cross sections show that the stream width of the degraded section was restored to class B4c to imitate the healthy section of stream.

Introduction

In 2002, 1,400 feet of the Little Elk River was restored. This portion of the river has a drainage area of 26 square miles. The watershed is predominantly wooded with some cropland. The stream bottom is rocky. The soils are mapped Stambaugh silt loam, a glacial outwash material. Most of the Little Elk River is a Class 3 trout stream; however, there is a 2.6-mile stretch of Class 1 trout stream located 0.8 miles downstream of the site (figs. CS3–1 and CS3–2 (WDNR 2002)).

Degradation of trout habitat is usually a result of human activities. Trout habitat is lost to activities that change water temperature or oxygen levels, reduce access to spawning areas, or eliminate trout hiding places. In northern Wisconsin, trout habitat has been degraded by logging, construction of dams (both manmade and beaver-built), draining of wetlands, uncontrolled cattle grazing, soil erosion, and loss of stream corridor vegetation. A section of the Little Elk

Figure CS3–1 Site of Little Elk River

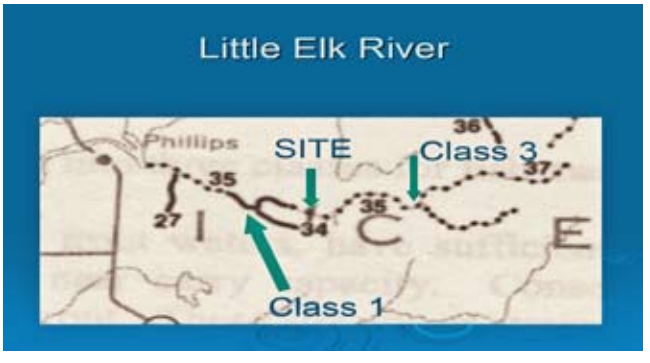
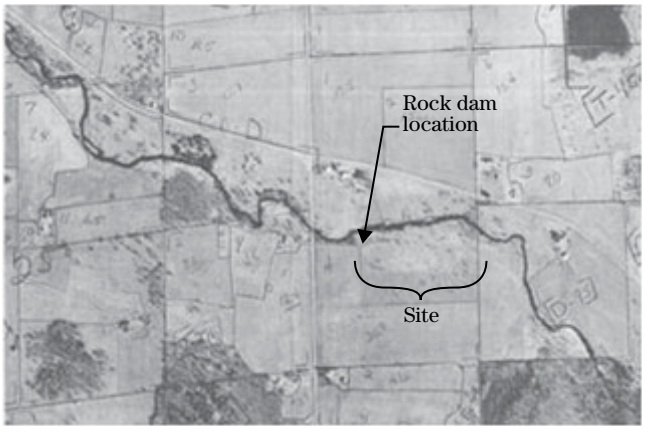


Figure CS3–2 1963 aerial photograph of Little Elk River



River in Price County, Wisconsin, was degraded by a rock dam crossing installed in the early 1990s, and uncontrolled cattle access led to loss of vegetation and widening of the stream (figs. CS3–3 and CS3–4). The crossing was located at the downstream end of the restoration project. Price County is located in the Wisconsin North Woods and supports 71 trout streams totaling 244 miles. Sixty miles are Class 1, 114 miles are Class 2, and 70 miles are Class 3. WDNR classifies trout streams as Class 1 if the high quality trout waters support natural reproduction to sustain populations of wild trout at or near carry capacity, Class 2 if there is some natural reproduction, and Class 3 when there is marginal trout habitat with no natural reproduction occurring. Classes 2 and 3 require annual stocking of trout.

Uncontrolled cattle grazing has eliminated woody vegetation along 700 feet of the 1,400-foot-long site and damaged the remainder (fig. CS3–4). Continuous hoof traffic has broken down the undercut streambanks where trout could hide and eroded the banks, so that the stream width has tripled in places. In the degraded section, the length between pools and riffles, as well as the drop over the riffles, has increased, compared to the more protected section upstream of the property.

The landowner initially contacted the NRCS to sign up for a riparian buffer in the continuous Cropland Reserve Program (CRP). The NRCS buffer program funded fencing along a 160-foot-wide corridor 1,400 feet long and tree planting for 1,050 feet.

NRCS sent the landowner to the Price County LCD for possible stream restoration cost-share assistance. The Wisconsin DATCP, Land and Water Resource Management (LWRM) funds, and the NRCS Access Road Conservation Practice Standard were used to design a rock crossing in place of the rock dam. Price County Shoreland Improvement funds and the USFWS contributed funding to restore 1,000 feet of the Little Elk River for better trout habitat. The LCD worked with the NRCS engineer and the local fish manager of the WDNR to provide a design and use this site as a demonstration project. The landowner contributed the rock riprap he had collected onsite over his years of farming. Assistance for tree planting was provided by local high school students as part of a classroom field trip.

Figure CS3–3 Little Elk River rock dam crossing

Figure CS3–4 Uncontrolled cattle grazing

Design

The design concept was to restore the degraded stream to a similar cross section and stream classification type of a nearby undergraded section. First, in 2001, the rock dam was removed (fig. CS3–5) and replaced with a cattle crossing to meet Wisconsin NRCS Conservation Practice Standard No. 560, Access Road.

The most critical information determined was the bankfull stage. Due to cattle traffic in the disturbed area, a typical bankfull elevation was not pronounced, so staining on rocks and elevations of woody vegetation were used to determine the bankfull elevation. The bankfull channel width was measured in the riffle segment of the selected reach. Cross sections were surveyed through a riffle area in the degraded site (fig. CS3–6) and upstream in an undisturbed area (fig. CS3–7). This information was used to determine the entrenchment ratio and width/depth ratio.

The following spring, the morphological description of each section was compared by applying the Rosgen Stream Classification System. Entrenchment ratio, width/depth ratio, sinuosity, stream slope, and channel material were calculated (Wolman pebble count method) (fig. CS3–8).

Figure CS3–6 Cross section of disturbed site

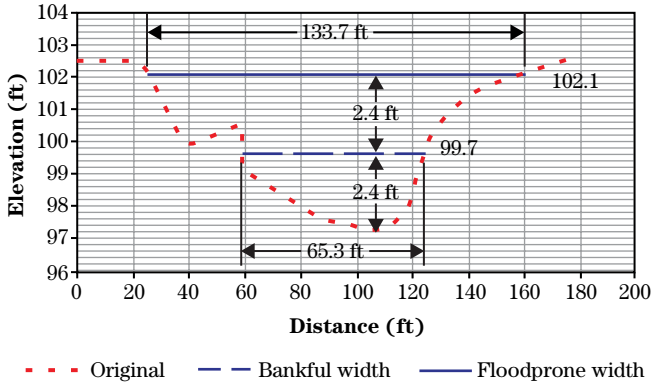


Figure CS3–7 Cross section of undisturbed site upstream

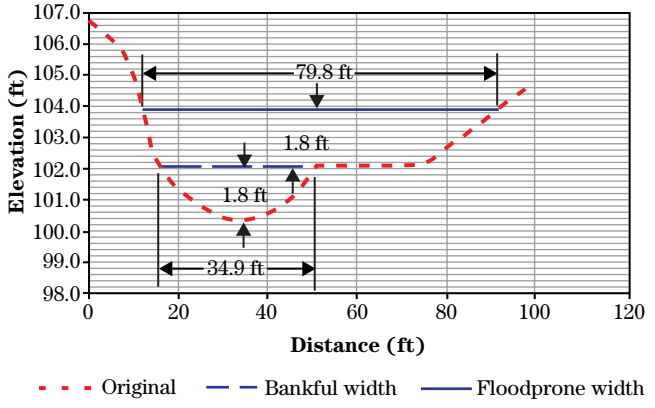


Figure CS3–5 Removal of rock dam crossing, Nov. 2001

Figure CS3–8 Performing the Wolman pebble count

Entrenchment ratio is equal to the floodprone width divided by the bankfull width.

entrenchment ratio$_u$ = 79.8 ft×34.9 ft = 2.29

entrenchment ratio$_d$ = 133.7 ft×65.3 ft = 2.0

Both are moderately entrenched.

Width/depth ratio is equal to the bankfull surface width divided by the mean depth of the bankfull channel.

width/depth ratio$_u$ = 34.9 ft×0.9 ft = 38.8

width/depth ratio$_d$ = 65.3 ft×1.2 ft = 54.4

Width/depth ratio is changing from a moderate toward high ratio.

Sinuosity was measured from an aerial photograph (fig. CS3–2).

sinuosity = 8,250 ft×6,435 ft = 1.28

moderate sinuosity

Moderate entrenchment, width/depth ratio, and sinuosity are characteristics of stream type B.

Water surface elevation was surveyed at two stations 660 feet apart. Both points were in a riffle.

slope = 2.24 ft/660 ft = 0.0034 ft/ft

<0.02 ft/ft, designates "c" in the classification

Median particle size, D_{50}, from the pebble count was 20 millimeters, which is a coarse gravel and designates a "4" in the classification (fig. CS3–9).

The Little Elk River classified as a B4c stream according to the Rosgen method. The width/depth ratio was changing from a moderate to high ratio in the degraded section.

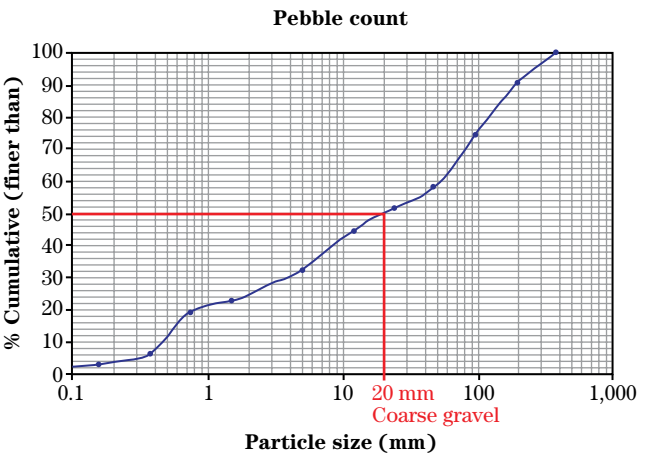
Applied River Morphology (Rosgen 1996) recommends suitable fish habitat improvement structures by stream classification. Channel type B4 is suitable for most structures. NRCS consulted the fish manager from WDNR to concur on the best structures for this site. Random boulder placement was chosen, as well as narrowing the stream to match the upstream undisturbed width.

The stream was narrowed to a 35- to 40-foot width at bankfull elevation, using rockfill. Rock piles were on site from many years of rock picking in the adjacent crop fields. The rock gradation was determined using the following criteria:

Percent passing by weight	Size in inches
100	$2\times D_{50}$
60–85	$1.5\times D_{50}$
25–50	D_{50}
5–20	$0.5\times D_{50}$
0–5	$0.2\times D_{50}$

The gradation of the rock piles measured as D_{50} = 5-inch diameter. This greatly exceeded the required rock size of 2-inch D_{50} based on the computed bankfull velocity of the stream using Manning's equation and the 3.5-inch D_{50} rock size for a 10-year event. The velocity of the stream was computed to be 3.4 feet per second at the bankfull elevation and 5.1 feet per second in a 10-year, 24-hour storm event. Earthfill was placed over the top of the rockfill, seeded, and covered with erosion control mat. Due to budget constraints, only one strip of erosion control mat was used along both banks.

Figure CS3–9 Channel material D_{50}



The construction plan included a note for random boulder placement:

> ***Place approximately 30 boulders, in groups or singly, in a random fashion between stations 1+00 and 11+40. Boulder size shall be of 2-foot diameter or larger. Boulders shall be placed in riffles for added fish habitat cover. The placed boulder shall not divert water flow into the bank. The technician must be onsite during placement.***

Boulders were placed primarily in riffles to provide resting places for trout. Boulders were arranged in the field to direct flow away from the banks (figs. CS3–10 and CS3–11).

Construction		
Rock	1,465 yd^3×\$7.50/yd^3	= \$10,987.50
Earthfill	350 yd^3×\$5/yd^3	= \$ 1,750.00
Boulders	30 ea×\$20 ea	= \$ 600.00
Erosion control mat	10 rolls×\$76.70/roll	= \$ 767.00
Seed	Job	= \$ 110.00
Rock dam removal	Job	= \$ 875.00
Total cost		= \$15,089.50

Equipment used: JD–450, excavator, front end loader included in the cost of material.

Time to do the project: 3 days (about 26 hours).

Figure CS3–10 Boulders used for random placement in the stream

Figure CS3–11 Random boulder placement with DNR fish manager and NRCS engineer

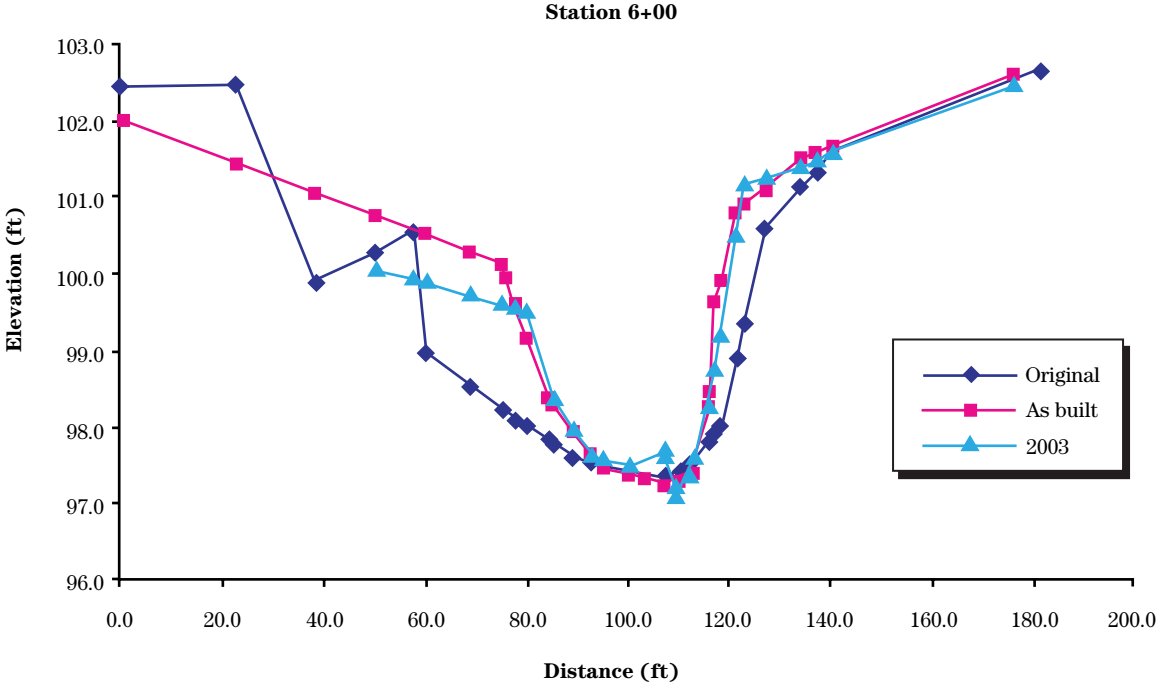
Figure CS3–12 shows the project 1 month after construction; figure CS3–13 shows a cross-sectional comparison.

Figure CS3–12 1 month after construction

Conclusions

The rocky stream and soil conditions of this project supported narrowing the stream width with rockfill. The material was in existing stockpiles adjacent to the site, which helped make the project very cost effective. Working together with multiple agencies and disciplines early in the planning facilitated the permitting process and assured that both biological and engineering needs were met. The section of restored stream was tested with flooding just 2 days after installation, and the restoration features remained intact. A survey the following spring showed no significant change in the cross section geometry. The trout stream classification has not yet been reevaluated by the WDNR, but it is anticipated that the current Class 3 rating will improve to Class 2 or even Class 1 as found just 0.8 miles downstream.

Figure CS3–13 Cross-sectional comparison



Case Study 4

Silver Creek, Silver Creek, New York

Issued August 2007

Cover photo: Completed section of Silver Creek, Silver Creek, New York

Advisory Note

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Case Study 4

Reinforced Soil Wall Silver Creek, New York

By Frank T. Cousin, Jr., soil bioengineer, P.E.; and Fred Gasper, civil engineer, P.E.; U.S. Department of Agriculture, Natural Resources Conservation Service, East Lansing, Michigan

Introduction

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) designed a 40.2-meter-long (132 ft), reinforced soil wall on Silver Creek in Silver Creek, New York, as part of an Emergency Watershed Protection (EWP) project. The geotextile-reinforced soil wall stabilized an eroding streambank that threatened a local road and exposed an old landfill. The rock-faced, reinforced soil wall is 3.4 meters high (11 ft). The cost of the reinforced soil wall saved about $50,000 over designs that included concrete retaining walls and other conventional systems. The reinforced soil wall also saved design and construction time over concrete retaining walls. Design of rock walls, such as presented in this case study, is provided in NEH654 TS14M.

Background

In January of 1996, southwestern New York State received a combination of rainfall and snowmelt that caused flooding and severe erosion. New York received federal assistance under the EWP program. The NRCS had the charge of planning and designing the repairs. The planning and design phase of EWP sites must be performed in a timely manner to prevent further damage from occurring on these already damaged sites. The Silver Creek EWP site was one of these sites.

The Silver Creek EWP site is located in Silver Creek, New York. The town of Silver Creek is located in the western part of the state on Lake Erie, approximately 40 kilometers (25 miles) southwest of Buffalo. The site is located on the west bank of Silver Creek, on an outside bend, immediately south of Highway 20.

High flows in Silver Creek ranged between 4.6 meters per second (15 ft/s) and 6.1 meters per second (20 ft/s), causing excessive toe erosion above the shale bedrock underlying the streambank. At the project site, bedrock is located near the bankfull or channel forming discharge elevation. The streambank at the site is located in a former municipal landfill. The eroded slope was 40.2 meters long (132 ft), with a height from the toe to the top of bank ranging from 4.6 meters (15 ft) to 5.8 meters (19 ft). Erosion of the streambank toe and subsequent sloughing of the banks was causing excessive amounts of sediment to be deposited in Hanford Bay.

In addition to the sediment being deposited into the bay, a local road at the top of the bank was in danger of being lost due to erosion. The local road, Spencer Place, is the only access to the Petri Baking Products, a major employer in the area.

Engineering alternatives

After the problems were identified and field data collected, alternative solutions were considered.

The first alternative was to build a compacted earthfill slope and provide protection at the streambank toe with rock riprap. It was estimated that a 2H:1V (horizontal to vertical) slope configuration would be required for a stable slope. This configuration would not fit the physical constraints at the site without encroaching on flows within Silver Creek. A reach of Silver Creek would need to be relocated for this alternative design. This alternative was not viable.

The second alternative was to excavate the slope back on a 2H:1V slope and place rock riprap at the toe. This alternative would have required relocating Spencer Place and several homes. This alternative was not desirable.

A third alternative that was investigated was to build a nearly vertical wall at the base of the slope and build a 2H:1V or 3H:1V slope above the wall up to the top of the slope. Several types of walls were considered: gabion, bin wall, sheet piling wall, and concrete retaining wall.

The sheet pile wall (fig. CS4–1) was not considered a viable alternative because of the shale bedrock foundation.

The gabion wall (fig. CS4–2) was not selected because of abrasion expected from bed-load and suspended load.

Both the bin wall (fig. CS4–3) and the concrete retaining wall (fig. CS4–4) were considered as viable alternatives. The cost estimate for a bin wall or concrete retaining wall was $90,000.

A fourth alternative, a reinforced soil wall or slope, was investigated (fig. CS4–5). The estimated cost for a reinforced soil wall or slope was $45,000. After researching this technique, it was chosen as the best alternative for stabilizing the site.

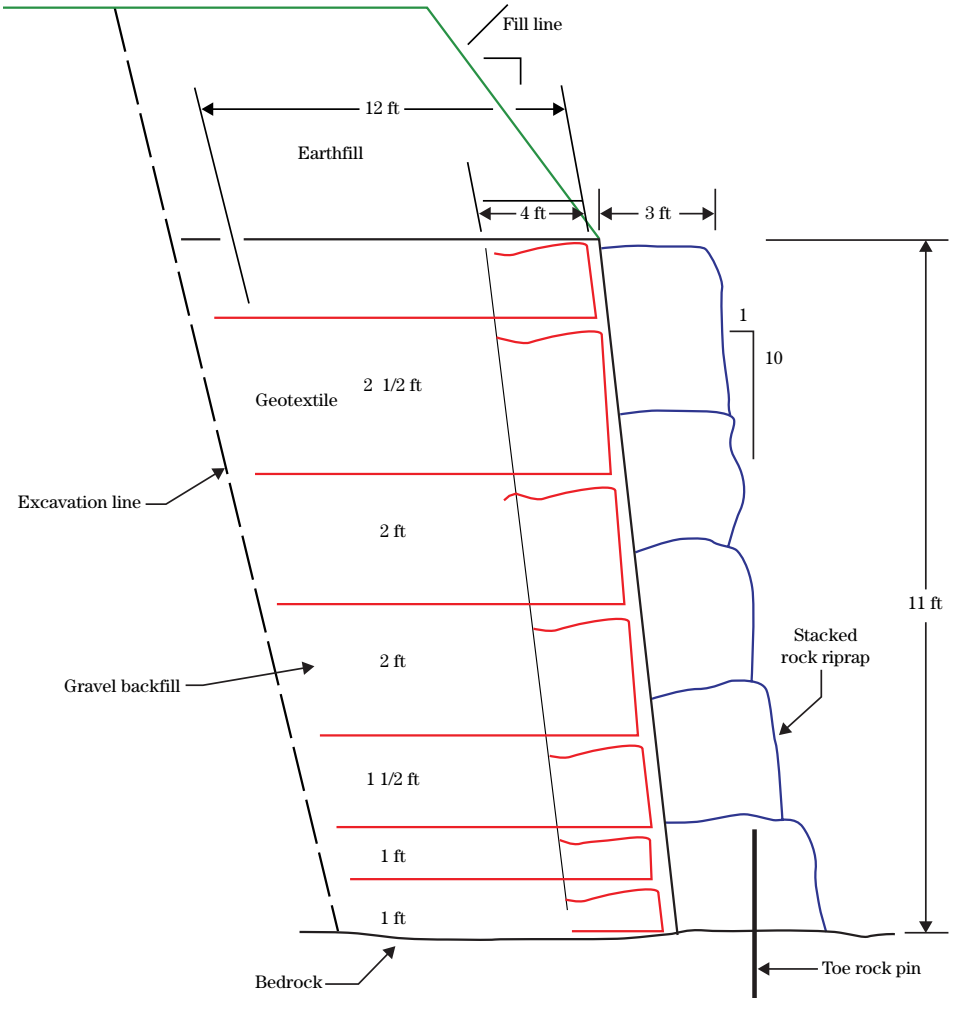
Figure CS4–1 Sheet pile wall

Figure CS4–2 Gabion wall

Figure CS4–3 Bin wall

Figure CS4–4 Concrete wall alternative

Figure CS4–5 Reinforced soil wall detail, Silver Creek, NY



Reinforced soil walls

Reinforced soil walls use elements to provide tensile reinforcement and increase shear strength of the soil backfill. Reinforced soil systems have been used successfully in the United States for more than 25 years. The USDA Forest Service has been building geotextile reinforced walls since 1974. Reinforced walls have many advantages over conventional reinforced concrete retaining walls (Mitchell and Villet 1987), in that they:

- are coherent and flexible and thus can tolerate relatively large settlements
- are easy to construct
- are relatively resistant to seismic loadings
- can form aesthetically attractive retaining walls and slopes because of a variety of available facing types
- are very often less costly than conventional retaining structures, especially for high steep slopes and high walls
- can use a wide range of backfill material

Before cost comparisons could be made, different types of tensile reinforcement had to be considered. The tensile reinforcement types that were considered were polymeric geogrids and geotextiles. After the analysis of the performance criteria for different geogrids and geotextiles, the decision was to use a geotextile for tensile reinforcement. Site conditions that made geotextiles the material of choice were: low height, 4.6 meters (15 ft) to 5.8 meters (19 ft); the ability to use a steep slope face (nonvertical), instead of a wall face (vertical); type of facing material; and cost.

Because of the layout of the site, a steep reinforced slope was able to be used instead of a vertical wall. There was enough area behind the facing to provide both internal and external stability. Also, there was enough area to provide a steep slope without encroaching on Silver Creek. This significantly reduced the amount of tensile reinforcement required to stabilize the streambank. Because of the steep slope, 1H:10V, the reinforced soil slope is called a reinforced soil wall for this application.

The facing of a reinforced soil wall is used to prevent the wall face from unraveling. The facing provides protection of the geotextile from degradation due to ultraviolet rays and provides an abrasion resistant surface, protecting the geotextile from streamflow and vandalism. The cost of reinforced soil walls is heavily dependent on the cost of the facing. The type of facing that was selected for the Silver Creek site was a rectangular-shaped massive limestone. The stones varied in size, but averaged 61 centimeters high (24 in) by 61 centimeters wide (24 in) by 76 centimeters long (30 in).

Design

AMOSPEC™ design software, by Amoco Fabrics and Fibers Company, was used to design the reinforced soil wall on Silver Creek. AMOSPEC™ evaluates reinforced soil wall and slope stability design using limit equilibrium analysis. The output derived from the computer program specifically addresses design and construction of geotextile wrapped-face walls.

AMOSPEC™ assumes that additional strength provided by the facing is not considered in the design, that the backfill material is free draining and does not consider deep slope stability, including shear failure surfaces through the foundation.

AMOSPEC™ evaluates the following modes of failure:

- sliding along the base of the reinforced wall
- bearing capacity at the toe of the reinforced wall
- internal stability of the reinforced wall

AMOSPEC™ selects a minimum geotextile embedment length and a minimum geotextile strength or spacing.

The input variables are:

- reinforced wall height
- reinforced wall inclination
- backslope angle of earthfill
- soil properties

- soil/geotextile interface strength
- geotextile strength or spacing
- uniform surcharge loading
- stability factors of safety

Input variables

The height of the slope varies from 4.6 meters (15 ft) to 5.8 meters (19 ft). The height of the reinforced soil wall is 3.4 meters (11 ft) (fig. CS4–5).

Several trials were run to achieve the optimum combination of compacted earthfill slope and reinforced soil wall. The backslope angle of the compacted earthfill above the wall is 3H:1V. The inclination of the wall face is 1H:10V.

The gravel backfill material was required to meet the gradation shown in table CS4–1.

No laboratory tests were performed. Soil properties were estimated based on gradation and correlations to similar soils. The estimate of the angle of internal friction was estimated to be 36 degrees. The assumptions that were made to estimate the angle of internal friction were: a poorly graded to well-graded gravel, GP or GW based on the Unified Classification System, and soils were compacted to 70 percent relative density. Based on these assumptions, the angle of internal friction was estimated to be between 37 and 40 degrees. Without laboratory tests, a conservative value of 36 degrees was chosen. A moist unit weight of 2.08 grams per cubic centimeter (g/cm^3) (130 lb/ft^3) was used.

The wall is located on a shale bedrock foundation. A unit weight of 2.24 grams per cubic centimeter (140 lb/ft^3) and an angle of internal friction of 40 degrees were chosen for the foundation. These values are very conservative. Deep seated slope stability, including failure surfaces through the foundation, were not a concern in the bedrock foundation.

A uniform surcharge load of 4.8 kilopascal (kPa) (100 lb/ft^2) was chosen to represent the load from semi-trucks that use Spencer Place.

The following geotextile properties were used:

- Ultimate wide-width tensile strength was 70 kilonewtons per meter [kN/m] (4,800 lb/ft).
- Design tensile strength was 11.7 kilonewtons per meter (800 lb/ft).
- Geotextile-soil interface friction angle was 20 degrees.

The following were chosen as factors of safety, FS:

- FS against block sliding was 1.5.
- FS against bearing capacity failure was 2.
- FS for geotextile strength was 6.
- FS for geotextile spacing was 1.4.

The spacing of the geotextile ranged from 0.31 meters (1 ft) to 0.76 meters (2.5 ft), with an embedment length equal to the height of the wall (fig. CS4–5). No secondary reinforcement was required because the backfill soil was compacted against the rock riprap facing. This resulted in no sagging of the geotextile-wrapped face.

Table CS4–1 Backfill particle size requirements

U.S. standard sieve size	Sieve opening size (mm)	Percent finer by weight
3 in	75.0	100
No. 4	0.425	50–0
No. 200	0.075	5–0

Cost

The cost of the 3.4-meter-high (11 ft), 40.2-meter-long (132 ft), reinforced soil wall, including earth work above the wall and concrete associated with an existing structure, was $39,334. Of this total cost, approximately $34,300 was for the reinforced soil wall. The reinforced soil wall cost approximately $250 per

square meter ($23/ft^2). The reinforced soil wall saved an estimated $50,000 in construction costs over concrete retaining walls and other alternatives proposed. The cost breakdown is listed in the table CS4–2.

The U.S. Department of Transportation Federal Highway Administration publication, Reinforced Soil Structures Volume I, Design and Construction Guidelines, states that for segmental concrete-faced structures, the typical costs are:

- Reinforcing materials equal 10 percent to 20 percent of the cost.
- Backfill materials, including placement, equal 30 percent to 40 percent of the cost.
- The facing system equals 40 percent to 50 percent of the cost.

The cost of the backfill was on the low side, and the facing was on the high side of ranges given above. This can be attributed to the bid schedule. The backfill was a lump sum item in the bid schedule, while the riprap facing was bid by the ton.

Table CS4–2 Costs of installation

Work or material	Quantity	Cost	Percent of total cost
Mobilization	Lump sum	4,000	10
Compacted gravel backfill	Lump sum	4,000	10
Facing, stacked riprap, in-place	394,980 kg (435 tons)	21,760	55
Geotextile, in-place	1,272 m^2 (1,522 y^2)	4,566	12
Miscellaneous		5,008	13
Total costs		39,334	100

Construction

The project was completed on schedule with no problems (figs. CS4–6 through CS4–13), even though the contractor had no previous experience with reinforced soil stabilization.

The backslope of the proposed reinforced soil wall was prepared by excavating a smooth surface for good soil contact between the *in situ* material and the compacted gravel backfill material.

Of concern was the potential for erosion of the first course of rock being at high velocity flows. This concern was addressed by pinning the first course of rock (toe rock) to the underlying shale bedrock. The pinning was accomplished by 1-inch-diameter steel bars driven through the toe rock into the bedrock.

One of the big advantages of using stacked rock riprap as the facing was that the gravel backfill could be compacted against the facing, eliminating the need for slip forms or some other method of compacting the backfill material near the facing. Small pieces of limestone were used to fill in the voids between the larger stacked riprap to protect the wrapped geotextile face.

The geotextile reinforcement and gravel backfill were "brought up" with the stacked riprap facing (figs. CS4–8 and CS4–9). The gravel backfill was compacted with a rubber-tired Case 1285 backhoe, a small bulldozer (Caterpillar® D4 equivalent), and a manually directed vibrating drum roller (figs. CS4–6 and CS4–7). The backfill within 2 feet of the wall facing was compacted with a manually directed power tamper (plate).

The geotextile length was run parallel to the slope to reduce the number of overlaps (fig. CS4–8).

The reinforced soil wall was completed within 15 construction days (figs. CS4–12 and CS4–13). John Burt, Silver Creek Village Manager, reported that 2 weeks after the wall was completed, flows within Silver Creek came within 2 feet of the top of the wall. He stated that, "we would have lost the (Spencer Place) road for sure without the wall."

Figure CS4–6 Compaction equipment

Figure CS4–7 Manually directed vibrating drum roller

Figure CS4–8 Primary reinforcement geotextile

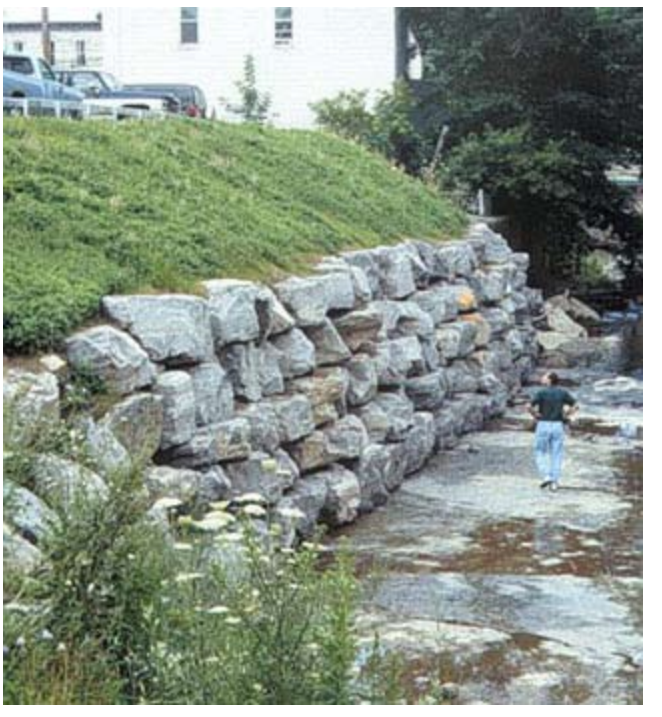
Figure CS4–9 Primary geotextile anchorage length

Figure CS4–10 Stacked riprap face placement

Figure CS4–11 Flow during construction

Figure CS4–12 Completed wall

Figure CS4–13 Completed wall



Observations

The reinforced soil wall has been tested by high flows and is functioning as designed. The limestone riprap provided an aesthetically pleasing face. There is some herbaceous vegetation growing on ledges and openings within the rock face. Over time, sediment trapped within the interstices of the stacked riprap should expedite vegetation growing on the rock face. This will add to the aesthetics of the reinforced soil wall.

Conclusion

The reinforced soil wall was easily adapted to the site. The wall is flexible and can withstand large strains and deformation. The wall is functional, attractive, and inexpensive, when compared to reinforced concrete retaining walls and cellular confinement systems. The limestone riprap provided an aesthetically pleasing face. EWP projects require fast action. Because of the limited time available for design and construction, a reinforced soil wall was an excellent choice for the Silver Creek site. Using the coarse-grained, free-draining backing reduced the design time and also reduced the earth load on the reinforced soil wall. Construction was fast and simple and did not require skilled labor. Cost and aesthetics were very important on this project. The stacked rock riprap facing addressed both of these concerns.

Case Study 5 Rose River, Madison County, Virginia

Issued August 2007

Cover photo: Rock weir on the Rose River in Virginia

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 5 Rose River, Madison County, Virginia

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Abstract

In 1995 and 1996, major storm events caused severe damage to the Rose River in Madison County, Virginia. In 1998, 4,200 feet of the Rose River was restored to a more stable condition and strengthened using a variety of natural and ecologically friendly techniques. This effort included the installation of nine vortex rock weirs and two sets of rootwads. Five years after installation, during Hurricane Isabel, the Rose River flooded again. Although some of the weirs and rootwads were damaged by this flood, the majority of the site remained in a stable condition.

History

During the mid-1990s, central Virginia experienced three major floods. The June 1995 storm was a 500- to 1,000-year frequency event. More than 30 inches of rain fell in 24 hours. Rain on top of 3 feet of snow produced a 25-year frequency event in January of 1996. In September 1996, Hurricane Fran produced 10.5 inches of rain for a greater than 100-year frequency storm. The Rose River, in Madison County near Syria, was severely damaged by these repeated events. In 1995 and again during the winter of 1998, the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) used the Emergency Watershed Protection (EWP) program to restore the Rose River to its preevent hydrologic condition. In September 2003, Hurricane Isabel again flooded the site. Since this was a 5-year event with 4.10 inches of precipitation, there was less damage than in the previous storms.

The project cooperators for the Rose River restoration were the NRCS, Virginia Department of Game and Inland Fisheries (DGIF), Virginia Department of Forestry (DOF), Virginia Department of Conservation and Recreation (DCR), and Graves Mountain Lodge Corporation. The U.S. Army Corps of Engineers (USACE) provided comments. Because this site was classified as a nonexigency after Hurricane Fran (September 1996), there was time to make detailed plans for this site. The work included removal of large sediment deposits along the channel and installation of nine vortex rock weirs, two sets of rootwad revetments, and 400 linear feet of riprap. The river was also rerouted back into its preflood location at the upper end. The overall cost of construction was approximately $120,000, or about $29 per linear foot.

Goals

The primary goal of the 1998 restoration was to restore the hydrologic function of the river. At the time of the initial assessment, the major problem with the upper third of the reach was a large cobble and debris bar that constricted the flood plain at the single lane bridge leading to several private residences. A 6- to 8-foot-high vertical bank existed along one driveway for a house at the very top of the reach. Between the time of the initial NRCS site visit and the time of final contract package preparation, the landowner did some work to protect the driveway.

In the lower two-thirds of the reach, banks were 6 to 8 feet high and vertical in several places. The most serious eroded area was located at the bottom of the reach and was about 10 feet from State Road 670. This lower reach also had large cobble bars that were constricting the flood plain.

Another goal for this restoration project was to address the concerns of the other stakeholders. The Virginia DGIF was very interested in this site because it was stocked with trout. It was also the site for an annual Children's Fish-For-Fun event. This meant that the site needed to be safe. The landowner was concerned with water access for cattle and tractor crossings. He also wanted to protect the driveway that paralleled the river on the upper end of the reach. Comments from the USACE indicated that revegetation of the site was critical.

These goals seemed to be reasonable and attainable. At the completion of the construction, the majority of these goals were met. The landowner used the Conservation Reserve Program (CRP) to exclude cattle from the site and to plant trees in the riparian zone. By this means, the site became well vegetated and stable.

Description of the watershed

The site is located in Madison County, Virginia, in a mountainous section of the Virginia Piedmont region. The watershed is about 14 square miles and mostly wooded with some grassland agriculture along the river. Houses and small businesses are scattered throughout the area. Tourism is important to the economy of this area.

The watershed will continue to be impacted by results of the June 1995 storm for many years to come. This large, slow-moving storm dropped more than 30 inches of water along the east side of the Blue Ridge Mountain range. According to the U.S. Geological Survey (USGS), hundreds of rock slides occurred. This continual supply of cobbles and gravel, therefore, will be available to the river for the foreseeable future.

Description of the reach

This reach of the Rose River is about 4,200 linear feet long and runs parallel to State Route 670 for its entire length. A small bridge crosses the river about a third of the way down from the upper end. A private driveway extends along the river in both directions from the bridge. At the time of construction, the entire reach had only one landowner and was in pasture with unlimited cattle access. The bed was covered in cobbles with some sand accumulation in the pools. Some large boulders were scattered throughout. The water was very clear, even after a storm, and macroinvertebrates have been noted.

Controls and assessment of risk

The only hard control in the reach was at the bridge. The proximity of the river to the state road at the lower end and the driveway at the upper end required installation of riprap at each location. This created two new control points. For the remainder of the channel, there were no constraints on moving the channel laterally as needed to achieve a more stable meander geometry. However, relatively few changes were made to the channel location during construction.

The interagency team was primarily concerned with control of the vertical dimensions of the reach. The fisheries stakeholders were particularly interested in maintaining the big pool in the lower reach. Concern also focused on cobble removal from the river, which would result in instability of the channel grade. This issue was addressed through the use of nine vortex rock weirs.

The stream in its degraded condition posed several risks. If the restoration work was not performed, the state road would have been undercut through continued migration of the river. The private driveway at the upstream end of the bridge was also threatened, but could have been moved away from the river. The cutbanks would continue to erode, adding sediment load, and fish habitat would deteriorate due to filling of the pools or by loss of the pools from bed instability. Flooding would be more significant because of the large cobble and debris bars that constricted the channel.

Repairing the reach also had risks. One risk that was beyond control was the possibility that there would be another large flood event. Landowners tend to expect stream restoration projects to function indefinitely. However, natural streams should be expected to move and evolve in response to large storms. In addition, very few stream structures or improvements have withstood the discharge of water associated with a hurricane or similar large storm. However, restoration of the hydrologic function of the river was considered worth the possibility of future damages.

The use of rootwads for bank protection was less of a risk, given the cost of the treatment. Under rules of the EWP program, NRCS cannot protect agricultural land that is of less value than the cost of the protection. Rootwads are commonly available in the flood plain after a flood event and can be used to provide streambank protection for only the cost of installation. Few suitable rootwads were available on the Rose River; however, but an adjacent EWP contract had many rootwads and no place to put them. An arrangement was made for the disposal of rootwads and other

woody debris on the Rose River site. The rootwads were used, and the other material was burned and buried.

Vortex rock weirs posed more risk. The methodology was still new in Virginia, and previous installations had mixed results. As part of the agreement with the partnering agencies, NRCS installed nine weirs. Some of the rocks were provided by the DGIF, and some came from rock slides located on the property.

Dominant processes

The major process that has repeatedly affected the Rose River is the heavy flooding caused by large storms and hurricanes. Since the time of construction, several floods have occurred that were 1 to 2 feet deep on the flood plain. Few changes occurred in the channel from these small events. Hurricane Isabel, however, caused some obvious changes in channel elevations in at least two locations and damaged some of the rootwads and weirs.

Design criteria/constraints

The funding program requirement that the stream could not be made better than the preevent condition was a significant constraint on the design process. The goal is to restore hydrologic function, therefore, and not specifically to establish new features or create elements that did not exist. However, the features that were there before the flood can be restored. For this site, the design included maintenance of fish habitat and grade control, restoration of the flood plain capacity, and protection of the roads. Although not a primary goal, the overall safety of the site was improved by grading the vertical banks to a more stable slope.

Data collected and analysis performed

Prior to construction, the profile and cross sections of the reach were surveyed. The tops and bottoms of riffles and the centers of pools were located. From these data, the average grade of the reach and the intermediate grades of the riffles and pools were identified. The average channel gradient was less than 2 percent in the lower reach and in most of the upper reach. The upper reach had one riffle with a grade of about 4 percent. The proposed bankfull dimensions were identified from Leopold's chart (Leopold 1994) that correlates drainage area with bankfull dimensions. Since no local regional curves existed at the time, Leopold's curve for the eastern United States was used. These dimensions seemed to fit with the visual appearance of the site. The curves were used instead of onsite information because major flooding tends to remove most of the bankfull indicators.

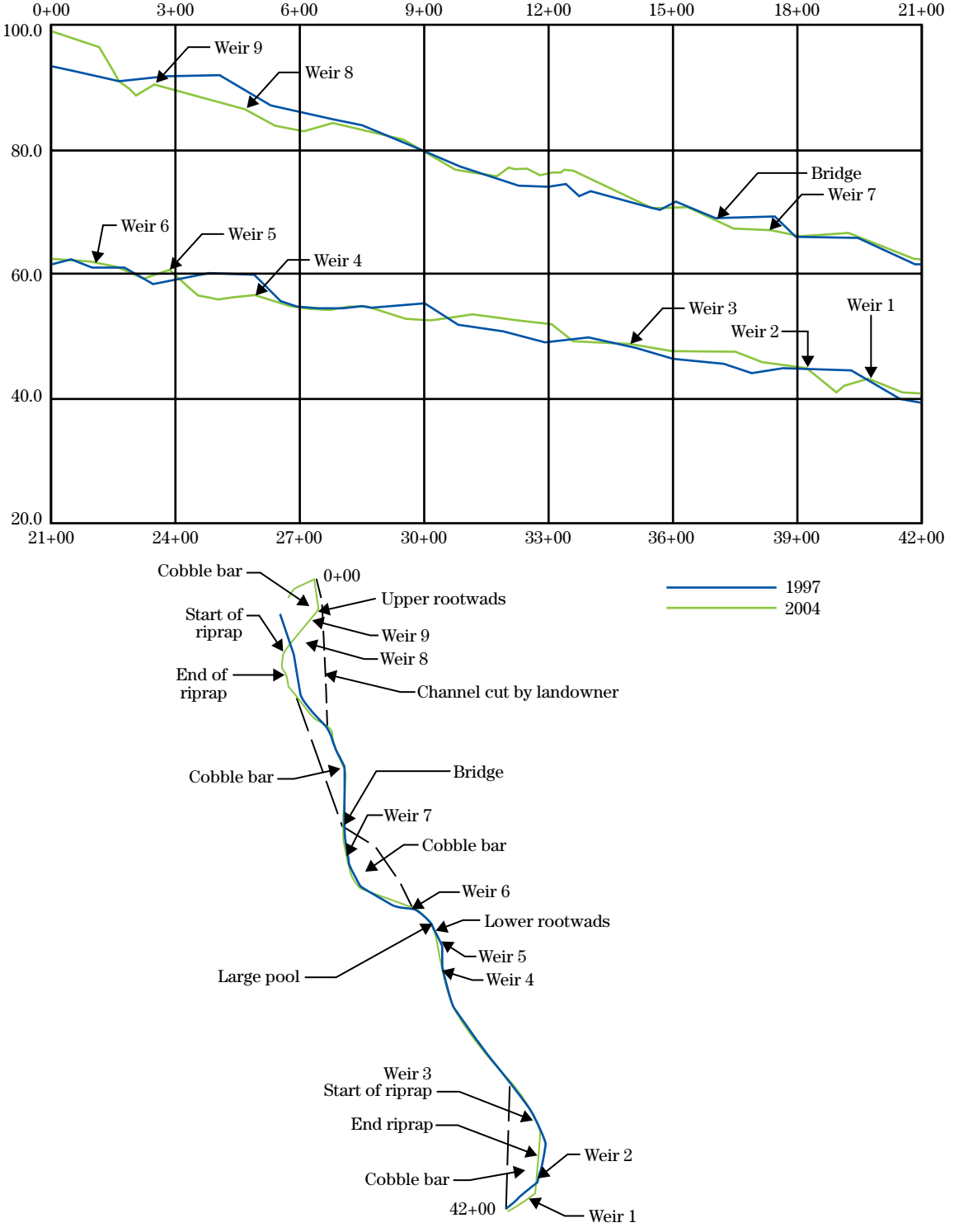
Figure CS5–1 shows complementary plan and profile survey information for Rose River for 1997 and 2004, after the severe storms.

Rosgen's Stream Classification System (Rosgen 1994) was used to describe the proposed restoration. For a bankfull width of 50 feet and an average depth of 3 feet, several geomorphic parameters were identified. The entrenchment ratio was to be greater than 2.2. This dimension could be achieved along most of the river by the removal of large cobble bars. A belt width of 200 feet and a sinuosity of 1.4 were also defined. A meander length of 500 feet and a radius of curvature of 100 to 115 feet were calculated. As a whole, very little of this plan view information was used because the river was left in the postflood location for the most part. At the upper section, where the river had been rerouted, the channel was restored to its original location. In the lower reach, the small braided channels through the cobble bar were pushed into the dominant channel. Achievement of the bankfull cross section and adjacent flood plain elevation was the primary use of the geomorphic information.

Design and installation features

The work started at the lower end of the reach with installation of riprap to protect the state road. To make it easier to key the rock into the channel, the contractor cut a diversion channel through the cobble bar that started above the riprap area and ended below the next riffle. Since the left bank of the river was too close to the road, the channel was moved to the right

Figure CS5–1 Rose River profile and plan view survey information (shown in ft, as surveyed) for 1997 and 2004



about 20 feet. Material from the cobble bar was moved across the river to fill against the vertical left bank. After a 2H:1V slope was established, the contractor placed Class III riprap on the bank and keyed it into the river bottom. About 200 linear feet of bank were armored.

Before the water was redirected into the main channel, two vortex rock weirs were installed at the top and bottom of the riffle immediately downstream of the riprap. Each weir required about 60 tons of Class III rock. The rocks are about one cubic yard in size, resulting in 20 to 30 rocks on the bottom row and 20 to 30 rocks on the top row. The left end of each weir was keyed into natural ground. The right ends were buried in the cobble bar, but had no real anchor point. The bottom row of rocks was completely embedded in the river bottom in a horseshoe shape with the sides higher than the center. The top of the center top rock was also embedded, and it was to extend no more than 3 inches from the bottom of the riverbed. The remainder of the top rocks were nested into the spaces between the bottom rocks and gradually became less embedded as they approached the shore. Because the work was done in the dry channel, it was difficult to accurately determine the correct elevation of the center rock. When the water was returned to the main channel, it was determined that the rocks were higher than planned but still acceptable.

After this work was completed, the water was redirected into the normal channel. The cobble bar was graded toward the right terrace to increase the entrenchment ratio. When the water was back in place, an abrupt change in the channel grade at the upstream end of the riprap was noticed. Another weir was installed "in the wet" to provide some grade control to correct this problem. By putting the track hoe in the water, the contractor was able to more accurately control the placement of the center rocks. Weir #3 was tied into natural ground on the left. As with weirs #1 and #2, the right end of this weir was buried into the cobble bar.

The next section that was of major concern was at the large pool. This pool was about 150 feet long and 3 to 4 feet deep at the outside of the curve. The outside bank was 5 to 6 feet high and undercut. Because of the excellent trout habitat provided by the deep water, the DGIF staff was interested in maintaining the pool at the existing depth and location. To do this, one weir (#6) was placed at the head of the pool and one at the foot, at the top of the riffle (#5) (fig. CS5–2). A third weir (#4) was placed at the bottom of this riffle. The vertical bank was graded to a stable slope, and rootwads were trenched in along the pool (fig. CS5–3). At

Figure CS5–2 Weir #5, March 1998, immediately after construction

Figure CS5–3 Lower rootwads, March 1998, immediately after construction

some time in the past, large slabs of concrete had been placed along the river. These were used as the footers for the first four or five rootwads. Large logs were used as footers for the remaining rootwads. Approximately 60 tons of Class III riprap was placed over the boles of the rootwads at the lower end of the set. These were covered by soil.

A cobble bar constricted the flood plain between the large pool and the bridge. This material was graded toward the first terrace. The vertical bank at the edge of the terrace was graded to a stable slope with the cobble material.

The river was cutting into the outside curve, in the meander between the bridge and the large pool, trying to cut a new channel. Since this new channel was approaching the lower driveway, the water was forced back into the preflood channel, and fill material was placed in the new channel. Several large rocks and a single rootwad were placed at the upper end of the fill material to provide some protection for the cobble material.

Weir #7 was located between the bridge and the curve. The left end was embedded into the cobble bar, and the right end was anchored into natural ground (fig. CS5–4). The scour hole that developed below this weir was several feet deep and formed excellent fish habitat.

In the upper reach, the river had been rerouted to protect the driveway. The initial survey showed that the slope between the stream cutoff and the driveway was about 4 percent in the original channel. Before the water was moved back into the original channel, weirs were installed at the top and bottom of this slope for grade control. Weir #8 was constructed at the bottom of the slope with large rocks that had been taken from one of the nearby rock slides. These rocks were flatter and more rounded than the Class III rocks used in the other weirs, but they were similar in weight. Weir #9 was installed at the top of the slope with 60 tons of Class III riprap.

After the weirs were complete, the water was rerouted back into the original channel. The cut-through channel was blocked with large rootwads and cobble material from the large debris bar above the bridge (fig. CS5–5). The cobble material was trucked up the gully and dumped against the back of the rootwads. This made a very large plug of soil and rocks that would not be vulnerable to washing away if the river overtopped the rootwads. Fill was placed in the gully until the cobble bar material was removed to the bankfull elevation. The gully was then graded and shaped to a stable

Figure CS5–4 Weir #7, July 1998

Figure CS5–5 Upper rootwads after construction, July 1998

slope. Lime, fertilizer, seed, and mulch were applied to the disturbed areas along the entire reach.

Before the final payment was made to the contractor, the site experienced a small flood event. The channel above weir #9 had downcut to the elevation of the weir. Some of the top rocks were moved out of position. The top rocks in weir #8 also rolled out of position and moved downstream. The fill material between the upper rootwads was washed out, and large holes were created. The river also eroded against the bank at the driveway. At this time, the landowner requested that NRCS install riprap along the driveway to protect it. Although it was possible for the driveway to be moved, NRCS decided to install 200 linear feet of Class III riprap along the bank (fig. CS5–6). This was done to protect an old dump that was located under the road. This dump was exposed during Hurricane Fran and observed during the initial site visit. When the additional work was done, 60 tons of Class III riprap were placed in the holes behind the rootwads. The disturbed area was then regraded and replanted. No additional work was done to weirs #8 and #9.

This small flood also affected some of the other weirs in the reach. Most of the top rocks in weirs #1 and #6 were moved out of place. Weir #2 had some cobble accumulation behind it. There were no repairs made to these weirs.

Performance

This site has been monitored visually for the past 7 years. Pictures taken annually showed little change from postconstruction in 1998 to April 2003 (figs. CS5–7 through CS5–11).

The profile was again surveyed in April 2004 (fig. CS5–1). As could be expected, some sections were cut down, and some had filled in. The largest fill occurred at the beginning of the reach. The channel had filled in by about 7 feet in the section above the upper rootwads. It is unclear why this occurred. The profile also showed that weir #9 is providing grade control as planned. Where the river was turned back into its original channel, the profile shows the 2004 slope to be 3 to 4 feet lower than the 1997 profile. Some of this material was removed during construction. The riprap is stable and shows no signs of change. Below the riprap, the elevation of the channel bottom has not changed for about 400 linear feet. A deep pool has developed below the outlet of the gully cut by the landowner. The profile shows that the channel has filled in 2 to 3 feet for a distance of about 350 linear feet below

Figure CS5–6 Upper riprap, July 1998, 3 months after construction

Figure CS5–7 Upper rootwads, April 2003

Figure CS5–8 Upper riprap, April 2003

Figure CS5–9 Weir #7, April 2003

Figure CS5–10 Lower rootwads, April 2003

Figure CS5–11 Weir #5, April 2003

this point. There are few additional changes as the river approaches the bridge.

After Hurricane Isabel (Sept. 2003), the section of the river immediately below the bridge filled in with about 3 feet of cobble material. Weir #7 was completely covered (fig. CS5–12).

Only the rocks on the edges can still be seen. The pool has shifted downstream 50 to 60 feet. The riffle has become longer with no intermediate pools in the curve between weir #7 and weir #6, and weir #6 is unchanged. However, the lower set of rootwads has been almost totally removed. The bank is raw and nearly vertical again (fig. CS5–13). The large pool remains, held in place by weir #5 (fig. CS5–14).

Weirs #5 and #4 appear to be unchanged. The drop in the channel grade from the 1997 survey occurred during construction in 1998. Very little channel work was done from weir #4 to weir #3. The survey shows that the river has experienced some fill and some degradation over about 500 linear feet. Based on the photographic record, the majority of this change occurred since April 2003. It is likely that most of it was done in Hurricane Isabel. Some fill also has occurred between weirs #3 and #2, along the riprap. A large pool has developed between weirs #2 and #1. Below weir #1, there seems to be little change in the channel grade.

When the plan views were compared, they showed that most of the reach had only minor changes from 1997 (fig. CS5–1). The biggest change occurred below weir #8. Part of this probably occurred during construction, when the riprap was installed. However, the river has moved more to the right below the riprap, and the outside bank is vertical and unvegetated (fig.

Figure CS5–13 Lower rootwads, April 2004, after Hurricane Isabel

Figure CS5–12 Weir #7, April 2004, after Hurricane Isabel

Figure CS5–14 Weir #5 and lower rootwads, April 2004, after Hurricane Isabel

CS5–15). A large, fairly steep riffle has formed below this point. The second area where major changes occurred is in the curve between weirs #7 and #6. The curve seems to have become sharper, and some meander development has occurred.

It is important to note that the landowner placed the entire reach into the CRP at the conclusion of construction. Planting grasses and trees and excluding the cattle from this area played a significant role in improving the long-term stability of the river.

What features worked and what did not

By the time this contract started, the use of rootwads was an established technique in flood recovery in Virginia. The contractor used readily accessible material, which was an economical means to protect stream banks. Previous experience showed that rootwads that were trenched into an existing bank and backfilled were more stable than rootwads used to rebuild banks with only cobble material over them. The rootwads on the Rose River site performed completely opposite of this general observation. The upper rootwads were used as part of a gully plug to cut off the manmade channel. The material used behind this was primarily cobble with some purchased riprap. To date, there seems to be little or no deterioration in these rootwads (fig. CS5–16). One possible reason is that the flood plain elevation is at least 4 feet lower on the side across from the rootwads. Previous failures on other sites were because of overtopping. This has yet to occur on this site.

The lower rootwads were trenched into the bank with 10 to 15 feet of the bole embedded. Some of the rootwads at the upper end of this group are still in place. At the lower end, some of the rootwads are missing entirely, while others appear to have been snapped off. One possible explanation is that the flood plain became more entrenched immediately above the rootwads. Starting at the bridge, the right bank was 6 to 8 feet higher than the left bank. The excess cobble had been removed from the left bank to increase the width of the flood plain. Just below weir #6, the left bank intersected the first terrace. This constriction would have increased the flow velocity against the left bank and the rootwads along it. The water elevation may have been above the top of the rootwads, contributing to the problem.

Figure CS5–15 Raw bank downstream of riprap

Figure CS5–16 Upper rootwads, April 2004, after Hurricane Isabel

As a whole, the use of vortex rock weirs made a significant contribution to the stability of this reach. Although not all of them have retained their original shape, the surveyed profile seems to indicate that they are all performing their grade control function. All of the weirs were still visible after Hurricane Isabel, except weir #7. It was almost completely buried in the bed. Prior to that event, only weir #2 had trapped a significant amount of bed-load material. From past experience, the top row of weir rocks need to be spaced one fourth to one-half of their diameter apart to reduce trapping. For this site, most of the top rocks were set about one-half of their diameter apart.

However, a few things could be done to enhance performance. Weirs #1, #2, #8, and #9 were installed "in the dry." Of these, only weir #2 has retained its function of grade control and flow direction. Weirs #3, #4, #5, #6, and #7 were installed without diverting the water. Of these, only the rocks in weirs #5 and #6 have shifted from their installed position. It was much easier to place the weirs at the desired elevation when the water was flowing. It is also easier to put them in the right place on the profile. For example, weir #9 should have been placed further upstream. After the first flood event, the river cut down to meet the weir. By placing the weir higher up, less change would have occurred in the channel in that area. The other main advantage of working "in the wet" is that it is viewed as being less disruptive to the stream ecology. It takes a full day to build a weir, if the water is diverted from the site. It only takes about 2 hours, if the equipment is allowed to work in the stream.

Another change would be to use only angular rocks. The rocks used in weir #8 were native stone and did not interlock well. The top rocks rolled away in the first storm after installation.

Overall lessons learned

The primary lesson learned is that a successful restoration takes planning. However, time is restricted in a flood recovery situation, and possible shortcuts and solutions that can be used need to be identified fairly quickly. Rootwads and weirs are valuable tools for providing bank protection and grade control and are appropriate for many locations. However, these treatments will not last forever. Adequate vegetation and livestock exclusion will often contribute as much to stream stability as the installed structures.

Successful stream restoration requires a vision of the big picture. The majority of the stream restoration work done in Virginia is done under the EWP program. Good interagency cooperation contributes to project success. By considering the needs and issues of interested parties, better design and better results can be achieved.

Case Study 6

Big Bear Creek, Lycoming County, Pennsylvania

Issued August 2007

Cover photo: Cross vane on Big Bear Creek in Pennsylvania

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 6

Big Bear Creek, Lycoming County, Pennsylvania

By Dr. Larry K. Brannaka, U.S. Fish and Wildlife Service (USFWS); David B. Putnam, USFWS; William Worobec, Dunwoody-Big Bear Hunting and Fishing Club, Lycoming County, Pennsylvania; and Donald Stover. (All are members of the Keystone Stream Team.)

Introduction

Big Bear Creek is a mountain stream in Lycoming County, Pennsylvania, that has been classified as a B3 stream using the Rosgen stream classification system (Rosgen 1992). The stream is in a moderately steep valley with sides of relatively gentle slope, matching the Rosgen Valley Type II classification. The location is indicated on a U.S. Geological Survey (USGS) 7.5-minute topographic quadrangle map (figs. CS6–1 and CS6–2). The stream is characterized by moderate slopes and cobble and gravel-bed materials. The riparian lands are mostly wooded. Big Bear Creek is a perennial stream with a significant ground water derived baseflow. Several springs occur along the treatment reach that contribute to the baseflow. The streamflow responds directly to surface runoff from precipitation events. Originally, one dam created a relatively small backwater pond in the project area. The dam has since been removed.

Project description

The restoration project on Bear Creek commenced in the summer of 1999. It was performed as a phased project ending in late summer of 2001. The overall project treated 3.7 miles of stream and included more than 200 instream structures, making it the second largest demonstration project of its kind in the eastern United States at that time. It also was the first project of its kind done by the U.S. Fish and Wildlife Service (USFWS) Pennsylvania Field Office and served as a demonstration project, classroom, and experimental lab.

Over the course of the 3 years it took to complete the project, many lessons were learned, some of which are related in this case study. Bear Creek is classified as a high quality cold-water fishery and has a long history of providing quality trout fishing. The Dunwoody-Big Bear Hunting Club has owned or controlled access to the creek for more than 100 years. The club has detailed records documenting the quality of the fishery, primarily native brook trout, over that time period.

Three bridges that act as constriction points for the flood plain cross the stream. The uppermost bridge, known as the Red Ridge Bridge, was built by the Dunwoody-Big Bear Hunting Club (fig. CS6–3). The

Figure CS6–1 Upper reach of Big Bear Creek restored in phase I

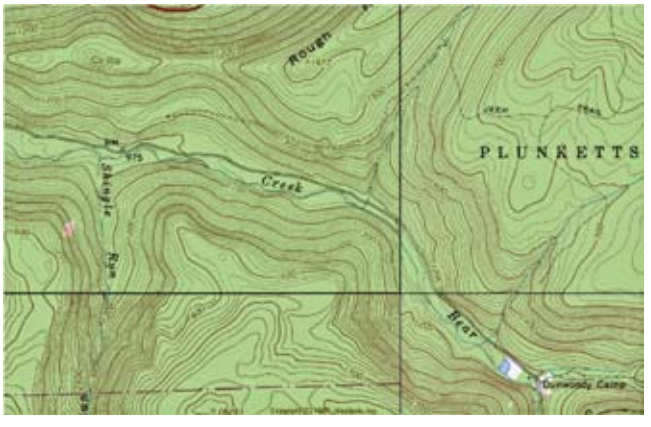
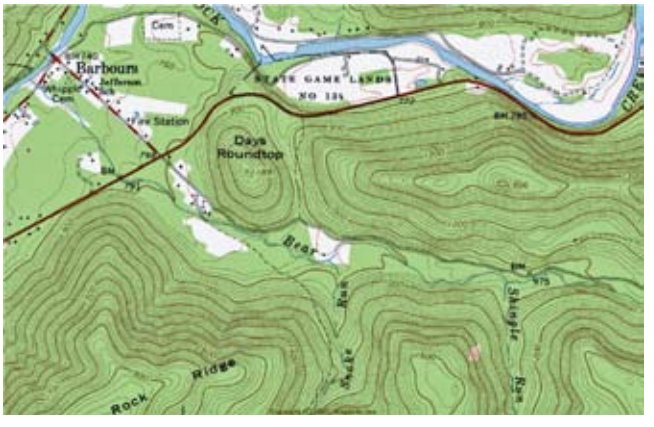


Figure CS6–2 Lower reach of Big Bear Creek restored in phases II and III



lower two bridges were built by the Pennsylvania Department of Transportation (PennDOT). The farthest downstream bridge just above the confluence of Big Bear Creek with Loyalsock Creek (fig. CS6–4), frequently filled with sediment—mostly gravel, cobble, and boulders.

Starting with Hurricane Agnes in 1972, Bear Creek has endured three significant natural flood events and several anthropogenic events. These natural flood events were a direct result of the arrival of Hurricane Agnes and, in 1975, Hurricane Eloise. On January 19, 1996, a 100-year rainfall event on frozen ground with a significant snowpack resulted in a flood event that moved significant amounts of sediment into and down the channel. The floods caused severe erosion and moved vast amounts of sediment into the stream channel. The primary anthropogenic event that further degraded the stream was the removal of a 100-year-old dam declared unsafe by inspectors in 1996. The short-term removal of the dam released 100 years worth of accumulated sediment and debris into the downstream channel. This sediment was comprised of not only silt, fine sand and gravel but also relatively large materials such as coarse gravel and cobbles. This large slug of coarse sediment washed downstream and overwhelmed the sediment transport capacity of the stream. Aggradation in the stream filled in pools, created mid-channel bars, transverse bars, and in some instances, channel avulsions. The aggradation split channel flows and put stress on the channel banks, which in turn began to erode, adding more sediment to the system. The result was a domino effect of erosion, channel migration, and elimination of aquatic habitat.

An aerial view of the stream before this project began (fig. CS6–5) provides an illustration of the condition of the stream. Following this flood event, some stream channel work was performed by the U.S. Army Corps of Engineers (USACE) and Federal Emergency Management Agency (FEMA) in Plunketts Creek Township, but this channel work ultimately contributed to the further degradation of fish habitat in the creek. The hurricane flood events put stress on the ecology of the Big Bear Creek system. However, the ecology of the system was more severely affected by the forced removal of the dam on the Dunwoody-Big Bear Hunting Cub property that had fallen into critical disrepair.

Figure CS6–3 Author Bill Worobec at the Red Ridge Bridge on Big Bear Creek

Figure CS6–4 Most downstream PennDOT bridge on Big Bear Creek

Figure CS6–5 Aerial view of phase II treatment reach of Big Bear Creek prior to restoration

Project objectives

The original goal of this project was to stabilize and improve aquatic habitat on approximately 3.7 miles of stream and restore the stream to a high-quality, cold-water (class A) fishery dominated by native brook trout. Subobjectives of the project included arresting and preventing further scour at the Red Ridge Bridge and transporting sediment efficiently through the two downstream PennDOT bridges.

Stakeholders

The primary stakeholder for the project was the Dunwoody-Big Bear Hunting Club, a private group that initiated the project. The dam that was removed was situated on their grounds. Other stakeholders included the USFWS, which provided technical assistance, equipment, training, and services for the restoration project, along with construction monitoring; and the Lycoming County Conservation District, which administered the Pennsylvania Department of Environmental Protection (PADEP) Growing Greener Grant that funded the restoration activities.

Project design and construction

The project was designed using natural stream channel design techniques. After an initial assessment survey, a contractor was hired to perform a geomorphic assessment survey of the treatment reach. This included a total station survey for topographic features, as well as geomorphic features such as the stream thalweg, edge of water, and bankfull indicators. Physiographic features were also included. Streambed substrate was sampled using pebble counts and bar samples. In addition, stable reaches of Big Bear Creek were identified and surveyed as reference reaches for the restoration design. One such reference reach is shown in figure CS6–6.

The watershed drainage area ranged from 10.1 to 12.6 square miles. Regional curves of fluvial geomorphic relationships showed the bankfull width to be in the range of 38 to 42 feet, cross-sectional area ranging from 90 to 100 square feet, and bankfull depth to be from 2.1 feet to 3.2 feet. The reference reach information yielded a bankfull width of 39 feet and bankfull depth of 2.3 feet. The restoration design was performed by the USFWS Pennsylvania Field Office. The design included channel relocation and realignment, construction of flood-prone benches, bank sloping and bank stabilization, and installation of rock vane structures for grade control and bank stabilization.

Construction began in the summer of 1999. The cost of construction for the initial phase of the Big Bear Creek restoration was approximately $160,000 for treatment of 4,000 linear feet of stream. The treatment included 38 rock structures (J-hook and cross vanes) with seeding, mulch, and geotextile fabric stabilization for impacted streambanks and other disturbed areas. An example of a J-hook rock vane is shown in figure CS6–7. Dimension rock, cut from a quarry, commonly known as wall rock, was used to construct rock vanes, an example of which is shown in figure CS6–8. The approximately 500 tons of wall rock used for the structures in the first phase of the project, valued at $12 per ton, was donated by a local quarry. The rock was transported to the site at a cost of about $6,000, paid for with a Watershed Restoration and Assistance Program (WRAP) grant from the PADEP. The equipment used to set the rock cost $17,000 which included an excavator with a Balderson™ progressive link thumb and a 3.5-cubic-yard, rubber-tired loader. Approximate-

Figure CS6–6 A stable reach of Big Bear Creek, a B3 stream

ly $20,000 was spent for the preliminary stream survey, design, and preparation of permits. Onsite supervision and construction labor was estimated to be $60,000. Miscellaneous construction materials, such as seed, mulch, and geotextile material, cost about $2,000. The total cost broke down to $40 per linear foot of treated stream. However, caution must be exercised when using unit costs for estimating or comparing stream projects, as each project has its own level of preparatory effort and construction intensity. This project was performed when the natural stream channel design approach was still, for this region, in its infancy. Today, costs can be much higher and typically include performing a watershed assessment, as well as addressing more detailed and rigorous permitting requirements.

Another way to examine the project costs for the first phase of the Big Bear Creek restoration project is to divide the cost among the structures installed in the treatment reach. For the Big Bear treatment reach, the estimated actual construction cost for the rock vanes was about $650 each for the J-hook vanes and $1,300 for each cross vane. These figures are only for the construction phase of the structures and do not include the preparatory work such as stream analyses, survey, design work, and permitting, nor does it include the stream channel work needed to construct flood-prone benches and to bring the channel itself to within proper and appropriate channel dimensions and geometry.

Rootwads were not used in the project design or the construction phase of the project, but it was estimated that they could be installed for approximately $400 each.

Rock vane performance

For the most part, the rock vanes performed well. However, problems were encountered with some of the vanes. Some of the vanes had to be tweaked, some needed to be rebuilt after being damaged by high flow, and some vanes were torn out and relocated to achieve the objective for each vane. Two construction crews worked in the phase II and phase III of the project. One crew had very good luck with their structures, but the other did not. Unfortunately, communication between crews was lacking, especially with regard to procedures, construction techniques, and expectations. Consequently, the good luck was not always shared. When the construction operator paid attention to detail and maintained the patience required in fitting the rocks securely together, the structures held up against bankfull events. Attention to detail in the construction resulted in a stable structure.

Figure CS6–7 J-hook rock vane on Big Bear Creek

Figure CS6–8 Cross vane on Big Bear Creek

Problems encountered with rock vanes

Problems were encountered when the construction crew was rushed to complete the vane installation. Early problems were a product of inexperience on the part of both construction crews, each experiencing a learning curve for developing the most efficient procedures to use in these types of projects. Initially, for example, the rock vanes were laid out in great detail with rebars driven into the streambed (fig. CS6–9). Later, a technique was developed where the butt rock was laid to specifications, and then a target rock was placed out in the stream along the line of the vane. It became the job of the person on the bank to sight along the butt rock and target rock during the installation of the other footer and vane rocks to direct the operator in aligning the vane rocks to that line. The vane was then built by checking the elevation every 10 feet and holding the vane rocks to a 0.1 feet tolerance.

Typically, the problems encountered were related either to the alignment and design of the structure and its effect on the streamflow or the problems related to the construction of the structure itself, where one or more of the vane rocks (and sometimes the footer rocks) would be washed out of position, compromising the function of the vane.

Problems related to the alignment and design of the structure also appeared in the effect the structure had on the flow lines of the stream. Early designs consisted of only a plan view. The structures would be built and then field evaluated. Sometimes subtle adjustments were required to align the streamflow properly, and other times, the structures were relocated or removed. In the subsequent phase of the restoration project, a different approach was tried where all structures were designed in great detail and constructed exactly to the design specifications. Again, several structures had to be relocated or reconstructed due to the inability of the designer to anticipate every aspect of the design in three dimensions and the lack of appropriate field adjustments. The only way this approach would be viable is when the designer also stakes out the structures in the field. The designer can then see what design adjustments may be necessary. The designer can return to the office to draft a final, revised design that can be handed off to build. While a detailed design is valuable, some flexibility in adjustment of the structure design to the site must be allowed, as well as to be able to adjust the implementation of the design to unforeseen elements in the field.

Some structures were found to be out of spec with the design drawings. Some of the vane slopes were steeper than the 10 percent maximum recommended in the design specifications, based on Rosgen's experience. Since the time of this project, the recommended maximum slope has been reduced to 7 percent. In some cases, the slope of the structure, although within range, was actually too steep for the particular setting. The steep slopes reduced the effectiveness of the vane in providing a gradual reduction in the fall energy of the water flowing near the banks. Further information on these structures is provided in NEH654 TS14H and NEH654.11.

Occasionally, a vane was constructed at too great an angle from the bank. Vanes with an angle greater than 30 degrees with respect to the bank were found to be less effective. In some cases, this larger angle resulted in significant backwater eddy currents that served to scour the bank behind the structure. In other cases, the design specifications showed the correct alignment, but the layout of the structure during construction was not accurate. On occasion, the operator

Figure CS6–9 Staking out a cross vane

built the structure using the eyeball technique. That is, the structure was built by an operator substituting a trained operator eye for the use of a construction transit or laser level. Consequently, the slope along some of the problem structures measured greater than 7 percent (or even 10%), as constructed.

Working with contractors

The construction crew must be aware of allowable tolerances and know the importance of adhering to the design specifications. If the construction supervisor is either inexperienced or not insistent enough to maintain the proper tolerance with the rest of the construction crew, problems can occur. In rock vane construction, strict attention to detail is critical for the structure to maintain its physical integrity during high-flow events and maintain its design functionality. Ultimately, the construction supervisor must ensure that the structures are built according to the design specifications and that the construction crew understands and complies with the project procedures, specifications, and objectives. The supervisor must insist that the contractor build the structures according to design and use the proper techniques. The supervisor should be experienced enough with natural stream channel design procedures to make field adjustments, if required.

Construction of natural stream channel design structures should be contracted on a time and expense basis. This ensures that the contractor will be justly compensated for taking the time to construct the structures correctly; hence, the contractor is more willing to make adjustments (or rebuild a structure, if necessary). The authors note that the success of the project largely depends on the disposition of the operator. Patience, persistence, and secure self-esteem are qualities to look for in an operator. The setting of rock in difficult conditions requires both patience and persistence. Operators must also have enough self-esteem not to take it personally when asked to rip out and rebuild their work if it is out of specification or the structure alignment or location does not produce the desired effect on the streamflow.

Allowances must be made in the design and permitting procedures for in-the-field changes or adjustments to the restoration design. The experienced supervisor or designer must have the latitude to make adjustments according to observed flows through the structures in the field. Having this latitude can make the difference between a successful project and one that must later be adjusted or rebuilt.

Additional lessons learned

Rock—Wall rock (fig. CS6–10) is preferred over smaller R5–R7 size rock. Vanes constructed of the smaller rock give the appearance of being simply piled, and it is much harder to plug the holes between the rocks. The wall rock is more massive and lends itself to placement with an excavator. The wall rock also provides good footer rocks. Size specifications for the wall rock for this project indicated rock dimensions should be between 3 and 6 feet, with no dimension less than 3 feet and no dimension greater than 6 feet. The rock was to be of hard sandstone with an alkaline pH, or limestone. One of the biggest challenges is transporting rock of this size. Large steel-bed dump trucks were used to deliver 8 to 10 rocks at a time. Depending on the truck tailgate configuration, it was sometimes necessary for the excavator to unload the rock from the truck. Typically the rock was stored at a staging area near the construction site and delivered

Figure CS6–10 Loading a typical wall rock used for vane construction

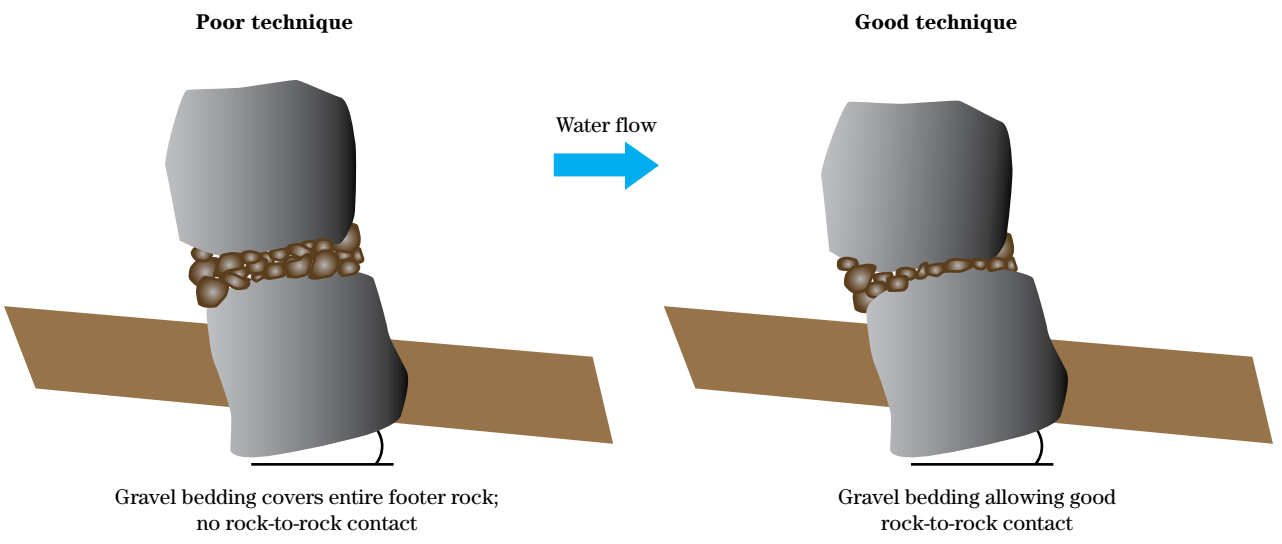
to the excavator using a rubber-tired front end loader (fig. CS6–10).

Excavator—The key to efficient and successful placement of rocks for the rock vanes was finding an excavator with a Balderson progressive link thumb, coaxially mounted on the bucket pin (axle) (fig. CS6–10). The progressive link connection shares the same pivot axle as the bucket, thereby allowing the thumb to follow the bucket along its entire pivotal swing. In other words, the thumb can grasp a rock and hold it, no matter how high the operator swings his bucket up. Other thumbs not coaxially mounted and without the progressive link have a limited range of radial motion, so that when the operator rotates the bucket back upwards, the thumb cannot follow. Consequently, the bucket pulls away from the thumb, and whatever is in its grasp falls out. The Balderson™ thumb is not the only thumb assembly that will work for rock vane installations, but it is the most efficient.

Rock vane installation—Installation of the rock vane usually begins with keying a footer rock into the bank and a vane rock that constitutes the butt rock of the vane. Footer rocks should be of comparable size to the vane rocks. Typically, a target rock is then placed in the stream for sighting alignment purposes. A second person with a two-way radio to talk directly to the operator is usually needed to guide the operator in aligning the vane rocks. Another lesson learned is to angle the footer rock slightly, tipping it in the upstream direction as shown in figure CS6–11. The vane rock is less likely to be pushed downstream off of its footer rock. Tipping the footer rock also facilitates fine adjustments in the vane rock elevation. Simply by moving the vane rock a bit laterally (perpendicular to the vane line), the vane rock elevation is adjusted slightly up or down. Most of the vanes on Big Bear Creek have the vane rocks set to the design elevation with a tolerance of ±0.1 feet. A laser level was typically used to check elevations, usually at a 10-foot interval along the vane.

In the second year of the project, a second crew was brought in to help with the construction. This crew was experienced, having just completed another similar project. However, this team had slightly different approaches to constructing rock vanes. Perhaps the most significant difference was that many of the vanes were constructed of large rocks with either no footer rocks for the throat rocks of the cross vanes or relatively small and flat footer rocks. This technique

Figure CS6–11 Schematic of vane rock installation



resulted in less excavation into the stream subbase to set the throat rocks at or near the streambed level. Over the long term, it was found that these structures were much more susceptible to washouts and displacement of the throat rocks than those where all the vane rocks were carefully placed on footer rocks of comparable size. High velocity flows moving over the vanes scoured out the downstream pavement and subbase of the streambed, tipping or causing settling of the footer rock, which in turn caused the vane rock to fall into the scour hole. The process repeated itself, eroding a scour hole downstream of the tipped rock until it rolled into the hole, thereby moving the rock downstream. An example of this is shown in figure CS6–12.

Footer rocks of the proper size are typically embedded into the streambed to a depth greater than the scour depth and, thus, resist washout. Figure CS6–13 shows a footer rock being set. Two other features of a properly footed vane rock also help to resist tip-outs of the vane rock. First, the vane rock is typically offset to the upstream side of the footer rock (fig. CS6–11) leaving a small sill at the base of the vane rock. This sill sometimes acts as an energy dissipater for water pouring over the vane rock. The second feature is that a properly installed footer rock is tilted slightly upstream. For the vane rock to move downstream, it must also move uphill (fig. CS6–11).

Once the footer rock is set, leaning slightly upstream, gravel bedding may be dribbled on top of the footer rock. The vane rock is then placed on top and scrunched back and forth until there is direct contact between the vane and footer rock at least at one point (fig. CS6–11). For a time, one construction crew ignored this tenet and just placed vane rocks on top of bedding gravel. Without the rock-to-rock contact, it is relatively easy to displace the vane rock and roll it off of the footer rock. Some of the vane rocks observed had most of the gravel bedding scoured out from beneath the vane rock. Patience and persistence are required. If the vane rock is not set at the proper elevation, the footer rock must be raised and some streambed material moved underneath to support it. If the vane rock must be lowered, a deeper hole should be dug before replacing the footer rock.

In soft, fine material, it is sometimes extremely difficult to achieve the proper elevation and alignment between adjacent rocks. Where one rock is properly set according to grade and alignment but repeated

Figure CS6–12 Example of vane rocks with small footer rocks and inadequate long-term performance

Figure CS6–13 Setting a footer rock (upstream direction is to the right)

attempts to set the adjacent rock fail, it was found to be advantageous to set the troublesome rock close to proper position and then set the next rock down. Once this next rock is set, the operator can go back to the troublesome rock and complete its positioning according to proper grade and alignment. The adjacent rocks on either side of the troublesome rock help to hold it in place during minor adjustments. This procedure may be repeated down the vane.

Subtle adjustments to the vane rocks during placement or judicious selection and orientation of the vane rocks offer opportunities to be creative. One example noted during the construction of this project was that angling the top surface of the throat vane rock toward the upstream side tended to increase the hydraulic jump downstream of the vane and promoted better scour in the structure pocket pools. The use of dished rocks for throat rocks and pour-overs to concentrate flow can create an aesthetically pleasing effect.

The most prevalent problem—Most of the problems observed with structure meltdowns where the vane rocks washout or are displaced during a high-flow event are a result of inattention to detail during construction. There must not be any open spaces between the structure rocks (fig. CS6–14). Open spaces result in the formation of suck holes during high-flow events. The water becomes accelerated as it passes through the hole between the rocks resulting in a high-velocity water jet. This jet will have much more localized power than the stream in general and can dislodge and cause the erosion of the bed material around and behind the footer rock. If the erosion persists, it can result in the movement or tip-outs of the footer rock, which in turn dislodges the top vane rock. It was noted that filling the gap holes with tightly packed coarse gravel is **not** a sustainable solution or practice. Where the streambed material is of fine material and there is a shortage of delivered cobble rock, grout bags can be used to fill the holes. The grout bags used in this case were sand bags filled approximately one-half to two-thirds of a mixture of sand and Portland Cement. Another method is to use a geotextile fabric (filter fabric) as a barrier to keep finer material from washing through the holes. The fabric is placed on the upstream side with the top of the fabric kept even with the fill line. The upstream side of the rock vane should be filled in with bed material up against the filter fabric. This procedure is labor intensive and may present challenges working in moving water.

Figure CS6–14 Gap holes, which in high flow, become suck holes

Evidence of this process was observed in many of the failures that occurred in the second year of construction in Big Bear Creek (figs. CS6–15 and CS6–16). The spaces between the rocks must be filled and preferably barricaded on the upstream side using large rocks that will span the hole. In later projects, the authors found that it is advantageous to have smaller rock delivered along with the wall rock for this purpose if the streambed material does not contain sizable cobbles.

In many cases, filling the gap holes makes the difference between a successful and sustained structure and one that will have to be rebuilt following a high-flow event.

Pool construction—Each structure should have a scour pool associated with it. Over the long term, this scour pool will develop naturally by eroding the streambed materials. The problem with this approach is that the system remains relatively unstable until the scour pools develop, and the potential fish habitat is not fully realized until that time. Since it requires several bankfull events to complete the pool scour, it could be years before the pools fully develop. The natural scour of the pools also adds to the sediment

load of the system. The sediment removed from the pools may be deposited in a riffle section below the structure resulting in a splitting of flow. This behavior can increase the shear stress near the banks and, in turn, may increase bank erosion. Any disproportionate input of sediment at one location can set off a series of impacts downstream—the domino effect.

A much better approach is to give nature a hand, and excavate the scour pools during construction. The pools for cross vanes should be excavated so that the pool starts about halfway through the structure, with the deepest part of the pool roughly across from the butt rock of the structure. The glide- (tail-out slope) out of the pool on the downstream side of pool should have a slope based on the analyses of pool characteristic dimensions from stable stream reaches (reference reaches).

For J-hook vanes, pools should begin two-thirds of the way into the structure with the deepest part roughly across from the butt rock. As a rule of thumb, the glide should extend approximately one vane length downstream from the butt rock.

Fill from the pool excavation may be used to fill in against the rock structure on the upstream side of the vane rocks. In some cases, it may also be used in the construction of flood-prone benches, a technique used to stabilize a steep, eroding bank. For improved fish habitat, make riffles at low flow half the pool width for deeper riffle flow.

It was found that rounded throats for cross vanes were more effective than straight throats. Similarly, the J-hook vanes needed to retain the shape of a "J," rather than an "L." The more pointed throats concentrated flow better than those that were blunt. By concentrating the energy more efficiently, sediment was more readily transported. Consequently, the scour pools are more likely to be maintained without aggradation.

Habitat rocks—The installation of habitat rocks is an advanced technique for fish habitat enhancement. In several instances, habitat rocks were placed in the stream and found to be a detriment, rather than an enhancement. Habitat rocks placed in glides resulted in aggradation on the downstream side. In-line placement of habitat rocks parallel to the streamflow caused aggradation between rocks. A better technique was to use a cluster alignment of three rocks, one upstream and two downstream, but offset from the first with respect to the streamflow lines. Adequate spacing is also needed between structures to incorporate habitat rocks; otherwise, the habitat rocks promote aggradation. The authors suggest placing the habitat rock cluster no closer to the butt rocks of the upstream structure than one bankfull width. The downstream extent of the habitat cluster should be no closer to the downstream vane than half of the distance between

Figure CS6–15 Vane with a tip-out vane rock due to the scour from a gap hole

Figure CS6–16 Tip-out due to scour from a suck hole

the vanes. If these dimensions cannot be met, the habitat rocks should not be installed at that location.

Habitat rocks must also be installed with footer rocks of comparable size. For habitat purposes, biologists suggest that the habitat rocks be set at an elevation to be just submerged during normal spring flows. This rule of thumb was supported by trial and error on this project.

Maintaining cross section—It is important that the cross-sectional area be maintained through the structure. If the structure is too wide, there is a chance that the flow will spread out across the downstream portion of the structure with a subsequent reduction in velocity. With the velocity reduction comes a reduction in the power of the water and in the capacity for sediment transport. This ultimately leads to aggradation downstream of the structure. A structure which constricts the flow may promote additional scour as the velocity of the stream accelerates through the structure, also promoting erosion of the banks upstream of the constriction. In fact, the installation of an undersized structure may defeat the purpose for which it was installed.

One significant lesson learned was related to the gravel and streambed fill material that is placed between the structure and the bank (representing the acute angle of the structure). In several cases, the fill placed by one of the construction crews ran from the upstream edge of the vane rock to the top of the bank (at the bankfull level) all along the leg of the vane. So, instead of a relatively flat, tapering ramp extending horizontally to the bank bordered on the downstream side by the vane rock (fig. CS6–17), the fill in these cases ran from the vane rock to the top of the bank all along the leg of the vane. The bank then extended to the edge of the vane (now at a different slope), illustrated schematically in figure CS6–18. The result of this mistaken practice was a reduction in the cross-sectional area and an increase in flow depth and high-flow velocities. The structures could not function as designed since there were no ramps for the water to run up on and expend its energy on. Rather, the flow was deflected away from the vane rocks and maintained much of its velocity as it was channeled to the center. The vane functioned as a channel constriction. Once discovered, the construction crew was required to dig out the fill along these structures until flat ramps were formed.

Machine tracks—The tracks of the excavator were visible in the reworked streambed, and it was noticed that these tracks acted as energy dissipaters. While it was probable that the tracks would have been filled in by the stream over time, the tracks were dusted out before the machine left the stream to minimize effects on flow patterns in the stream.

Figure CS6–17 Cross vane illustrating the proper upstream fill along the vanes, leaving a flat horizontal ramp along the vane

Figure CS6–18 Schematic representation of the improper fill technique on the upstream side of the vane

Working instream

Perhaps one of the most potentially controversial issues in the construction of the restoration project was working in the stream channel, in the wet, with heavy construction equipment (fig. CS6–19). The USFWS personnel practiced a no tolerance policy regarding equipment malfunction and leaking fluids. If equipment leaked or dripped any nonaqueous fluid, it was immediately removed from the channel area and repaired.

Typically, the biggest concern over working in the wet channel is the amount of sediment stirred up by the equipment and allowed to migrate downstream. Allowing the construction equipment in the stream actually minimizes the construction time (and, therefore, the disturbance time) over other options designed to arrest some of the stirred-up sediment. Where equipment is working in the stream, the sediment that moves downstream is predominantly sediment that is already in the stream system and does not represent a new sediment input into the system. Construction activities are performed during low flow so that the sediment mobilized during the construction is mostly fine sand, silt, and clay. The release of this fine sediment is episodic for usually less than 10 hours per day and many days less than 8 hours. The streams usually clear up between construction events.

Two of the authors of this case study, Putnam and Worobec, studied the sediment transport characteristics of Big Bear Creek and its unstable reaches prior to this project. Their estimates ran as high as 10,000 tons per year released into the system prior to construction. This sediment included coarse and very fine sediment. The amount of fine sediment released during construction activities pales in comparison to this estimate. Consequently, this construction technique was deemed the most cost-effective method with relatively low risk of ecological impact. The authors' evaluation was supported by the postconstruction monitoring data. Results of ecological monitoring, both pre- and postconstruction, indicate that equipment working in the channel during construction caused no long-term adverse effects. In fact, monitoring showed that macroinvertebrates made a healthy rebound within 2 months of the cessation of construction activities in the channel. Monitoring also showed a significant increase in trout populations the following season.

Other options can be considered for doing construction in a stream (although not considered for this project), such as:

- working strictly from the bank
- diverting the stream to an alternate channel and doing construction in the dry channel
- pumping the water around the construction site
- diverting the water to one side of the construction activities

Several comments will be made on each in comparison to doing construction with the equipment in the channel. The amount of the sediment released for each scenario must be evaluated over the long term along with other effects on the stream.

Working strictly from the banks may be a viable option in an urban setting where the riparian vegetation has been removed or is minimal. On Big Bear Creek, this scenario would have destroyed large portions of riparian vegetation, which in turn would most likely have destabilized the banks and resulted in an increase in bank erosion, moving large quantities of coarse and fine sediment into the system.

Figure CS6–19 The open heart surgery approach, working with heavy equipment in the channel of Big Bear Creek

An alternate channel would also destroy riparian vegetation, and a significant amount of sediment would be released from this freshly constructed channel due to the recent disturbance of the soils during construction. The cost of this option rivals the cost of the restoration activities just on its own.

A pump around system is also very costly for the amount of flow that would need to be pumped. This option carries with it a distinct potential for erosion at the discharge of the pumping system, requiring the construction of energy dissipation structures. In addition, a stilling well would to be required for the upstream intake. Aquatic life would be either be prevented from passing the project reach or pulverized by the pumps. This option is also very energy intensive.

Diversions in the channel limit the mobility of the equipment and prevent the construction crew from evaluating the effect of the constructed structures on the streamflow lines. The velocity and volume of the streamflow is constricted to the remaining portion of the channel, which significantly increases the stress on the opposite bank, raising the potential for significant erosion during construction. Sediment will still be dislodged in the setting up and moving of the diversion barriers.

Project evaluation

Much has been said about the deficiencies and problems with the design and construction of this project. In some respects, it lends itself to discussing lessons learned, particularly since this was the first project of this type and scope built by the personnel of the USFWS Pennsylvania Field Office. There were many lessons to be learned in the process. It should be noted, however, that overall, the project was a large success. What was once a highly unstable stream contributing upwards of 10,000 tons per year of sediment to the stream system is now 3.7 miles of stable stream with a sediment load of approximately 2,100 tons per year.

Examples of successful restoration techniques are shown in figures CS6–20 and CS6–21. In figure CS6–20, a landslide area is shown before restoration. A flood-prone area was constructed along the eroding bank to move the main channel away from the bank, leaving only relatively slow moving water on the flood-prone bench next to the susceptible slope. The relocated and resized channel was stabilized with several cross vanes along the reach. A completely aggraded channel reach that resulted in the formation of a channel avulsion is depicted in figure CS6–21. The gravel was excavated from the original channel, and the banks were resloped. The channel avulsion was filled in. The newly excavated channel was stabilized with cross vanes. As vegetation fills in, the need for the vanes lessens.

Gravel bars

The effects of this project were evaluated based on the results of an extensive monitoring program that evaluated aquatic plants, macroinvertebrates, fish populations, and sediment transport before and after project completion. The aquatic life and macroinvertebrates showed signs of rebound 2 months after the completion of construction activities in the stream. A marked increase in the population of brook trout was found the following season. Aquatic plants are now thriving, as well as a diverse population of macroinvertebrates. The rocks in the streambed are now turning blackish indicating the streambed is now stable enough for moss and algae to grow on the cobbles that make up the streambed. Prior to construction the rocks were light gray, their native color, indicating active transport. Local fishermen noted that Loyalsock Creek used to run brown during an intense rain storm with a more noticeable, intense brown streak coming into the Loyalsock at the confluence with Big Bear Creek. Since the completion of the restoration project, fishermen observe the confluence waters of Big Bear Creek as it pours into the Loyalsock Creek and note that the waters from Big Bear Creek create a plume of clear water within the brown muddy waters of the Loyalsock.

There is now no evidence of continuing instability along the treatment reaches. Based on the parameters described above, the authors feel the objectives of the project have been met and that the project is a success.

Some of the most important lessons learned on the Big Bear Restoration project are:

- When using multiple construction crews, communication is one of the keys to success. Communication must start with making sure everyone is onboard with the project objectives, techniques to be used, and performance expectations.
- Attention to detail during construction is paramount.
- Review the second point.
- Re-review the second point.

Figure CS6–20 Treatment technique used for a landslide area

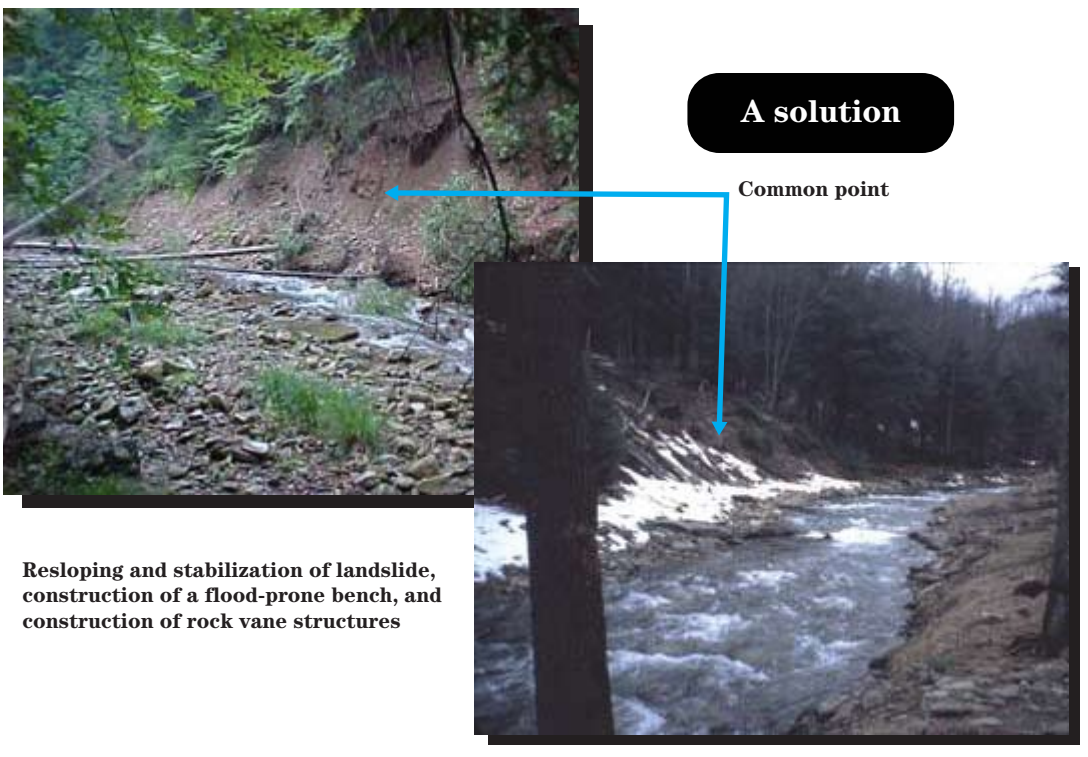


Figure CS6–21 Treatment of a channel avulsion with rock structures

Case Study 7

Spafford Creek, Otisco Lake Watershed, New York

Issued August 2007

Cover photo: Completed section of Spafford Creek, Otisco Lake Watershed, New York

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 7 Spafford Creek, Otisco Lake Watershed, New York

By David Walowsky, civil engineering technician, U.S. Department of Agriculture, Natural Resources Conservation Service, LaFayette, New York

Introduction

The Otisco Lake Watershed is in a rural, primarily agricultural area with approximately 4,000 acres of forest. Slopes in the forested areas are in excess of 35 percent. The soil type in the stream channel is Teel silt loam. This soil type is characteristic of flood plains, and these soils are formed from alluvial deposits of silt and very fine sands. The Teel series is normally flooded in the spring and rarely flooded during the growing season, as found on this site. This soil is easily eroded, by evidence of the existing conditions along Spafford Creek, south of Sawmill Road (fig. CS7–1). Phelps gravely loam, Rhinebeck silt loam, and Fredon loam soil types are found associated with the Teel series.

The stream has been typed as a G4, according to the Rosgen Stream Classification System. A G4 stream type is deeply incised in depositional material, very unstable due to its high sediment supply, moderate gradient, low width to depth ratio, and low sinuosity. The reference reach is an E4 stream type with a low sinuosity, gentle to moderate channel gradient, and low width to depth ratio. The E4 type stream has a riffle-pool pattern. Streambanks of this type are of finer material than that found in the channel bed. The banks are stabilized with wetland vegetation that has formed extensive, deep root mats similar to the stabilized reach of Spafford Creek north of Sawmill Road to Otisco Lake. The proposed stream type is a C4, meaning it is a slightly entrenched, meandering, gravel dominated, high width to depth ratio, riffle-pool channel with a well-developed flood plain. The sediment supply is low due to the stability of the streambanks. The natural progression over many decades is to accelerate the process and transform the existing G4 to a C4.

Figure CS7–1 Typical picture of streambank erosion along entire project reach south of Sawmill Road, Spafford Creek, NY

Until the 1930s, the landscape included a riparian forest buffer approximately 250 feet in width on both sides of the stream channel. Clear-cutting and straightening of the channel have detrimentally impacted the fish habitat and water quality. In 1998, the riparian forest buffer was eliminated by the landowner. Large, mature black willow and sycamore were cut and removed from the stream corridor, with stumps left in place. Banks are steep to vertical, and the vegetation varies from none to moderately dense grasses and weeds. Some redosier dogwoods are present. Bank materials are mostly fine-grained with one gravel lens.

Establishing a vegetative buffer along the banks of Spafford Creek would eliminate the detrimental effects of tillage equipment on the plant root systems that are stabilizing the streambanks. The team evaluated the conditions of the stream corridor upstream and downstream of the proposed project. The downstream reach is well vegetated with a riparian forest buffer and has low-velocity flows and deep pools. The upstream section was similar to the proposed project reach, but with a steeper slope. Phase I of construction began in August 2001 and concluded in October 2001. Phase II of construction began in July 2002 and concluded in September 2002. The total project cost was $263,649.

Determining project goals

The team approach was probably the most important aspect in developing the project plan. The cooperation between the following people and agencies led to success of the project:

- the landowner
- Onondaga County Soil and Water Conservation District (SWCD)
- U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS)
- New York State Department of Conservation – Region 7 (DEC)
- U.S. Fish and Wildlife Service (USFWS)
- the Honorable James Walsh, Congressman

A technical team was created and consisted of representatives from the SWCD, NRCS, DEC, and the USFWS. The team took approximately 8 months to develop a comprehensive plan. The restoration objectives were to:

- reduce streambank instability and erosion and improve the quality of the fishery by designing a natural stream system that neither aggrades nor degrades
- use principles of fluvial geomorphology (how landforms change with time under the influence of streams and rivers) to stream corridor management
- reduce the amount of sediment load to Spafford Creek, which is the main tributary to Otisco Lake and a drinking water source managed by Onondaga County Water Authority (OCWA)
- reconnect the riparian corridor that is present at the upstream and downstream extents of the project area

The stream has the flexibility to move laterally with little interference. What is lost on one side is gained on another. Three bridges limit lateral movement, but they are approximately 1 mile apart. Due to the past logging operations, the stream experiences a significant amount of turbidity after approximately a half-inch rainfall event.

The biological attributes of the site were analyzed during the summer of 2001 and September 2002. The data shown in table CS7–1 are from the USFWS State Office located in Cortland, New York. The results of the fish sampling are shown graphically in figure CS7–2.

Design criteria

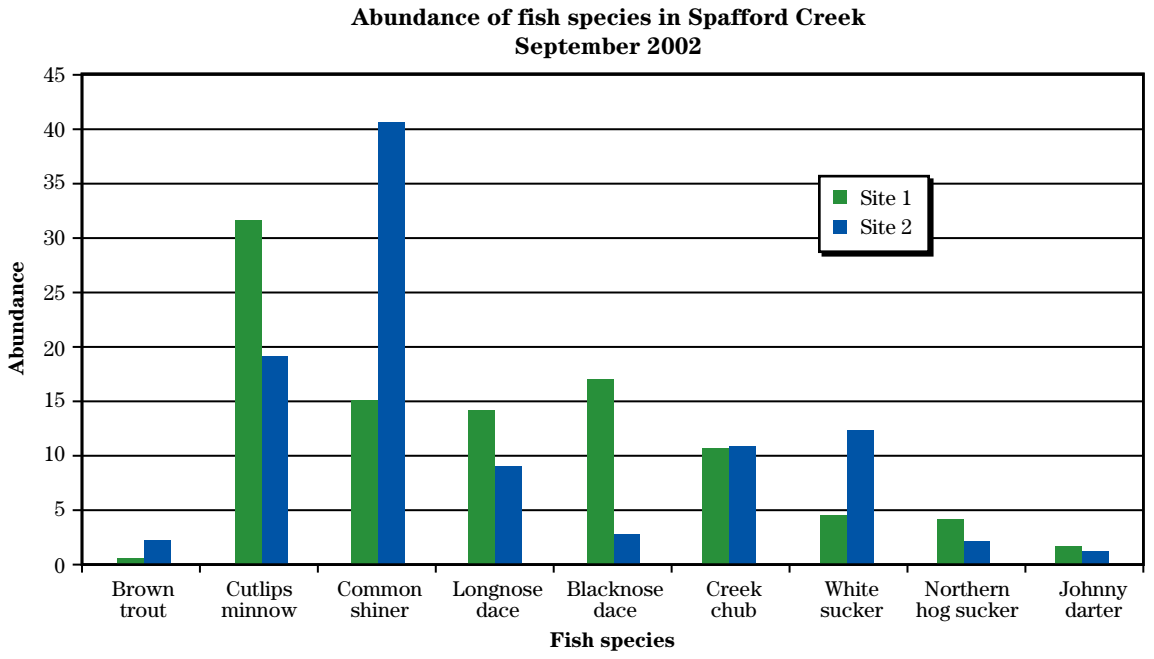
The design storm was the 10-year, 24-hour storm. This estimated discharge is 975 cubic feet per second. The hydrologic investigation of this reach included an in-depth analysis of the flow, the duration and frequency, and the sediment-carrying capacity. One of the most critical characteristics of a stream channel reach is identifying the bankfull stage. For this project, all bankfull indicators pointed to the top of the flood plain. This stream reach was typed as a G4 using Rosgen's Stream Classification System.

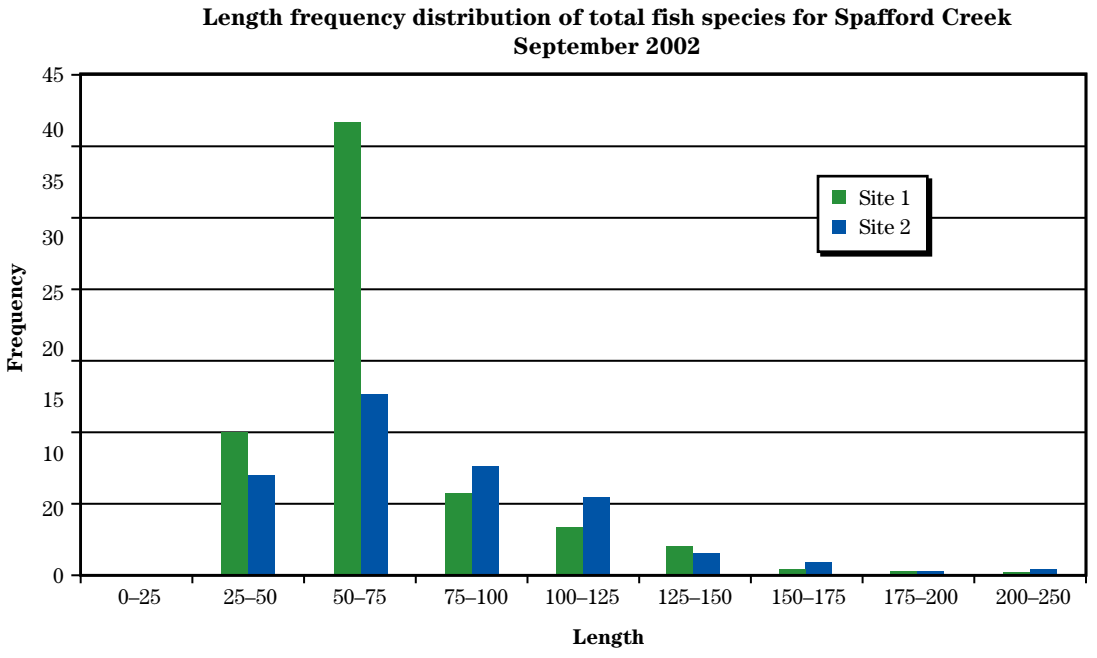
The analysis of the flow included the retrieval of stream gage data from the U.S. Geological Survey (USGS). The stream gage used for this project was located on Spafford Creek at the Sawmill Road Bridge. Baseflow velocity was measured at 1.9 feet per second. Bank pins were set to determine future erosion rates, and a cross-sectional survey was conducted on June 19, 2001. A typical 1.2-year storm event of 1.9 inches was experienced on June 21, 2001. This event caused 3.3 cubic yards of material to be eroded in a 75-foot length of streambank.

Table CS7–1 Spafford Creek fish species total numbers and percent abundance for two electro-fishing sites in September 2002

Fish	—Site 1—		—Site 2—	
	Total	%	Total	%
Brown trout	3	2.1	1	0.5
Cutlips minnow	28	19.2	68	31.5
Common shiner	59	40.4	33	15.3
Longnose dace	13	8.9	31	14.4
Blacknose dace	4	2.7	37	17.1
Creek chub	16	11.0	23	10.6
White sucker	18	12.3	10	4.6
Northern hog sucker	3	2.0	9	4.2
Johnny darter	2	1.4	4	1.8

Figure CS7–2 Results of fish sampling in Spafford Creek





Geomorphic data and analysis

Table CS7–2 shows the data that were collected or derived for the Spafford Creek restoration reach and a suitable reference reach. The reference reach is one that has similar valley characteristics and is located within the same geographical area. The stream chosen was Fall Creek in Cortland County. This stream reach was typed as an E4 using Rosgen's classification system (Rosgen 1994). The proposed reach was to be designed as a C4 stream type using Rosgen's Stream Classification System and the geologic time frame for stream channels. The Rosgen approach to geomorphic channel design is described in NEH654.11.

Tables CS7–3 and CS7–4 display results of sediment sampling and sediment transport analysis for Spafford Creek.

Construction issues

The riprap was hand selected at the quarry and identified with marking paint for shipment to the site (fig. CS7–3). The large riprap for the project was to be limestone of at least 4,200 pounds and stackable. The riprap was selected from the Callanan Quarry located north of Morrisville, New York. The riprap was hauled to the site by RMS of Dryden, New York. A total of 29 loads of riprap were delivered to the site. Delivery took approximately 6 days.

Table CS7–2 Spafford Creek hydraulic and geomorphic data for restoration reach and reference reach

Attribute	Units	Restoration reach (existing condition)	Reference reach	Restoration reach (designed condition)
Drainage area	mi^2	8.06	3.08	8.06
Bankfull width	ft	36	39	25
Bankfull depth	ft	4.2	3.7	5*
Width/depth ratio	—	8.6	10.5	7.2*
Bankfull cross-sectional area	ft^2	151.2	103.2	180*
Bankfull mean velocity	ft/s	2.3	2.0	1.9*
Bankfull discharge	ft^3/s	348	207	348
Bankfull maximum depth	ft	6.6	4.8	7.0*
Maximum depth of riffle to depth bankfull ratio	—	0.66	0.83	0.75*
Low bank to maximum depth at bankfull	—	0.60	0.76	0.76
Width of the flood-prone area	ft	58	165	40
Entrenchment ratio	—	1.6	4.2	1.6
Meander length	ft	400	520	400
Ratio of meander length to bankfull width	—	11.1	13.3	16
Radius of curvature	ft	131	179	131
Ratio of radius of curvature to bankfull width	—	3.6	4.6	5.3
Belt width	ft	200	330	200
Meander width ratio	—	6.9	4.2	8.0
Sinuosity	—	1.38	1.55	1.38
Valley slope	ft/ft	0.0028	0.0045	0.0028

* Indicates a mean value based on Spafford Creek survey and Fall Creek survey

Table CS7–3 Sediment characteristics, Spafford Creek

Materials	Spafford Creek	Fall Creek (reference reach)
Particle size distribution of channel material (mm)		
D_{16}	3	2
D_{35}	8	7
D_{50}	13	12
D_{84}	24	25
D_{95}	40	30
Particle size distribution of bar material		
D_{16}	4	2
D_{35}	7	4
D_{50}	10	5.5
D_{84}	12	10
D_{95}	65	12
Largest size particle at the toe (lower third) of bar	75	4

Table CS7–4 Sediment transport validation (based on bankfull shear stress)

Analysis	Result
Value from Shields diagram	0.15 lb/ft^2
Critical dimensionless shear stress	0.0663
Minimum mean bankfull depth calculated using critical dimensionless shear stress equations	1.56 ft

Figure CS7–3 Equipment and rock used for construction, Spafford Creek, NY

Design features

The vegetation present within this reach is sparse redosier dogwoods, sumac, and weeds of various types. The mature trees that once existed had been eliminated in November 1998. The proposed vegetation for this reach was a mixture to be seeded on the finished streambanks and the 20-foot vegetated buffer established along both sides of the top of streambanks (table CS7–5). Note: The seed mixture was broadcast-seeded and mulched within the confines of the streambanks and no-till-drilled within the buffer area.

A series of 34 J-hooks and 18 cross vanes were installed. The design angle of the vanes is 25 degrees with a vane length of 21 feet. The J-hook vanes and cross vanes provided stability for the bed and banks and also provided wildlife habitat, in addition to the specific seed mixture used for bank stabilization and wildlife cover. The cross vanes are designed to:

- create instream cover
- reduce excess shear stress away in the near-bank region
- direct flows to the center of the channel to maintain lateral stability
- increase sediment transport capacity
- prevent downcutting below the structure

Figures CS7–4 and CS7–5 show the installation of a J-hook vane. The J-hook vanes are designed to:

- redirect velocity distribution and high velocity gradient in the near bank region
- stabilize the streambanks
- provide energy dissipation in the deep pool that they create
- create cover for fish spawning and protection

In addition to the structures, the seed mixture planted on this site provides additional wildlife cover and benefits including:

- shading to reduce water temperature
- promoting deposition of sediment and nutrients
- providing a visually pleasing green area
- maintaining aquatic food sources

Table CS7–5 Seeding specifications, Spafford Creek, NY

Certified seed	Cultivar	P.L.S.* (lb/acre) (cool season)
Reed canarygrass	N/A	15
Orchardgrass	Axiom	5
Perennial rye	Linn	20
Switchgrass	Blackwell	10
Kentucky bluegrass	N/A	10

* Pure live seed

Figure CS7–4 Installation of a J-hook vane, Spafford Creek, NY

Figure CS7–5 J-hook vane installed and final grading of streambank followed by seeding and mulch, Spafford Creek, NY

Project performance

The design performed as planned except for some minor shifts in the top course of stone of four J-hook vanes. These stone shifts have not negatively impacted the streambanks. The original goals were met. It will be interesting to evaluate the habitat that has been created and the number and species of fish that hopefully have used this project area for spawning.

The designed structures chosen appear to be working as planned. The vegetation that was planted on the streambanks is still intact. The silt fence used during construction was left in place through the winter months (figs. CS7–6 and CS7–7).

Overall lessons learned

This project was the first of its kind for the author as a design engineer. The use of an excavator with a hydraulic thumb was specified in the contract but was not available. The large stone could have been handled much better and faster.

Figure CS7–6 Completed section of streambank stabilization, Spafford Creek, NY

Figure CS7–7 Completed section of streambank stabilization to include J-hook vane and intentional point bar, Spafford Creek, NY

Case Study 8

Copper Mine Brook, Burlington, Connecticut

Issued August 2007

Cover photo: Treated section of Copper Mine Brook, Burlington, Connecticut

Advisory Note

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Case Study 8

Copper Mine Brook, Burlington, Connecticut

By Charles Galgowski, Design/Planning Engineer, P.E., U.S. Department of Agriculture, Natural Resources Conservation Service, Tolland, Connecticut

Introduction

The purpose of this case study is to illustrate some of the typical problems and goals of stream rehabilitation in a suburbanized area of Connecticut. To accomplish this, the Copper Mine Brook Emergency Watershed Protection project (EWP) is described. This project illustrates the work done to address various stakeholders' goals; analyze risks, consequences, and uncertainty; select appropriate design tools and features; and evaluate performance. It is assumed that the tools Connecticut needed will also be useful for other areas with high-population density, particularly areas of the country with both glacial till and alluvial soils where erosive forces are produced by both water and ice attack.

Stakeholders for stream rehabilitation projects in Connecticut desire to address increasing numbers of objectives for stream projects. The Copper Mine Brook EWP project is presented to illustrate what some of these objectives are, what the local landscape looks like, and what design constraints exist. Two main objectives are to protect flood plain infrastructure and simultaneously maintain or enhance aquatic habitat. What stakeholders wanted to do, what rehabilitation features were finally used, and how the project has performed are described.

Location, stream order, and drainage area

Copper Mine Brook is located in west-central Connecticut and begins at the confluence of Whigville Brook and Wildcat Brook in the town of Burlington. It is a third order stream. At the confluence, the drainage area of Whigville Brook is 4.8 square miles, and the drainage area of Wildcat Brook is 2.3 square miles, for a total of 7.1 square miles for Copper Mine Brook (fig. CS8–1).

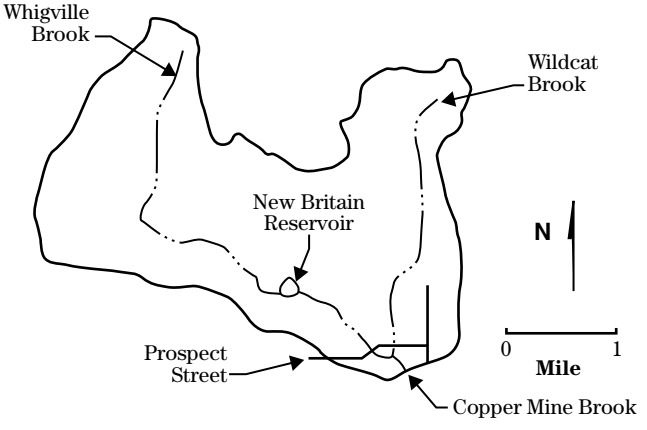
Site description

Damage assessment

Hurricane Floyd occurred on September 16, 1999. Two days later, the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) engineering staff in Connecticut inspected damages on Copper Mine Brook and Whigville Brook. The damages were viewed in the context of what measures could be used under the EWP to restore the stream. The EWP program is meant to remove sudden watershed impairments caused by catastrophic events. For streams, the goal is to restore the stream to prestorm or precatastrophic events. Using additional funds to improve the stream beyond prestorm conditions is not within the scope or intent of the program.

What ultimately became the project site started at Prospect Street and extended down Whigville Brook for 900 feet and continued down Copper Mine Brook for another 500 feet (fig. CS8–1). The aerial photograph (fig. CS8–2) shows the land use in the vicinity of the project site. The view shows woodland and hay fields north of Prospect Street. Whigville Brook flows from the west through New Britain Reservoir. Wildcat

Figure CS8–1 Plan view of Copper Mine Brook Watershed, near Burlington, CT



Brook flows from the east. Both pass under Prospect Street and form Copper Mine Brook. The locations of the damages found during the investigation are referenced to the stream centerline stationing used in figure CS8–3. The site features and damages found, starting at Prospect Street and working downstream, are shown in table CS8–1.

After the damage assessment, the Connecticut NRCS resource conservationist performed a geomorphic classification and assessment and an aquatic habitat assessment. Following are highlights from those assessment reports.

Geomorphic classification and assessment

The proposed project is located in a stream reach that displays characteristics of a Type C4 stream (Rosgen 1994) based on the approximation of entrenchment ratio, slope, width/depth ratio, sinuosity, and dominant channel materials. The reach is slightly incised with uninhibited access to the flood plain. It is a third order stream with average bankfull width of 20 feet (size S–4), dominant depositional features are point bars (B–1), and the meander pattern can be classified as irregular (M–3).

Figure CS8–2 Aerial photo of project area

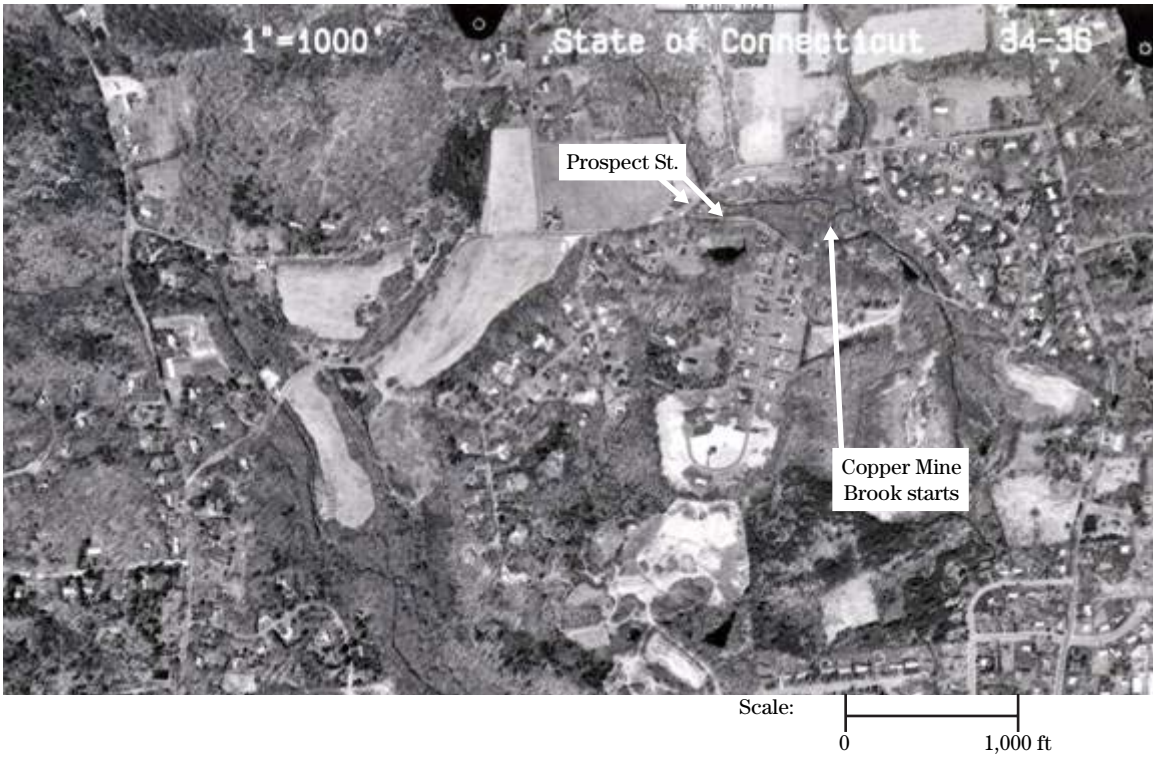


Figure CS8–3 Stream centerline stations, Copper Mine Brook EWP project

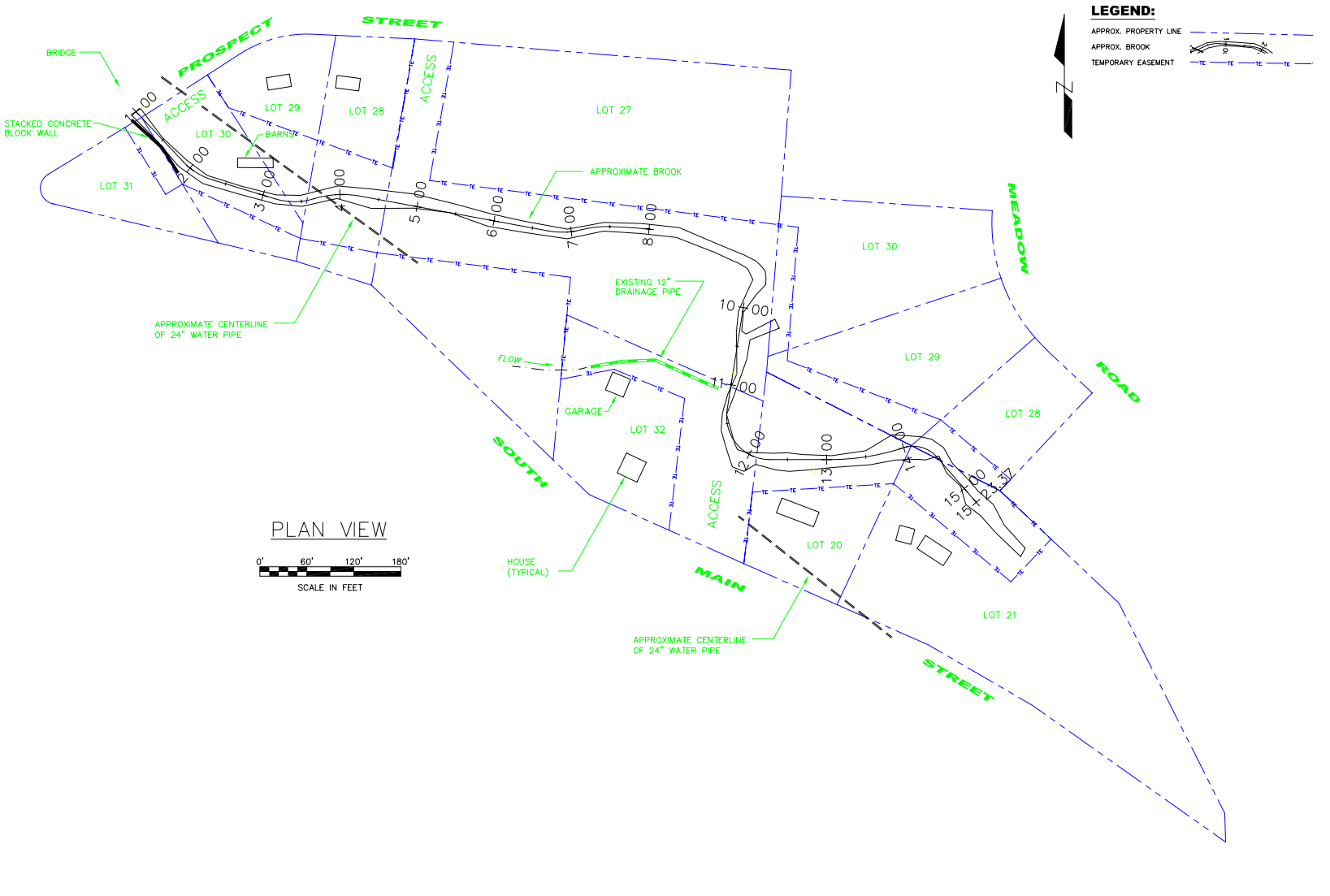


Table CS8–1 Site features on Copper Mine Brook EWP project

Station	Description
1+00 to 4+00	The brook flowed through three properties with houses, small barns, and onsite septic systems
1+00 to 2+00	Immediately downstream of the Prospect Bridge, the channel had started to incise along a 7-ft-high stacked concrete block wall near a house. This was a concern because the wall had fallen down two times previously
2+75	The brook had flooded two small cojoined barns and placed some debris on the flood plain
4+00 area	A 24-in-diameter water main, supplying the city of Bristol and owned by the New Britain Water Company, passed underneath the brook. This area had gravel and woody deposition that increased the flooding hazards to the houses and barns upstream. The debris had raised flood levels and caused much of the Hurricane Floyd flood to pass down the right (west) flood plain, increasing flood hazards on two houses further downstream
4+50 to 10+00	The brook flowed through 600 ft of woods. This part of the brook was in a fairly natural condition with one 2-ft-high waterfall created by LWM. It is not known whether this was here prior to Hurricane Floyd. Streambank erosion here was minimal, but there were some gravel deposits
10+25 to 15+25	The last 500 ft downstream flowed through five residential properties with onsite septic systems. Two of the properties had experienced some bank erosion. Some trees were being undermined, as were two stormwater outlet headwalls. Gravel deposits in this area had created higher flood elevations with an adverse impact on the foundation drain of one of the houses at lot 32. This house and its garage were also affected by overland flows on the west flood plain. These flows had entered the flood plain farther upstream because of debris partially blocking the channel at station 3+75 near the 24-in water main. These flows had to pass through a 12-in culvert previously used for local drainage

In general, the stream system within this reach can be classified as stable with isolated areas of accelerated streambank erosion. The areas of streambank instability are associated with previous alterations to the riparian area by the streamside landowners. As a result of Hurricane Floyd, the channel has experienced some morphological alterations including streambank erosion and redistribution of bed materials.

Aquatic habitat assessment

A physical stream assessment was conducted using the Stream Visual Assessment Protocol from the NRCS National Water Quality Handbook. The assessment revealed a Habitat Suitability Index (HSI) rating of ten (10). An HSI score greater than nine (9) is classified as excellent. The instream fisheries habitat identified included large woody material (LWM), deep pools, overhanging vegetation, cobbles/boulders, riffles, undercut banks, and thick root mats.

The current stream morphology provides the habitat complexity necessary for the maintenance of a sustainable cold-water fishery. The stream system currently supports a population of wild brook trout and wild brown trout. During the assessment, adult and juvenile brook trout and blacknosed dace were observed.

A cursory review of the benthic invertebrate community revealed dominance by pollution-sensitive invertebrates including mayfly, caddisfly, and water penny of the orders Ephemeroptera, Trichoptera, and Coleoptera, respectively. This is expected due to the health of the watershed.

Some of the stream features of noted value and their locations are shown in table CS8–2.

Watershed description and history

This watershed has a system of cold-water streams that support a sustainable wild brook trout and brown trout fishery and provide water for the City of Bristol. The tributaries are in steep watersheds with the headwaters of Whigville Brook at elevation 1,000 feet above mean sea level and with Wildcat Brook at elevation 900 feet. The confluence of the two is at elevation 380 feet where a flat flood plain area has formed. The New Britain Water Company has a water supply reservoir located about 1 mile up Whigville Brook with a drainage area of 4.1 square miles and a surface area of 10 acres (fig. CS8–1). The New Britain Water Company sells water to the City of Bristol and frequently draws water levels down in the reservoir. This drawdown provides some flood storage that can significantly reduce flood flows from storms smaller than the 10-year event. Although 80 percent of the watershed is forested, significant housing subdivisions have been built on the east side of Wildcat Brook, and less intense development has occurred throughout the watershed.

In the mid 1950s, the project site and surrounding land was predominantly used for vegetable crops. Since that time, much of the land has grown back to forest with trees about 30 to 40 feet high. Many of the houses within the watershed were built in the 1980s and 1990s. In 1955, a major hurricane produced flooding in the project site as shown in figure CS8–4. This view is looking upstream at Prospect Street. The 1955 flood deposited a significant amount of sediment in this channel and flood plain. The U.S. Army Corps of Engineers (USACE) removed much of this material from the channel. This is a flat flood plain with no buildings and few trees. Note the open fields on the left flood plain and upstream of Prospect Street and the amount of water flowing over Prospect Street and the bridge guide rails still visible.

Figure CS8–5 is a photo taken in 2000 from approximately the same area looking upstream at the Prospect Street Bridge. Note the red barn, house, and numerous trees in the flood plain that were not there in 1955. The left flood plain has another barn and two more houses on the left flood plain not shown. Compared to figure CS8–4, this shows how watershed land use can change. In the late 1950s, a local farm family also built a 4-foot-high dam on the river within the project site to create a swimming area. This small dam had been removed prior to 1999. During Hurricane Floyd, it is surmised damages were high in the project area because the New Britain Water Company had opened the dam gates during the storm to lower levels in the reservoir. If this happened near the peak of the

Table CS8—2 Stream features, Copper Mine Brook EWP project, CT

Station	Stream features
1+00 to 4+50	This section contains a deep pool (2–3 ft) with an adjacent undercut bank (3.5 ft undercut) on the southwest bank. This undercut was formed by Hurricane Floyd. Currently, this undercut and associated pool area provides exceptional fisheries habitat. Two spawning pairs of blacknosed dace were observed at the downstream end of this pool area. The root mats of the vegetation forming the undercut also provide excellent substrate for insect production
4+50 to 10+00	Throughout this reach there are numerous locations of undercut banks on both the east and west sides of the channel. The lateral stability of C4 streams is related to the presence and condition of riparian vegetation. The riparian vegetation, a mix of deciduous trees including, but not limited to maple, black locus, and red twig dogwood, are essential for providing coarse particulate organic material, fine woody material and LWM to the stream system. The current LWM facilitates the maintenance of a deep hole immediately downstream of the LWM. The hole is approximately 2.5 ft deep. The bottom of the pool is obscured by bubble-cover. This is a significant habitat element of this stream reach
10+30 to 12+50	There is a deep pool directly adjacent to a deep undercut bank on the extreme south bank of Copper Mine Brook. The variable topography of the stream bottom, in addition to the pool area and undercut bank, provide the habitat complexity needed for sustainable fisheries
14+00 to 14+75	There is evidence of extensive streambank erosion in this location. The north/northeast bank is vertical and unstable. Currently there is a significant undercut bank at the downstream end of this pool, which provides excellent habitat complexity

Figure CS8–4 1955 flood looking upstream at Prospect St.

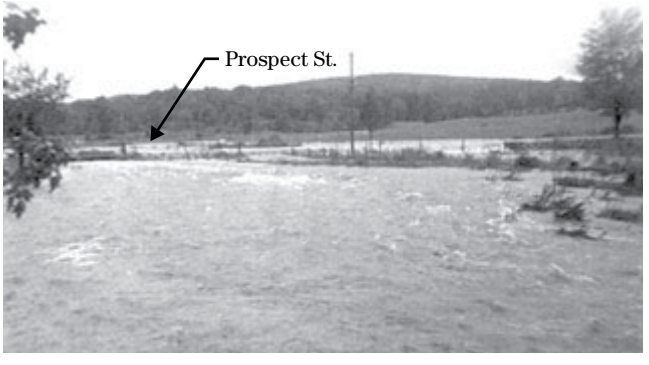
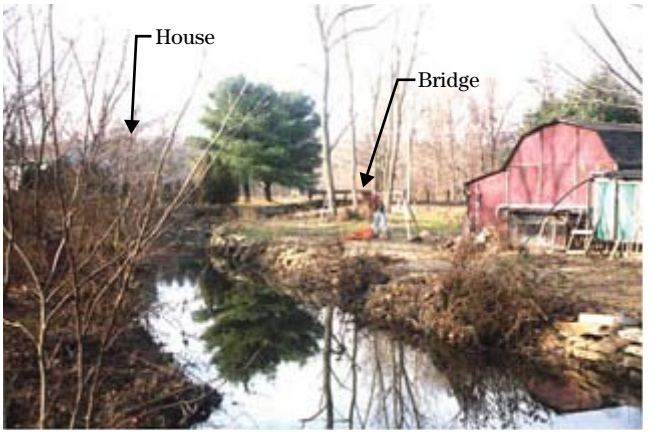


Figure CS8–5 Year 2000 looking upstream at Prospect St.



hydrograph, flows and damages would have likely increased downstream.

In summary, the project site has been affected by the following activities:

- The watershed hydrology was affected by land use changes and flood storage.
- The channel hydraulics were modified by direct excavation and by a small dam within the area of the project site.
- Vegetation along the riverbanks over the years has changed, ranging from farmland to trees to suburban lawn.

Some geomorphic design approaches to natural stream restoration try to allow the channel to reach a dynamically stable equilibrium with the hydrology of the watershed. This is difficult to achieve at Copper Mine Brook because the hydrology is constantly being modified by reservoir releases and land use changes. These changes will probably continue into the future.

Stakeholders and goals

Stakeholders in a stream project are the individuals and groups who either fund the project or are affected by the stream. Ideally, all the stakeholders work together to set goals for the stream design. The stakeholders for this project were the:

- owners of 10 residential riparian properties
- town of Burlington
- city of Bristol
- New Britain Water Company
- Connecticut Department of Environmental Protection, Inland Waters Division
- Connecticut Department of Environmental Protection, Fisheries Division
- NRCS

The goals of this project were set to:

- prevent streambank erosion on 10 residential properties to protect infrastructure
- prevent flooding of 10 residential properties caused by debris in the channel
- protect the town of Burlington's bridge on Prospect Street
- protect the New Britain Water Company's water main
- maintain fish habitat
- maintain water quality

The residential homeowners were predominantly interested in repairing eroded banks and removing debris blocking the channel to protect their yards, drainage pipes, septic systems, retaining walls, barns, and houses. The New Britain Water Company and the city of Bristol wanted the 24-inch water main secured. The town of Burlington did not want a headcut to erode the bridge abutments at Prospect Street. The Connecticut Department of Environmental Protection Inland Waters Division was predominantly concerned with protecting human infrastructure. The Connecticut Department of Environmental Protection Fisheries Division was predominantly interested in maintaining or improving aquatic habitat. The NRCS was focused on achieving the goals of all the stakeholders, maintaining water quality, and doing the job quickly.

In general, the stakeholders' interests produced goals that can be grouped into two main categories. These categories with their corresponding goals are:

- maintain or rehabilitate environmental quality by designing and constructing stream rehabilitation projects that:
  - look natural
  - function naturally with channels connected to flood plains
  - provide desirable stream and riparian habitat including overhanging root cover and LWM
  - maintain water quality
  - are economical to design and build
- protect infrastructure in channels and flood plains by designing and constructing stream rehabilitation projects that:
  - do not increase flooding

- do not migrate across flood plains
- remove trees in jeopardy of falling over
- do not send debris downstream to plug bridges and culverts
- maintain water quality
- are economical to design and build

Sometimes these goals are incompatible, and sometimes they are mutually supportive. Some instances of incompatibilities are:

- Natural streams can migrate across flood plains and can cause trees to fall over. The trees can fall on houses or travel downstream, plugging bridges.
- Woody material can increase flooding, even without plugging bridges.

Some instances of mutually supportive goals are:

- LWM is valuable for aquatic habitat and on some streams can help achieve some channel stability.
- Natural streams with channels connected to flood plains can reduce tractive forces in the channel, thereby increasing channel stability.

In some cases, a compromise needs to be reached between goals for infrastructure protection and aquatic habitat improvement. The following example at Copper Mine Brook illustrates one such compromise. During construction, an overhanging tree root was found to have a cavity extending 8 feet horizontally beneath it into the bank. The adjacent homeowner, fearing the tree could fall on his house or well casing, wanted the cavity filled with boulders. NRCS and the Connecticut Department of Environmental Protection wanted it left open for fish habitat. An optimal solution for infrastructure protection would have been to cut the tree down or fill the cavity with boulders, but this might have been very detrimental to aquatic habitat. After weighing the relative benefits, risks, and consequences, NRCS directed the contractor to partially fill the cavity with boulders and deflect the current somewhat by placing boulders upstream and downstream of the cavity. The downstream boulders were used to protect against back eddies formed by the protruding tree roots. NRCS felt this was a risk worth taking to maintain aquatic habitat. Since the boulders placed in the cavity had gaps between them, numerous small refuge areas were created. It is possible this created better habitat than one single large cavity. When some fish locate themselves by large instream boulders, they will exclude other fish they can see from their side of the boulder. So it is possible that more fish will inhabit a multisegmented cavity where they cannot see each other.

Risks, consequences, and uncertainty

Evaluating risks, consequences, and uncertainty helps designers and stakeholders make decisions on design choices. Risk is the probability of some event happening. Consequence is what happens if the event occurs. Uncertainty describes the level of error in estimates of risk and consequences. Examples of these are:

Risk—There is a 50 percent chance that a 2-year storm will be equaled or exceeded in a year. However, this storm could occur at any time and several times during a 1-year period.

Consequences—If the 2-year storm occurs, the following series of consequences could happen:

- The streambank could erode 5 feet.
- Part of a state highway could slide into the river.
- Motorists could be killed, and highway repairs would be expensive.
- Uncertainty—tools to predict the discharge and velocities from various frequency storms are somewhat accurate and precise. Given a certain frequency storm, present tools to evaluate the certainty of the bank eroding with resultant damages are not that accurate or precise.

The risks and consequences at Copper Mine Brook can be divided into two categories. The first involves infrastructure concerns, and the second involves biological and physical stream processes. The following list describes these categories.

Infrastructure concerns—The uncertainty of the following risks and consequences was moderately high. Regarding risk, it is known that at some time a large event like the 100-year storm will occur, although it is not known exactly when it will occur. Furthermore, existing techniques cannot accurately predict how much damage would be done when it does occur. However, if the brook were left as it was, subjective judgments estimate that future flooding could:

- undermine the Prospect Street Bridge abutments
- undermine a stacked concrete wall (that had fallen twice before)
- flood houses and barns with overland flow and from backwater in foundation drains
- breach a 24-inch-diameter municipal water line
- encroach on septic leach fields and send polluted water downstream
- undermine and topple trees, one of which could fall on a house
- encroach on a well casing
- undermine stormwater outlet headwalls
- create a new channel in other parts of the flood plain

Biological and physical stream processes—If the infrastructure were protected by removing large amounts of debris, removing vegetation, widening channels, and installing large amounts of grouted and ungrouted riprap, the consequence with a high degree of certainty would be that aquatic habitat would be diminished. If the brook were left as it was after Hurricane Floyd, aquatic habitat would be reasonably good. There would be some chance that the brook could create a new channel in the west flood plain.

As-built design

By analyzing risk, consequences, and uncertainty, NRCS produced a design that attempted to fulfill the stakeholders' goals to the greatest extent. Construction was completed in May 2001 (see the as-built design plan views shown in figs. CS8–6 and CS8–7). Each stream rehabilitation measure used throughout the project was evaluated for risk, consequences, and uncertainty. In locations where future erosion or flooding could be tolerated, less armoring or excavation was included in the design. This project placed a greater emphasis on maintaining fish habitat than previous EWP projects. In addition, debris removal, grade control structures, grouted and ungrouted riprap, and bedding stone were used less vigorously than in past EWP jobs. In many cases, bank-placed boulders were used in place of riprap. The rock riprap was sized by tractive force methods. To achieve habitat refuge, the bank-placed boulders needed to be larger than the maximum riprap size so flood currents would not move them. However, there was some concern that fines might pipe out from behind and underneath the bank-placed boulders.

Pre- and postproject photographs, design objectives, and project performance

Figures CS8–8 through CS8–21 show what various parts of the project looked like before and after construction. Figure CS8–8 shows the locations of where photographs were taken. The associated commentary on the captions explains why various techniques were used and how they have performed. Note that the figures use standard streambank nomenclature defining right and left banks and flood plains as looking downstream. For those figures showing a view looking upstream, the right bank appears on the left side of the figure.

At time of this case study documentation (2 years after construction), this project has functioned as follows:

- Some erosion started to occur upstream of the bank-placed boulders near station 2+40, downstream of the stacked concrete wall. The roots above and around the first boulder were not well developed, and there were no tie-back rocks.
- The bank-placed boulders located downstream at station 14+50 to 15+10 are functioning very well. They have very well-developed roots around them and stable riprap upstream.
- So far, no piping of fines has occurred around the bank-placed boulders. The site has not been

subjected to any large floods since construction.

- The LWM at 10+00 that formed a waterfall is gone.
- Throughout the project, fairly good riffles and pools have been maintained within a gravel-armored channel. So far, there is no excessive deposition of stream gravel.
- The riprap is stable.

Conclusion

By describing the work on Copper Mine Brook, the reader should have a better understanding of the Connecticut stream landscape, goals, and design problems. The major design challenges at Copper Mine Brook were to prevent damage to infrastructure in the flood plain and channel and maintain fish habitat. Although the stream and valley types could change, this will be a very common design scenario in Connecticut.

Figure CS8–6 As-built plan view starting at Prospect St.

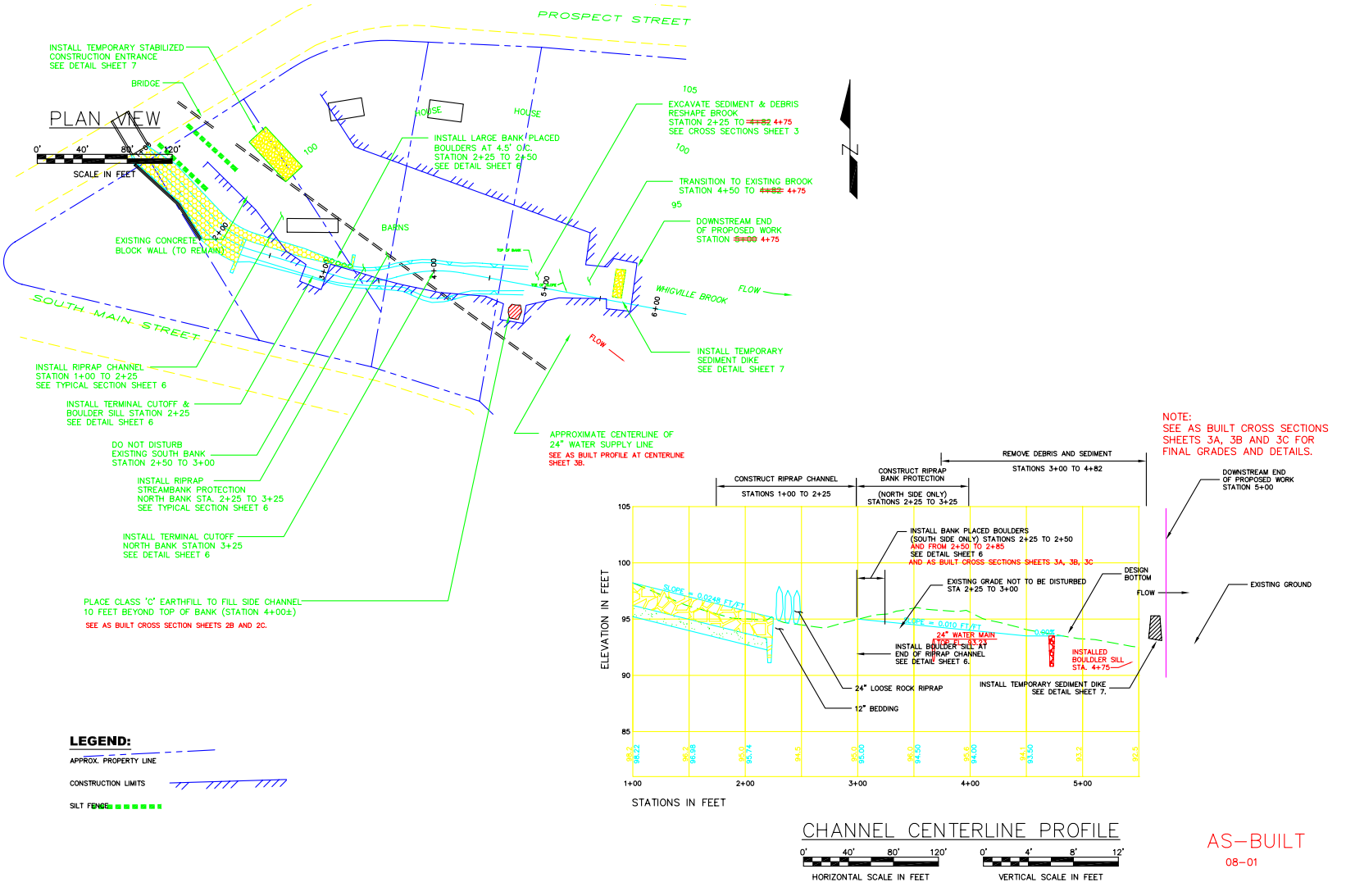


Figure CS8–7 As-built plan view starting 940 feet downstream of Prospect St.

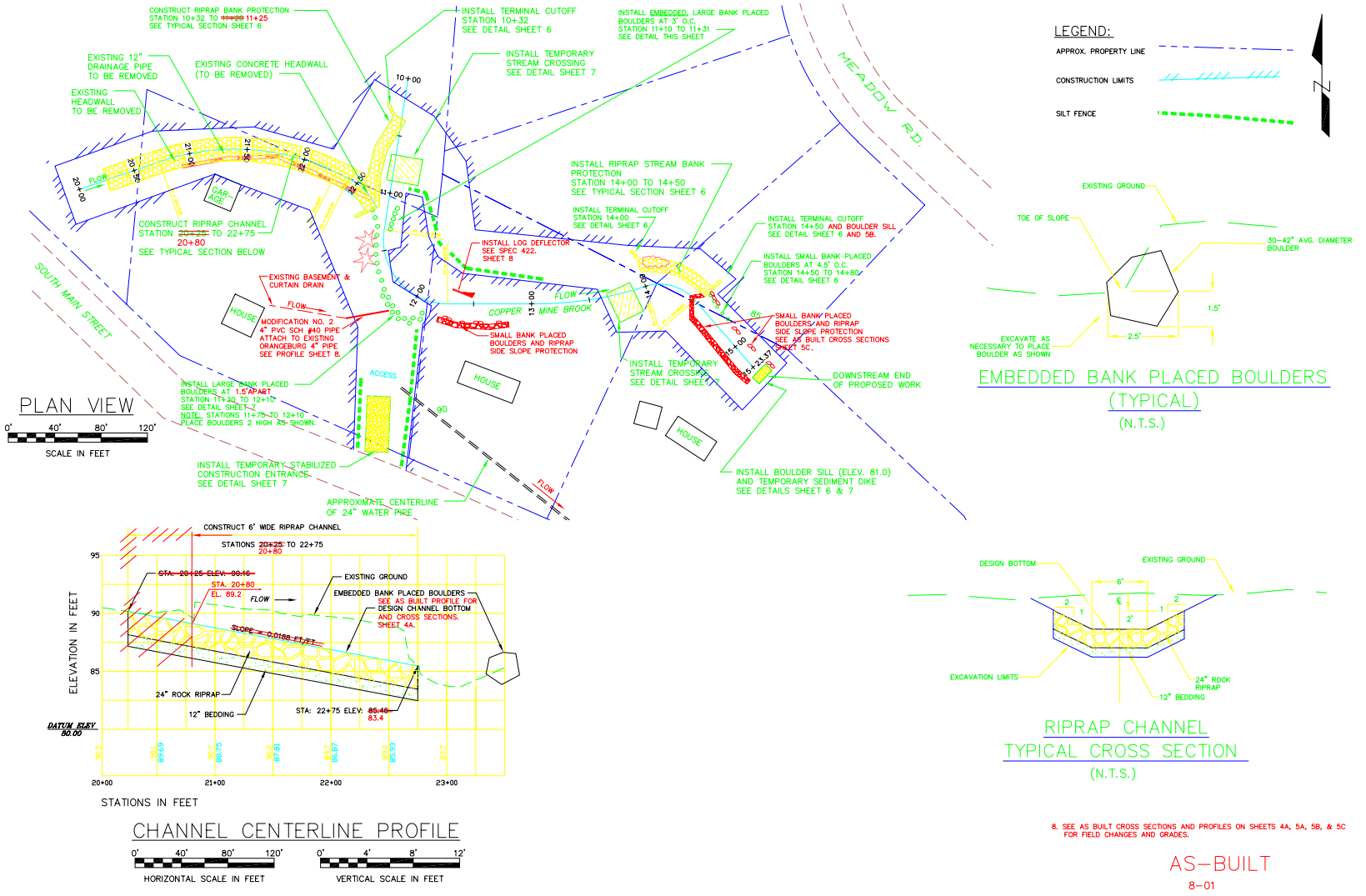


Figure CS8–8 Photo sites orientation aerial map

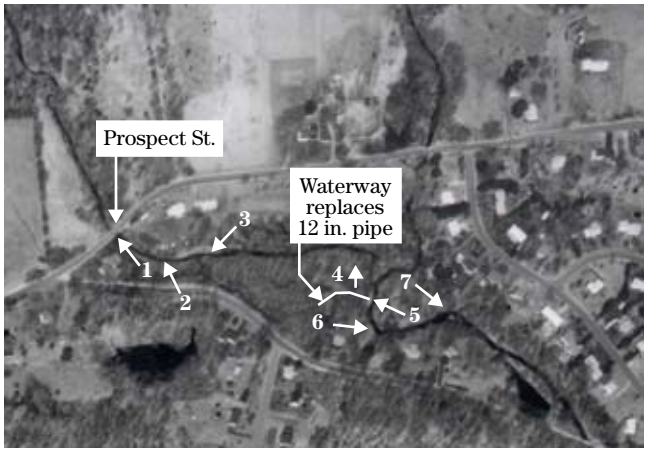


Figure CS8–10 Site 1 before project—looking upstream at Prospect Street Bridge

The stream is incising. Stacked concrete block wall on right bank had fallen down twice before. Some rock on left bank has moved onto the flood plain.

Figure CS8–9 Site 4—looking upstream at LWM waterfall

Site 4 is approximately at station 10+00 and was left unchanged during construction. The woody material created a 2-foot-high waterfall with a 2.5-foot-deep pool downstream. The woody material also helped prevent a headcut from migrating upstream. It decomposed or eroded away within 2 years of Hurricane Floyd. The brook has incised a small amount for the first 50 feet upstream. No major floods have occurred since construction.

Figure CS8–11 Site 1 after project—looking upstream at Prospect Street Bridge

This photo shows the brook 2 years after construction was completed. Consequences of failure here would jeopardize the bridge and the house on the right bank, along with its septic system. The original vertical rock wall on the left bank was removed, and the bank reduced to a 2H:1V slope. The channel and bank were armored with bedding stone and riprap. Bank riprap was topsoiled and seeded to grass. The toes of both side slopes were covered with round native stone, some of which has eroded away.

Figure CS8–12 Site 1 after project—looking downstream from Prospect Street Bridge

This photo shows the site 3 months after construction. The downstream end of the riprap terminates with a buried boulder sill. Rounded native stone was placed on both toes to help define a narrower low-flow channel and make the brook look more natural.

Figure CS8–13 Site 3 before project—looking upstream from station 4+50

Station 3+75 in the foreground has channel gravel and woody material over a 24-inch-diameter water main. The brook had started to carve a new channel on the right flood plain upstream at station 4+00. Existing debris or further aggradation at this site could increase flooding potential and encourage the brook to erode a new channel on the right flood plain. A head cut migrating upstream could expose the water main. The constructed project put a buried boulder sill 105 feet downstream of the pipe and removed a 0 to 2-foot depth of gravel for 175 feet of channel. Gravel removal in a wooded area downstream of this site was reduced to protect aquatic habitat.

Figure CS8–14 Site 5 before project—looking at head-wall at station 11+10

During Hurricane Floyd, when floodwaters started to cut a new channel upstream at station 4+00 on the right flood plain, this downstream garage area experienced increased flooding. This 12-inch-diameter culvert was plugged with sediment when floodwater and sediment entered its inlet located behind the garage at the right of the photo.

Figure CS8–15 Site 5 after project—looking at lined waterway, station 11+10

The pipe and headwall were replaced with this lined waterway entering from the right side of the photo. This was more economical than replacing the pipe and would provide much greater flood protection than a pipe. The upstream debris removal shown in figure CS8–13 had not been significant enough to remove all flooding potential here. Also, the area to the right of the lined waterway would revert to wetland plants, since the owner's access to it would be restricted with less chance of it being mowed.

Figure CS8–16 Site 6 during construction—looking downstream at station 12+00

Hurricane Floyd eroded the bank and cut below the tree on the right. Concerns here were that the tree could fall on the house to the right. Further erosion would encroach on the lawn area and a well casing 15 feet from top of bank. This deep pool, overhanging tree roots, and riffle underneath the excavator were valued for their excellent fishing habitat.

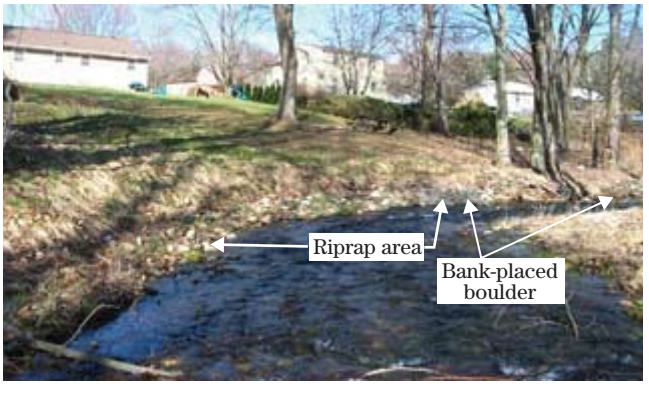
Figure CS8–17 Site 6 after construction—looking downstream at station 12+00

This is the site 2 years after construction. The angular bank-placed boulders are two boulders high into well-graded stream gravel. Lower row of boulders (mostly underwater) has 15-inch gaps for fish refugia. Cavity underneath the tree is only partially filled with boulders, providing more refugia and stability. Riffle downstream of meander was maintained. Fines have not piped out from gaps in bank-placed boulders. The site has not experienced any large floods since construction.

Figure CS8–18 Site 7 during construction—looking downstream at station 14+25

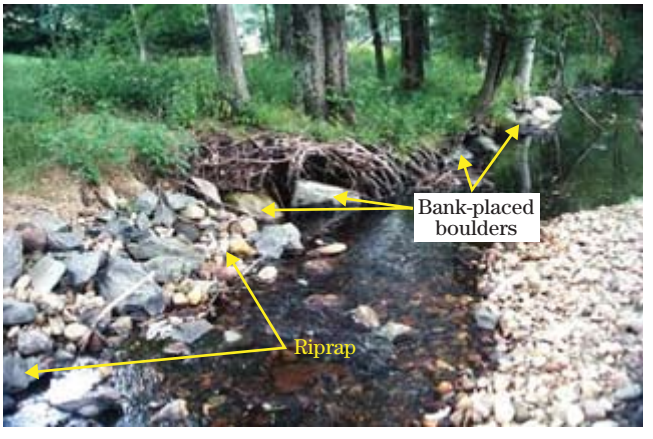
Survey tripod stands on the dewatering berm used during construction. Concerns here were that the brook was encroaching on the left flood plain, eroding into the lawn area and septic system and toppling trees. Area by the tree roots had deep pools and cover for fish, so saving the trees was desirable.

Figure CS8–19 Site 7 after construction—looking downstream at station 14+25



This photo shows site 2 years after construction. The sharp left meander is armored with bedding and riprap and covered with round native stone on the lower bank and topsoil and grass on the upper bank. The trees were protected with bank-placed boulders set into their root cavities, instead of cutting them down and placing riprap on the bank.

Figure CS8–20 Site 7—looking downstream at bank-placed boulders, station 14+50



This photo shows a close-up of bank-placed boulders 3 months after construction. Rounded and angular boulders were placed into the cavities of well-developed overhanging tree roots. Boulders were placed with gaps 1.5 feet apart for fish refugia. The upstream-most boulder is well protected by riprap to prevent erosion behind it.

Figure CS8–21 Site 2—looking downstream at bank-placed boulders, station 2+40

This photo taken 2 years after construction shows three boulders placed on the south (right bank) underneath overhanging roots. Concerns here were that the brook could carve a new channel into the right flood plain. The 5-foot gap between the boulders provides fish refugia. After a few months, bank erosion began to develop behind the boulder on the upstream side. This site is less stable than site 6 because there are no tieback rocks extending from the upstream bank-placed boulder back into the bank, and the root structure of the trees is smaller.

Case Study 9

Little Blue River, Washington County, Kansas

Issued August 2007

Cover photo: Treated section of the Little Blue River, Washington County, Kansas

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 9 Little Blue River, Washington County, Kansas

By Philip G. Balch, Stream Restoration Specialist, and Brock Emmert, Stream Specialist, The Watershed Institute, Topeka, Kansas

Introduction

Sediment is the most common pollutant in streams throughout the United States (U.S. Environmental Protection Agency (EPA) 1998). Streambank erosion is a major source of stream sediment (Simon 2003). As the EPA continues to focus on Total Maximum Daily Loads (TMDL), stream sediment reduction via streambank stabilization and erosion control methods will become a major financial commitment for states trying to comply with sediment standards. A cost-effective solution to streambank erosion must be developed to resolve this water quality problem and restore America's degraded stream corridors to a healthy condition.

The Little Blue River Stream Stabilization and Riparian Corridor Restoration Project is the first such attempt in Kansas to remedy large-scale streambank erosion with limited funds. Other project goals were to:

- reduce excess stream sediment
- improve stream channel dimension, pattern, and profile
- improve aquatic habitat
- establish a riparian ecosystem
- improve terrestrial habitat
- improve water quality
- reduce nutrients and chemical pollutants

The Little Blue River flows through the eastern portion of Washington County, Kansas, and has a drainage basin of approximately 9,065 square kilometers (3,500 mi^2). In late 1999, three landowners along the river contacted the Washington County Conservation District and the District Conservationist with the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) regarding severe streambank erosion on their properties. The district conservationist requested assistance from the Kansas State Conservation Commission (SCC).

During their preliminary site visits, the SCC staff determined that several stream reaches were severely overwidened by excessive bank erosion, and the river had become bed-load driven. Measurements of aerial photographs show a total cropland loss of 149.3 hectares (369 acres) along 12.9 kilometers (8.0 mi) of river between 1977 and 2001. This resulted in a dry weight sediment input of about 11,397,100 metric tons (12,565,300 tons), or approximately 502,600 semi truck loads. Soil analysis showed that nutrient content of the eroded streambank soils equaled 41,845 kilograms (92,270 lb) of nitrate (NO_3), 380,620 kilograms (839,270 lb) of phosphorous (P), and 3,156,400 kilograms (6,959,900 lb) of potassium (K).

Bendway weirs were chosen as the primary structure for stabilization because of their ability to help reduce width/depth ratios, reduce water velocities in the near bank region, induce sediment deposition, and maintain cost effectiveness. Additional project goals included reestablishing a riparian corridor and improving aquatic habitat. In early 2000, the SCC, Kansas Department of Health and Environment (KDHE), and NRCS staff began conducting total station surveys of problem sites.

Initial project surveys, maps, and designs were developed by the SCC staff and reviewed by David Derrick, U.S. Army Corps of Engineers (USACE), Waterways Experiment Station in Vicksburg, Mississippi. The project currently involves 29 project sites on 12.9 kilometers (8.0 mi) of the river. Project construction began in November 2001, and was completed in April 2004. This project stabilized 12.9 kilometers (8.0 mi) of eroding streambanks, established 44.5 hectares (110 acres) of riparian habitat, planted more than 70,000 trees and shrubs, and will reduce 495,520 metric tons (546,320 tons) of sediment to the river annually.

The Little Blue River

The Little Blue River flows through the eastern portion of rural Washington County, Kansas, and has a drainage basin of approximately 9,065 square kilometers (3,500 mi^2). More than half of the river basin is in south-central Nebraska. The bed material is predominantly sand and gravel (.062–64 mm in diameter) (fig. CS9–1).

The bank material composition varies from silts and clays (<0.062 mm in diameter) to sand (0.062–2 mm in diameter). The Little Blue River has not been impounded by large reservoirs and does not contain areas of major levee construction. The river is slightly entrenched. Natural riparian vegetation includes three species of willow (*Salix* spp.), eastern cottonwood (*Populus deltoides* Marsh.), silver maple (*Acer saccharinum* L.), box elder (*Acer negundo* L.), elm (*Ulmus* spp.), burr oak (*Quercus macrocarpa* Michx.), American linden (*Tilia americana* L.), black walnut (*Juglans nigra* L.), hackberry (*Cetlis occidentalis* L.), red mulberry (*Morus rubra* L.), and green ash (*Fraxinus pennsylvanica* Marsh.).

Riparian understory vegetation is dominated by wild ryes (*Elymus* spp.), poison ivy (*Rhus radicans* L.), reed canarygrass (*Phalaris arundinacea* L.), buckbrush (*Symphoricarpos orbiculatus* Moench), and wild gooseberry (*Ribes missouriense* Nutt.).

Most fields along the stream were under cultivation within a few meters of the streambank edge each year (fig. CS9–2). Among the stabilized areas, only site 3 had any permanent riparian vegetation.

Survey and design

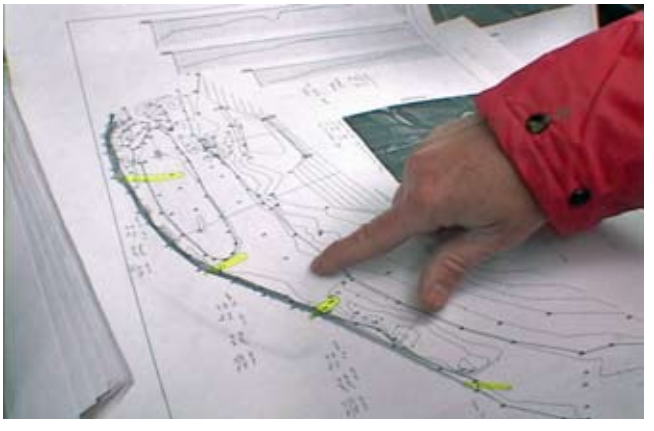
Each site was surveyed by the SCC and NRCS staff with a total station survey instrument. Data points were downloaded into computers, and topographic maps were produced for each site. The maps were then used for measurements and project stabilization design (fig. CS9–3).

Initial site assessments recognized that the Little Blue River had severe bed load problems. Numerous sites contained mid bars, and the stream was extremely

Figure CS9–1 Little Blue River streambed

Figure CS9–2 Site 22, prior to construction

Figure CS9–3 Typical project map and design



shallow. The areas surveyed with water depths greater than 0.5 to 0.75 meter (18–25 in) were upstream of a few isolated, large, woody material piles. The SCC and KDHE staffs designed all projects, choosing bendway weirs for the primary stabilization structure because of the stream's high width/depth ratio (fig. CS9–4).

Bendway weirs redirect water flowing over them, which slows water velocities along the near bank region (Derrick 2001). A weir also moves the thalweg away from the bank to the end of the weir. The design height of all bendway weirs was a third to a half meter (1–1.5 ft) above the water surface at low flow. David Derrick, USACE, reviewed the initial 20 project designs.

The design of redirective techniques, such as bendway weirs, is provided in NEH654 TS14H. Soil bioengineering practices are addressed in NEH654 TS14I.

On sites 8 and 21, the radius of curvature was very low. To keep from pushing the thalweg a great distance from the bank and keep from radically redirecting streamflow, rock vanes were chosen as the stabilization method for these sites (fig. CS9–5).

Project funding

The SCC's Riparian and Wetland Protection Program (RWPP) was originally targeted as the main source of project funding. Increasing numbers of landowners enrolling in the project rapidly grew beyond the RWPP's financial capabilities. Fortunately, KDHE was able to provide $265,000 of EPA Clean Water Act, Section 319 funds to the project. Additional financing came from the SCC's Nonpoint Source Pollution Control Program, the Kansas Governor's Water Quality Initiative, the Kansas Alliance for Wetlands and Streams (KAWS), and the Kansas Chapter of the National Wild Turkey Federation. Combining Federal and state funds provided 100 percent funding for the stabilization portion of the projects. This project required participating landowners to enroll a 30.5-meter-wide (100 ft) strip into the USDA's Continuous Conservation Reserve Program (CCRP). Costs associated with planting and maintenance of the CCRP strip were not included in the construction cost. Tree planting costs for the riparian area between the CCRP strip and the edge of water were included in the construction cost or shared with the Kansas Forest Service's (KFS) Forest Land Enhancement Program (FLEP) and RWPP. Total construction costs for the Little Blue River Stabilization Project and repairs are estimated at $550,000. This equals $42.63 per meter of streambank ($13.02/lf). This figure does not include any cost associated with the CCRP plantings.

Structure installation and revegetation

On early projects, weirs were constructed by excavating ramps into the streambanks, dumping rock on the ramp, and then pushing the rock into the stream with bulldozers (fig. CS9–6).

After the first few projects, rock was dumped directly over the streambank and then moved into place with an excavator (fig. CS9–7).

Following construction of the bendway weirs, the vertical banks were reshaped to a 3H:1V slope (fig. CS9–8). On all sites using bendway weirs for stabilization, the near vertical banks were shaped by pushing them into the river channel (cut and fill method). This accomplished three things: it eliminated the need to key the weirs into the bank, reduced construction costs by reducing required equipment time, and reduced the amount of valuable cropland required for the slope.

After reshaping the vertical banks, winter wheat or oats were sown on the slopes and mulched with native prairie hay. Projects constructed in phases 2 and 3 were sown to wheat or oats but not mulched (fig. CS9–9).

The riparian area between the CCRP strip and edge of water was planted with live willow stakes and bareroot cottonwood seedlings. Live willow stakes were planted on 1.2- by 1.2-meter (4 by 4 ft) spacing. Cottonwood seedlings were planted on 1.8- by 1.8-meter (6 by 6 ft) spacing. In the CCRP strip, trees were planted on 2.4- by 2.4-meter (8 by 8 ft) spacing or 3.0- by 3.7-meter (10 by 12 ft) spacing. All shrubs were planted on 1.8- by 1.8-meter (6 by 6 ft) spacing. Planting a 7.62-meter-wide (25 ft) strip of native grasses and forbs between the shrubs and the cultivated crop field completed the CCRP (fig. CS9–10).

All trees on the slope were planted by hand. Trees on the flat portion of the buffer were planted with farm tractors and tree planters (fig. CS9–11).

Figure CS9–4 Bendway weir detail

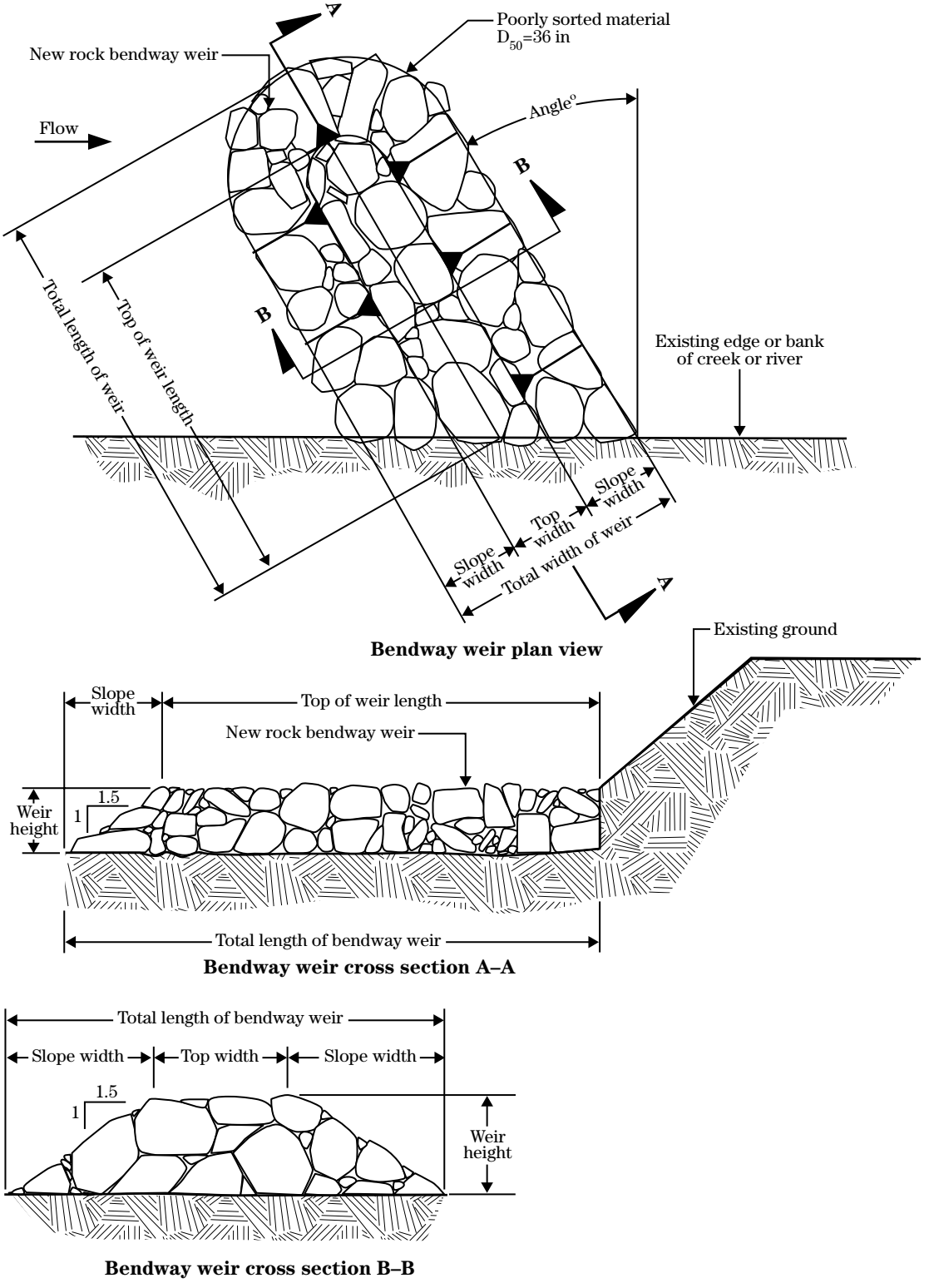


Figure CS9–5 Rock vane design detail

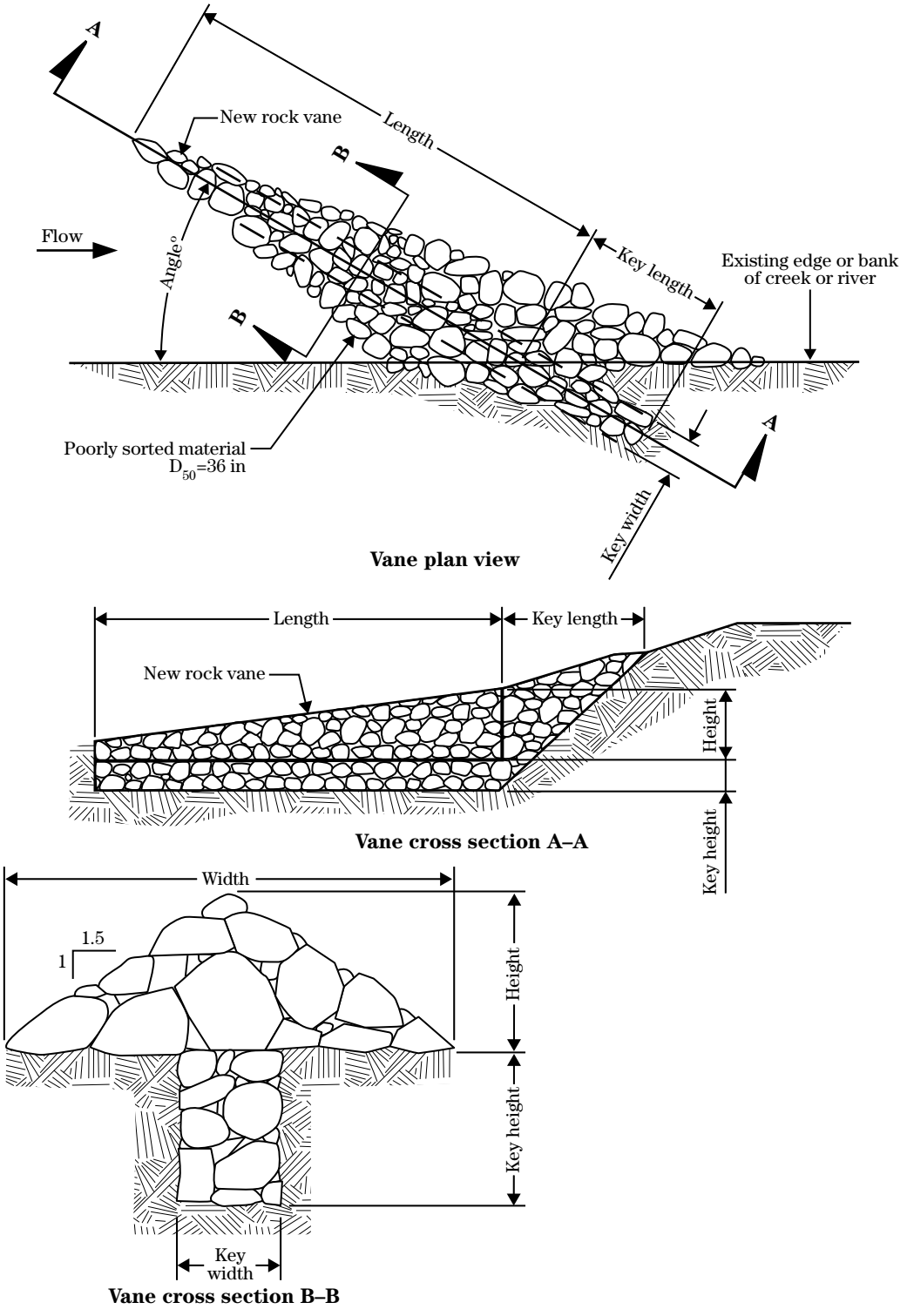


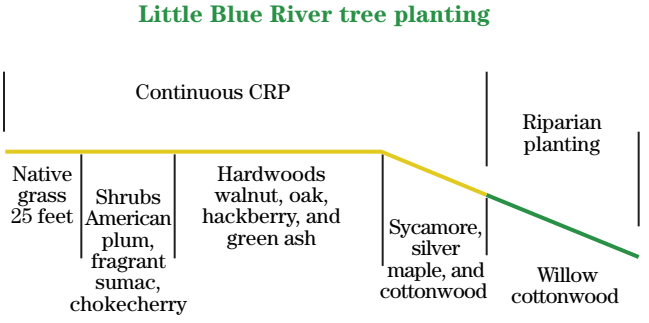
Figure CS9–6 Building weir on site 28

Figure CS9–7 Weir construction on site 22

Figure CS9–8 Bank shaping with bulldozer

Figure CS9–9 Site 1, 04/02/03: Oats beginning to grow

Figure CS9–10 Planting diagram



Agency personnel with NRCS, SCC, and the KFS measured and flagged the tree rows (fig. CS9–12).

Prior to planting, willows were soaked for a minimum of 10 days (fig. CS9–13).

Research has shown that the survival rate for live willow stakes doubles when the stakes are soaked for this amount of time prior to planting (Schaff, Pezeshki, and Shields 2002). Student members of area Future Farmers of America chapters harvested all willow stakes used in the 2002 planting season. Live willow stakes for the 2003 planting season were purchased from the KFS.

In the spring of 2002, landowners, agency personnel, and conservation district personnel planted the trees on 12 sites with volunteer help from several Boy Scouts of America troops. In the 2003 planting season, landowners and agency personnel planted all trees on 12 additional sites. Landowners planted native grasses with a no-till drill provided by the Washington County Conservation District. More than 70,000 trees and shrubs were planted during the springs of 2002, 2003, and 2004.

From drought to flood

The project area experienced a severe drought during the late spring and summer of 2002. Rainfall throughout the project area totaled less than 7 inches during the summer. Because of the drought, trees were replanted on a few sites dominated by sandy soils and south-facing slopes in April 2003.

Projects completed in early 2002 were inundated with two flows that approached the bankfull magnitude in June and September 2002. Another bankfull flow event occurred in early May 2003. Minimal erosion occurred at the stabilized sites during these flows. Slight erosion from the moderate flows required the addition of one structure on two sites. In late June 2003, severe weather and torrential rainfall in south-central Nebraska resulted in substantial flooding along the Little Blue River (fig. CS9–14).

The U.S. Geological Survey (USGS) gage logged the flood flows, which peaked at 1,132.8 cubic meters per

Figure CS9–11 Tree planting on site 11

Figure CS9–12 Measuring and flagging tree rows

Figure CS9–13 Willow cuttings soaking in ponds

second (40,000 ft^3/s) at the Hollenberg, Kansas, stream gage. Downstream, at the USGS gage near Barnes, Kansas, flows peaked at 906.3 cubic meters per second (32,000 ft^3/s). No damage occurred at 20 of the 24 completed sites (fig. CS9–15). Four sites did incur slight damage that was limited to a small portion of each. The problems all appeared on the lower third of the project sites and were corrected in the fall and winter of 2003 by installing Longitudinal Peaked Stone-Toe Protection or additional rock on the weirs.

Sediment deposition occurred on several sites. This was evident between the weirs and on the banks (fig. CS9–16).

Figure CS9–14 Site 1, June 24, 2003, prior to peak flood

Figure CS9–15 Site 1, July 2, 2003, following 1,133 cubic meter (40,000 ft^3/s) flow

Figure CS9–16 Sediment deposition

Research

Various types of research projects were conducted at project sites. One of the first was installation of bank erosion pins. In April 2001, 1.2-meter-long (4 ft) and 1.8-meter-long (6 ft) bank (erosion monitoring) pins were installed at six sites (fig. CS9–17). At another site, severe erosion warranted placing two benchmarks 8.5 meters (28 ft) away from the bank edge.

Five weeks later, an inspection trip discovered all pins lost at the six sites due to streambank erosion. At the other site, only 2.4 meters (8 ft) of the original 8.5 meters (28.3 ft) remained between the bank and one remaining benchmark.

Soil samples were also taken at each site. On most sites, one sample was taken for every meter of bank height (fig. CS9–18).

The Kansas State University (KSU) soils laboratory analyzed all soil samples. The total nutrient input associated with the bank erosion was calculated using the resulting data and soil loss calculations.

Dr. Charles Barden, KSU Research and Extension forester, assisted with tree planting design and also conducted research on various types of tree shelters (fig. CS9–19).

Figure CS9–18 Taking soil samples, site 6

Figure CS9–17 Installing bank erosion pins

Figure CS9–19 Tree shelter test on site 11

Fisheries biologists with Kansas Department of Wildlife and Parks (KDWP) conducted fish sampling sessions at several sites prior to project construction. These sites will be resampled in subsequent sessions to determine any changes in fisheries species composition and biomass (fig. CS9–20).

Similar studies in Mississippi have shown a greater increase in total biomass and species diversity at sites with rock weir type structures than at sites with other stabilization methods (Shields, Knight, and Cooper 2000).

In October 2001, researchers with the USDA Agricultural Research Station (ARS), National Sediment Laboratory in Oxford, Mississippi, conducted research on the root strength and density of various species of willow and eastern cottonwood (fig. CS9–21).

The ARS National Sedimentation Laboratory also conducted soil tension strength analysis on limited sites and is now investigating possible causes for the severe bank instability throughout the river basin.

Three sites were chosen for comparison studies of riparian planting methods. Two sites will look at riparian area natural regeneration. The other will compare direct seeding and nut plantings to sites planted with bare-root tree seedlings.

The KSU Department of Agricultural Economics was enlisted to conduct a socioeconomic study of the project. The results of this study showed the average landowner gained an additional $810 annually from participating in the project. Gains were realized by the value of cropland acres not lost to streambank erosion, income from the acres not lost, and income from the continuous CRP payments. Furthermore, the assessment showed a positive net present value to the landowner for establishing a riparian buffer in CRP and a negative net present value if removing an existing riparian buffer.

Figure CS9–20 Fish sampling on site 2

Figure CS9–21 Root strength testing

Conclusion

The Little Blue River stabilization and riparian corridor establishment project has proven that large-scale streambank stabilization can be constructed in a cost-effective, river-friendly manner. Bendway weirs on sand-bed streams can diversify fisheries habitat and assist in restoring a stable fluvial geomorphology to streams. A cost comparison between bendway weirs and riprap was conducted for site 28. The cost estimate for riprap at this site was $165,000. The actual construction cost to install six bendway weirs, reshape the 1,200 linear feet of streambank, and plant trees was $17,789. This project not only reduced the amount of sediment entering the stream due to bank erosion but also removed excess sediment from the stream during high-flow events as evidenced by sediment deposition in several locations.

The Little Blue River Stabilization and Riparian Corridor Establishment Project has reduced loss of valuable cropland to bank erosion, extended downstream reservoir life, increased wildlife habitat, increased fisheries habitat diversity, and improved water quality. Figure CS9–22 shows the sequence changes of site 1.

Figure CS9–22 Photo sequence of site 1

December 2001

January 2002

April 2002

July 2002

Figure CS9–22 Photo sequence of site 1—Continued

October 2002

June 26, 2003

July 2, 2003

December 5, 2003

March 17, 2004

Case Study 10

Newaukum River, Lewis County, Washington

Issued August 2007

Cover photo: Completed section of the Newaukum River, Lewis County, Washington

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 10 Newaukum River, Lewis County, Washington

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Introduction

Stream restoration and fish habitat have been a concern in the Northwest for many years. Often the practices used to stabilize a stream would have a negative impact on fish habitat, resulting in contentious relationships between implementation and regulatory agencies. In the late 1980s, the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) and Washington Department of Fish and Wildlife (WDFW) agreed to work together to design and install mutually acceptable soil bioengineered methods on a number of sites. The goal of this project was to demonstrate acceptable practices for streambank stabilization and fish habitat. The project was initiated in 1989, and annual reports were prepared for a number of years. Numerous pictures were taken and filed with a narrative record of events for each of the planned 5 years of monitoring and evaluation. In the first 5 years after construction, the sites experienced a variety of hydrologic and climatic/environmental events: two significant floods, a drought, willow borer infestation, beavers harvesting woody vegetation, and a fire. In 2005, these projects were revisited to determine current condition and overall effectiveness of the intended objective. A rationale for success of these projects is also examined in this case study.

Project sites

The initial project started with three sites and later expanded to include monitoring of eight sites, four of which were designed by the Lewis County Soil Conservation District. All sites were on the Newaukum River, historically an important salmon and steelhead-rearing stream, located in Lewis County, in southwest Washington. Table CS10–1 provides the site name, location, and hydrologic characteristics for each site at the time of design. Frequency discharge values based on current data at two U.S. Geological Survey (USGS) gage stations are included for comparison purposes. Since the installation of these projects in 1989, numerous storm events have produced significant discharges.

Table CS10–1 Site hydrologic data (design) and current USGS gage data

Site	Newaukum River location (river mi)	Reach (ft)	D.A. (mi^2)	Channel slope (ft/ft)	2-yr discharge (ft^3/s)	100-yr discharge (ft^3/s)
Nygard	Mainstem (4.3)	450	155	0.0015	5,700	11,800
	USGS gage near site		155		5,780	13,300 (56-yr record)
Olson	Mainstem (9.3)	1,100	143	0.0019	5,400	11,300
Teitzel	North Fork (.85)	500	70.5	0.0021	3,400	7,500
Fitzgerald	North Fork (4.1)	400	49.3	0.0022	2,600	6,000
Burton	South Fork (14.1)	225	63.5	0.0029	3,100	7,000
Hadaller	South Fork (15.1)	500	62.8	0.0027	3,100	6,900
Wesson	South Fork (15.8)	This site removed from study—channel changed course the first season.				
Hirtzel	South Fork (22.2)	400	42.0	0.0046	2,500	5,400
	USGS gage near site		42.4		2,270	4,790 (26-yr record)

The 2-year discharge has been exceeded at least 12 times at the Nygard site, and the 100-year discharge was equaled at least once. These events were the result of widespread storms in the Northwest, so it can be assumed that all sites have experienced similar flows. Table CS10–2 provides the actual date and USGS-recorded discharges for these events.

Initial conditions

Typical site characteristics consisted of:

- a gravel-bed stream with thalweg at the toe of slope
- high, near vertical, raw banks on the outside of curve
- high banks on the outside of a curved reach with seasonal erosion at toe, resulting in periodic upper bank sloughing

Bank soils varied from fine sandy silts with clay to fine sand with lenses of sand and gravel. The banks were devoid of woody vegetation. Raw slopes generally had herbaceous cover by late spring.

The ratios of radius of curve to channel width varied from <2 to 20, a nearly straight reach. The oversteepened bank slopes, combined with saturated soils, resulted in active bank sloughing during and following winter and spring high water events. These sites were not fenced prior to the project and were subject to livestock grazing.

Figure CS10–1 shows typical conditions that existed at the project sites prior to design treatments. Table CS10–3 lists site characteristics, stream classification (Rosgen 1996), treatment measures used at each site, and a summary of current conditions. Descriptions in this case study are generally limited to four of the sites which represent the range of site conditions, techniques used, and experience gained. The four sites selected are the Nygard, Teitzel, Fitzgerald, and Olson.

Design using rock is addressed in NEH654 TS14C and NEH654 TS14K, soil bioengineering techniques are addressed in NEH654 TS14I, and redirective techniques are addressed in NEH654 TS14H.

Table CS10–2 USGS-recorded annual peak flows, Newaukum River, WA

Water year	Date	Peak flow (ft^3/s) Mainstem USGS gage 12 025 000	Peak flow (ft^3/s) South Fork USGS gage 12 024 000
1990	01/09/90	10,400	NA
1991	11/24/90	10,300	NA
1992	NA	3,990	NA
1993	NA	3,730	NA
1994	NA	3,170	NA
1995	12/27/94	6,040	NA
1996	02/08/96	13,300	4,200
1997	12/29/97	9,700	NA
1998	01/14/98	6,580	NA
1999	11/26/98	10,000	NA
	12/27/98	NA	3,240
2000	12/15/99	NA	3,240
	12/16/99	8,100	NA
2001	NA	2,030	715
2002	12/17/01	7,920	NA
	01/25/02	NA	2,140
2003	01/31/03	8,940	2,640
2004	01/29/04	NA	2,740
	01/30/04	7,460	NA
2005	01/18/05	7,740	3,740

NA= Information not available or missing

Figure CS10–1 Typical conditions that existed at the project sites prior to design treatments

Table CS10–3 Site characteristics, stream classification, treatment, and condition

Site	W:D	Ent[1/]	Soils (bank)	Rosgen stream classification	R:W [2/]	Treatment [3/]	Bank stability—2005
Nygard	56.2	1.3	SM	F4	1.6 us 2.6 ds	RB, LS, FX, TR (limited to part of reach)	Stable with established willow canopy
Olson	66.8	2.7	ML SM	C4	11	RR, TR, ECB, BS, F, LS, FX	Stable with points of scour
Teitzel	24.6	5.5	ML CL	C4	2.1	F at top of TR, LS, S, FX	Stable/scour hole above project
Fitzgerald	34.5	4.5	ML SM	C4	20	RB, LS, no TR, S, FX	Stable/significant sediment deposition
Burton	42.6	1.1	ML CL	F4		SB, TR, BS, RW, FX	Stable/braided/rock weir downstream
Hadaller	61.4	3.4	SM	C4		LS, TR, boulders, FX	Stable/complete canopy over stream
Hirtzel	45.7	1.9	ML SM	B4c		RR full bank	Channel shifted away from rock bank; upstream reach unstable

1/ Entrenchment ratio
2/ Bend radius to water surface width (within bank flow)
3/ RB = rock and brush (willow) structures
LS = live stakes
TR = toe rock
F = willow fascine
BS = bank shaping
ECB = erosion control blanket
S = bank seeding
RW = root wad
RR = rock riprap
FX = fenced to exclude livestock

Selected treatments

The Newaukum River is important for fish rearing, so the selected treatment measures were designed not only to stabilize the bank but also to improve habitat. Critical habitat objectives were to reduce sediment, establish tree cover to reduce the water temperature, minimize rock riprap on banks, and remove livestock from direct access to the river. A description and function of treatments installed are shown in table CS10–4.

Table CS10–4 Description and function of treatments installed

Rock and brush structures (RB)	RB structures were 5-ft-wide trenches excavated into the bank perpendicular to streamflow. The trench bottom was excavated into the bed approximately 3 ft and filled with rock sloping up the bank at a 1.5H:1V slope to a predetermined elevation. Above this elevation, the trench was excavated into the bank 5 to 8 ft and filled with layers of rock and brush. The trenches were spaced at 25-ft intervals. Figures CS10–2 and CS10–3 are excerpts from the original design showing layout and detail of these trenches which later became known as RB structures. The design intent of this structure was threefold, (1) establish a hard point on the bank that would control bank cutting during flood events, (2) increase hydraulic roughness, and (3) provide points with dense woody vegetation for habitat enhancement. Some of these structures were installed in conjunction with toe rock and at other locations without toe rock. Installation required excavation at the bank toe and placement of rock under water.
Rock riprap (RR)	Since these projects were intended to improve fish habitat and reduce rock riprap, the use of rock was limited to critical areas for stability and other areas for habitat purposes. The Hirtzel site, initially designed separate from the study with full bank riprap, was installed as designed to serve as a control for comparison purposes. Failure of the upstream reach, as noted in table CS10–3, is not indicative of the design, but rather a result of not being able to treat an active eroding area due to a different property owner.
Toe rock (TR)	Placement of rock at toe of bank slope with the top of rock at or below the bankfull discharge elevation. Toe rock was placed in locations to demonstrate that full bank riprap was not needed when soil bioengineering techniques were installed.
Instream boulders	Selective placement of single, large rocks in the stream. This was intended as a habitat enhancement measure to create deeper pools and protection/resting areas for fish. The large rocks generally were in the range of 4-ft to 6-ft size and were placed 2 to 3 diameters apart near the center of the riverbed in a staggered fashion.
Live staking (LS)	Live cuttings of willow and red leaf dogwood. Cuttings were obtained from nearby sites and were 1 to 2 inches in diameter, 4 ft in length and inserted to a minimum depth of 24 inches. The exposed portion extended above the ground 8 to 12 inches. The stakes were driven to a depth where they would be in moisture during the drier parts of year to assure survival. Live staking provides rapid development of woody vegetation on the sloped banks for increased hydraulic roughness and bank stability and provides future shade for the stream and cover for wildlife (fig. CS10–4).
Fascines (F)	Long willow branches tied together forming a dense, continuous roll 6 to 8 inches in diameter. The roll was placed in a shallow trench and covered with loose soil. Fascines were located near the normal low water line to assure adequate supply of moisture. Live stakes at 4- to 6-ft intervals were used to help anchor the fascine. Fascines provide a dense line of willows along the water line providing shade and fish habitat during low water periods and reduced near-bank velocity during higher flows.
Fencing (FX)	All sites were fenced for livestock exclusion and the buffer area planted with an assortment of woody vegetation including, alder, dogwood, and willow. Livestock exclusion is a proven and necessary bank restoration measure.

Figure CS10–2 Nygard site plan—excerpt from original drawings

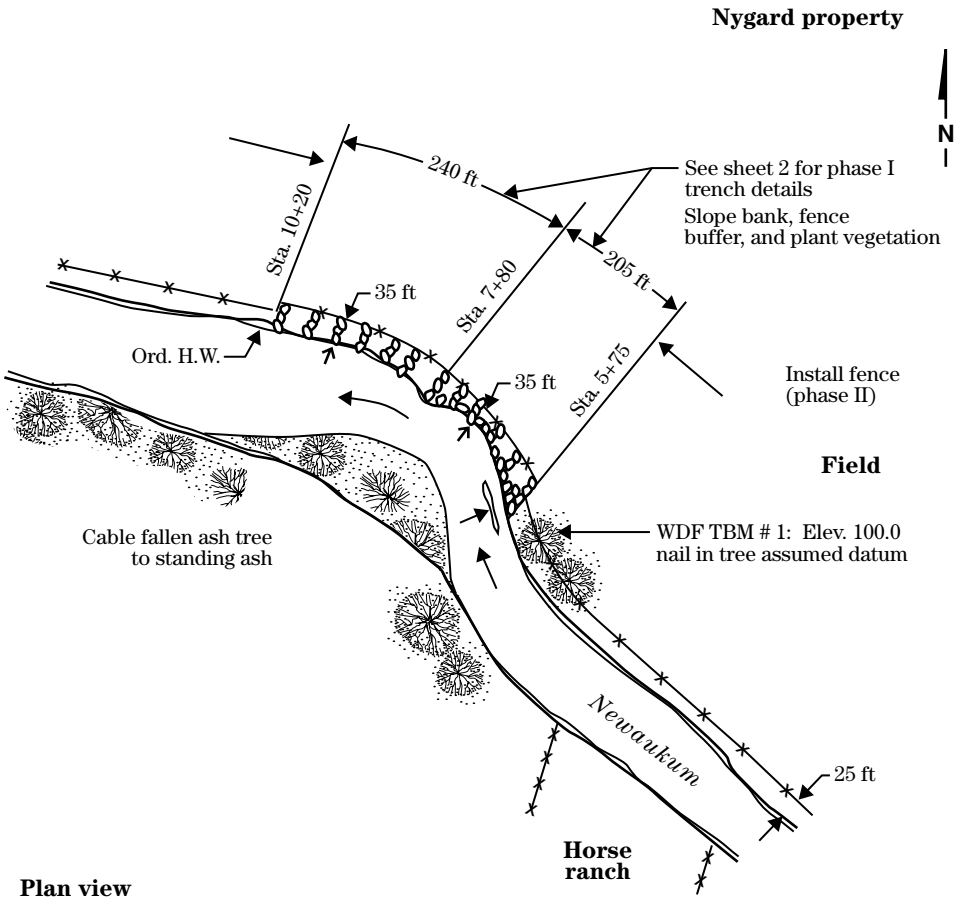


Figure CS10–3 Nygard site, RB detail—excerpt from original drawings

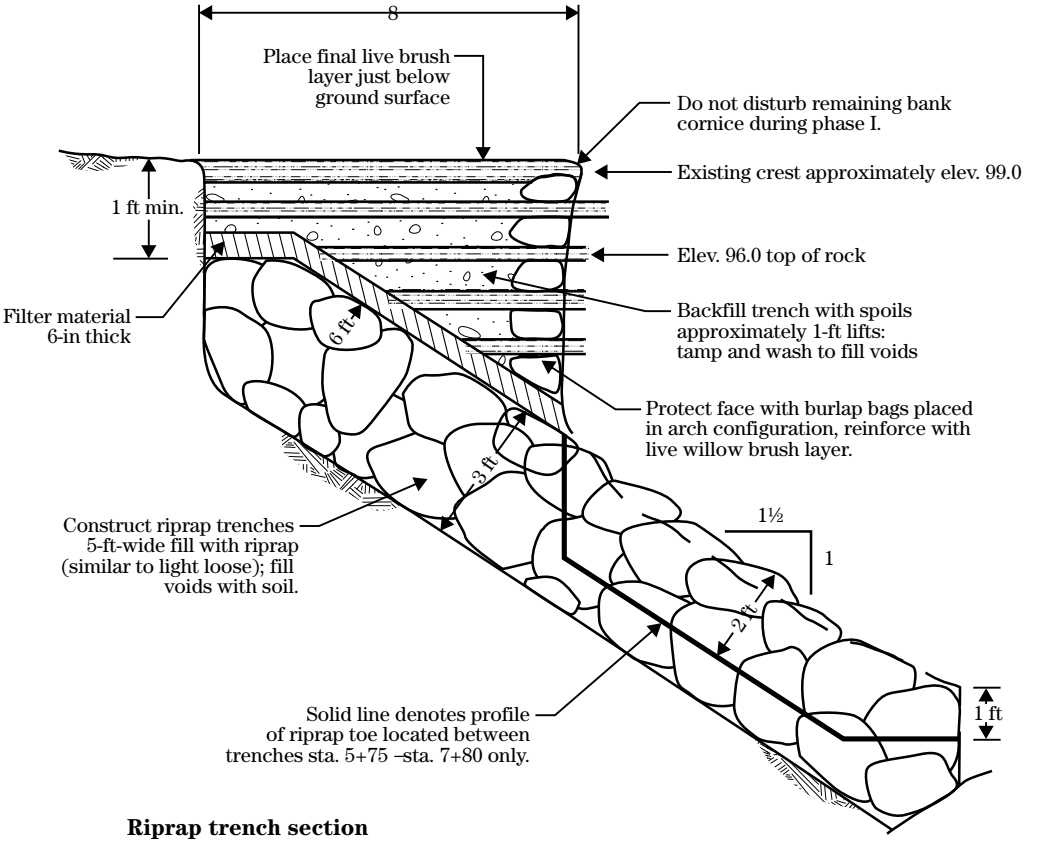


Figure CS10–4 Willow growth at RB structure

(a) Shading effect of willows

(b) Willow development above toe rock. There appears to be some loss of fine soils at the interface.

Description of treatments

The physical characteristics of the four sites described in this study are similar except for the ratios of curve radius to bankfull width (R:W), bank soils, and bank exposure. These characteristics are shown in tables CS10–1 and CS10–3 and further described herein.

The curve ratio has become a common tool in recent years for determining appropriate application of stream redirective-type structures such as stream barbs (refer to NEH654 TS14H). At the time these sites were designed, the ratio of curve radius to channel bottom width (or design surface width) were more commonly associated with design of manmade channels and a means to locate areas and magnitude of maximum stress in bends (USDA Soil Conservation Service (SCS) 1977).

The Teitzel site, with the lowest R:W ratio, was initially treated with narrow (approximately 12-in-wide) rock and brush (RB) structures at 8- to 10-foot intervals. These failed in the first season during an over-bank flood event. Following the flood, the bank was reshaped and treatments applied as indicated in table CS10–4. The live stakes produced an extensive stand of woody bank cover the first year after repair. This was followed by an extremely dry year and infestation of the willow borer, which killed off 80 to 90 percent of the willows. Natural recovery was further decimated by beaver activity in subsequent years. Currently, the bank is stable with heavy grass cover, but no woody vegetation (fig. CS10–5).

The Fitzgerald site is a near linear reach (highest R:W ratio), at the opposite end of the spectrum for curve radius. In the first years of monitoring, the RB structures appeared to be on the verge of failure with

Figure CS10–5 Teitzel site

(a) 1992, aerial view

(b) 1992, after installation of live stakes. Note the fascine just above the toe rock.

(c) 1992

(d) 2005

considerable erosion and scour of loose bank material (sand-gravel lenses) adjacent to the structures. As the willows in the RB structures developed, the roughness element is believed to have slowed the near-bank velocity and allowed for sediment deposition. Currently, the thalweg has moved away from the toe, and significant sediment deposition has occurred along this reach. Woody vegetation is dense at the RB structures and between structures as evident in figure CS10–6.

The Nygard site has a moderate R:W ratio and perhaps is the more consistent of all the sites in steady progress towards stability. It, too, has experienced the flood events and dry seasons, but lower losses (50%) from infestation of the willow borer and not significantly affected by beaver activity. The treatment at this site combines toe rock with the RB structures in the upper curved portion of the reach. Very little bank shaping was done at this site, other than what occurred during excavation of the rock and brush trenches. The steep bank slope was still evident during the site visit in 2005. As figure CS10–7 shows, the water level was above the toe rock. The RBs continue downstream through a shallower bend without toe rock. The thalweg remains at the toe of the bank in the upper curved reach. The willows are dense and extend out over the stream providing heavy shading over the thalweg.

The Olson site was originally installed with no rock riprap and only vegetative treatment for bank stabilization. Before the vegetation became established, it was subjected to the January 1990 flood, which resulted in the failure of this project. The site was then redesigned and constructed with multiple treatments as noted in table CS10–3. During field investigation in the fall of 2005, several points of bank scour were noted. Closer examination revealed that these scour points, some only 10 feet long, coincided with segments of reach without any toe rock. A review of the construction drawings verified that there were gaps in the toe rock. The toe rock was associated with various treatments and varied from rock placed in a single layer to the more typical placement of rock at toe of slope to a designed elevation. The bank has a northern exposure, but development of woody vegetation is somewhat random along the project reach and may be more a reflection of the bank treatments than site conditions.

The relationship between soils and willow condition is shown in table CS10–5. This table was developed from observations of the relative willow condition at each site in December 1998. Generally, the willow stands appear to be more dense and vigorous on the soils that contain less clay and more sand, especially on a south or east-facing bank.

Figure CS10–6 Fitzgerald site, photos taken from similar view points. Initial erosion at toe adjacent to RB; structures filled with sediment as vegetation developed

(a) 1990

(b) 2005

Figure CS10–7 Nygard site

(a) 1 year after installation

(b) Panorama of right bank on outside of curve. The appearance of an inside curve is due to stitching of several photos.

Table CS10–5 Bank exposure and soils relative to willow establishment

Site	Percent clay	Bank orientation	Soil series	Relative willow condition (1998 evaluation)
Teitzel	15–30	South	Chehalis silt loam	Failed
Burton	25–32	East	Chehalis silt loam	Failed (native willow starting to establish)
Fitzgerald	30–33	Southeast	Chehalis silt loam	Poor stand
Wesson	25–30	Northeast	Chehalis silt loam	Fair stand
Olson 1992	7–14	North	Newberg fine sandy loam	Good stand
Olson 1992	15–23	North	Chehalis silt loam	Good stand
Fitzgerald	12–20	Southeast	Newberg fine sandy loam	Very good stand
Nygard	12–15	Southwest	Newberg fine sandy loam	Very good stand
Hirtzel	12–18	Southwest	Cloquato silt loam	Excellent stand
Hadaller	12–18	South	Newberg fine sandy loam	Excellent stand
Hirtzel	15	Southwest	Newberg fine sandy loam	Excellent stand

Rationale for success

These projects have been successful in meeting the original objectives. The objectives were to demonstrate practices that would (1) reduce the use of rock riprap; (2) improve fish and wildlife habitat; (3) provide a stable bank; and, although not originally expressed, provide cost-effective designs. It was readily apparent to the authors during a field investigation in the fall of 2005 that stated objectives were satisfied. Following is a qualitative assessment of the underlying principles believed responsible for the success of these projects in general and, specifically, the four sites identified in the introduction.

It is almost a certainty that the Nygard and Teitzel sites would have failed without toe rock. Visual inspections in the fall of 2005 indicated that the thalweg continued to flow along the toe of bank. Even at low flow, the water depth along the toe was estimated at 4 to 5 feet. Without the rock toe protection, the bank would likely undercut during seasonal high water events with upper bank sloughing as flow subsides. The upper bank sloughing would be slowed because of dense vegetation, but slope failures would likely occur.

The effectiveness of toe rock is further demonstrated at the Olson site where bank scour has occurred at locations where there are small interruptions in the rock toe. Surprisingly, only a single line of rock has protected the toe along segments of this site. This contrasts with the Fitzgerald site where the RB structures were installed without toe rock. Although initial scour occurred adjacent to the RBs, this site now appears to be very stable. The differences in the R:W ratios should be noted. The Fitzgerald site with R:W≈20 and the Olson site with R:W≈8.8 are both relatively linear reaches. The results from these two sites seem to indicate that toe rock may not be necessary for high R:W ratios, other factors being equal.

A stable toe is critical to establish a good stand of woody vegetation on the upper banks. However, as noted in this study, factors such as an insect infestation or other natural events can destroy juvenile plantings. It is important that maintenance be a part of plant establishment. This was demonstrated on the Nygard site after the insect infestation wiped out most of the willows on the upper portion of site. Live staking was repeated to replace willows lost from insect infestation. The results are clearly evident in figure CS10–6, as the willows are now well established. Maintenance is an issue that is often neglected after the initial project installation. When live materials are used, it is important that follow-up care is provided through the establishment period.

Sometimes, even with proper maintenance, establishment of vegetative treatments can be difficult. The Teitzle site was also restaked after the willow borer infestation, but did not recover. Beaver activity and southern exposure at this site exacerbated the situation. In the fall of 2005, the banks were well grassed, but with no woody cover. It appears from table CS10–4 that soils and perhaps soil fertility have played a role at this site. Those sites with Chehalis silt loam soil series and south or east facing slopes did not fair well for establishment of woody vegetation.

When soil bioengineering methods became popular for stream rehabilitation in the 1980s, most engineers remained skeptical. The value of herbaceous cover to provide surface protection and woody vegetation to increase channel roughness are now better understood. The effect of root mass in strengthening soil, however, seemed reasonable, but very difficult to quantify. The major concern for engineers that affected both of these parameters was how to protect the streambank during the establishment period, which could take years. The net result for most engineers was to provide more positive protection measures, usually rock, until vegetation became established.

Initially, rock was placed well up on the bank to protect to a design storm event frequency, say a 10-year or 25-year event. With experience, it was noted that storm events that produced flow depths exceeding the top of rock most often caused little, if any, damage to the bank. Later, the concept of full bank discharge became the recognized elevation of choice for protecting the bank. Currently, this is the accepted practice for many NRCS engineers.

Classical streambed and bank analysis indicates that tractive stress is highest near the toe of the bank. Placing rock at the toe and partially up the bank make good analytical sense. In straight trapezoidal channels where the flow is parallel to the centerline, Lane and others (USDA SCS 1979) have shown that maximum shear stress is near the center of the bed and on the

lower half of the bank side slope. Critical variables in determining the shear stress are depth of flow and velocity. In channels with bends, the flow is no longer parallel and must be considered more in a three-dimensional sense with transverse velocities, impinging flow, and spiral motion. In lieu of complex computations, experience shows that flow in bends moves the thalweg towards the outer bank. So the deeper channel is now at the toe of the side slope. With higher shear stress at the toe of slope, erosion and removal of loose soils can be expected. In locations where the bank material is susceptible to erosion, the weak soils are removed, and the bank is undercut. As the flood event subsides and water level drops, the undercut bank begins to slough due to gravity and excess pore water pressure. Placing rock at these points of higher stress prevents undercutting of the bank and provides opportunity for vegetation to become established.

Conclusion

The foregoing rationale seems simple, but was not readily accepted when these projects were first installed. Some proponents of soil bioengineering methods argued against the use of any rock. Promoters of rock riprap argued that soil bioengineering methods would fail during significant flood events. Fifteen years of experience has shown that soil bioengineering methods can work, with limitations.

Current practice for NRCS in this area is to approach streambank protection with two general strategies. One is to reinforce the bank in such a manner that it can resist forces from the impinging flows using a combination of rock and soil bioengineering measures similar to those used in these projects. The second strategy is to redirect the thalweg away from the bank with instream structures such as stream barbs. When the flow is not impinging on the bank, it is much easier to establish vegetative measures. Redirective structures often require as much rock, or perhaps even more rock, than a riprapped bank, but have distinct advantages for stream restoration. These structures roughen the bed and do not increase velocity as does most bank riprap. Combining these two strategies has worked well and has found favor with most state and Federal regulatory agencies.

Case Study 11

Streambank Stabilization in the Red River Basin, North Dakota

Issued August 2007

Cover photo: Treated section of Cole Creek in the Red River Basin in North Dakota

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 11 Streambank Stabilization in the Red River Basin, North Dakota

By David B. Rush, Environmental Projects Coordinator, Red River Regional Council, Grafton, North Dakota; Frank W. Beaver, Assistant Professor of Geological Engineering, and Jason Warne, Graduate Research Assistant, Department of Geology and Geological Engineering, University of North Dakota, Grand Forks, North Dakota

Introduction

Geologic and geographic features of the Red River Valley in North Dakota, Minnesota, and southern Manitoba create unique geotechnical challenges for slope and streambank stabilization efforts (fig. CS11–1 (University of North Dakota (UND) Energy and Environmental Research Center). The extremely low-gradient landscape (0.2 to 1.5 ft/mi) formed less than 10,000 years ago by the retreat of Glacial Lake Agassiz and is frequently subjected to overland and out-of-bank flooding from the Red River and its tributaries. The relatively young river network of the valley cuts sinuous channels through glacial lacustrine sediments that have been developed into some of the richest agricultural land in the world. Beneath the rivers and agricultural lands are layers of highly plastic clays deposited in the ancient lakebed. The clays shrink and swell in reaction to the region's extreme seasonal climatic swings and are subject to slope failure where they are unconfined along the river meanders.

Figure CS11–1 Boundaries and shaded relief of the Red River Basin

Project planning

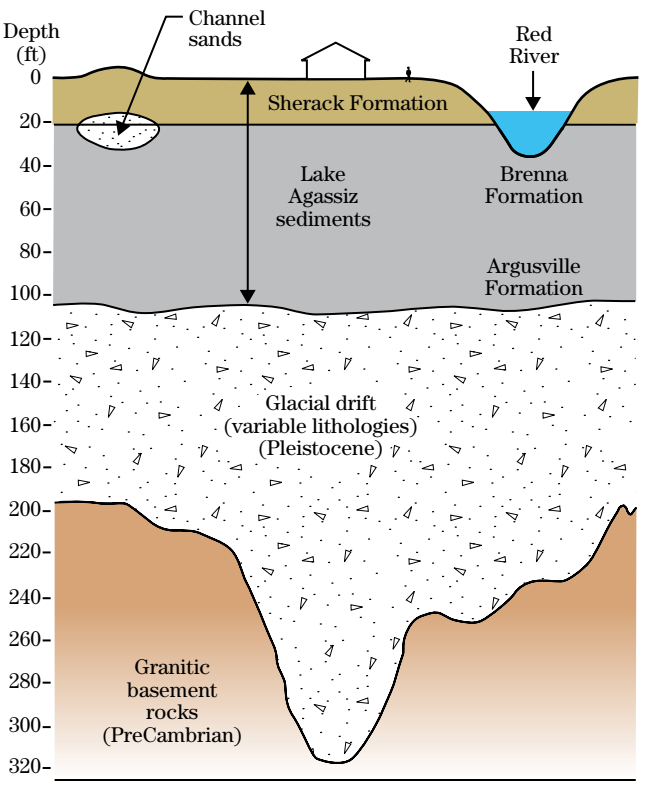
For the past 8 years, the Red River Basin riparian project staff have been working to restore riparian zones and stabilize stream channels and banks in the Red River Valley. The project, funded through the U.S. Environmental Protection Agency's (EPA) Clean Water Act Section 319 program, seeks to improve water quality throughout the watershed. Expertise from a variety of Federal, state, and local agencies is provided to the project through subcontracts and cooperative agreements. Project cooperators include the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), North Dakota Forest Service, North Dakota State Water Commission, University of North Dakota (UND), and local soil conservation districts. The project's involvement in several slope-failure stabilization efforts within the valley has ranged from providing geotechnical information to design and implement engineering and soil bioengineering solutions. The project staff are frequently contacted for assistance with a failed riverbank where the riparian vegetation has been removed or altered and the hydrology has been changed. Although the main goal of the project is to restore a functioning riparian forest to act as a filter between urban or agricultural land use and the river, stabilization of active slope failures is frequently necessary before riparian restoration can be implemented.

Geologic history and geomorphology

Project staff and cooperators recognized at the project's onset that understanding the causes of slope failure was essential to identify riverbank stabilization solutions. The main source of the riverbank instability

is the thick highly plastic smectitic clays and silty-clays formed by eroded Cretaceous shales and glacial tills (Schwert 2003). These suspended sediments were deposited over the glacial lakebed in thicknesses of 100 feet or more. The Sherack Formation (fig. CS11–2 (Schwert 2003)) comprises the upper 20 to 30 feet and consists of light colored, silty, laminated clays that were deposited in shallow water of the glacial lake (Harris, Moran, and Clayton 1974). Beneath this are the dark, highly plastic clays of the Brenna/Argusville formations (fig. CS11–2) that were deposited in deep water during higher lake levels (Harris, Moran, and Clayton 1974). Soil tests of the two formations yield high plasticity index values and liquid limit results and consistencies that range from very soft to rather stiff.

Figure CS11–2 Typical cross section of Red River Valley near Fargo, ND



Hydrologic effects and slope failures

These clay soils cause significant geotechnical problems, especially where they are unconfined. Because of the low-gradient and underfit nature of rivers in the valley, sinuosities are very high (>1.5), creating meanders with a very low radius of curvature. In tight meanders, higher velocity flows are forced against the outside of the channel, eroding the slope toe and steepening the bank. This natural process leads to frequent slope failures along the Red River and its tributaries. The two most common types of slope failure are rotational slumps and flow slumps (fig. CS11–3), with creep and earth flows occurring to a lesser extent.

Although these slope failures naturally occur in the Red River Valley, their frequency and severity has been exacerbated by clearing of riparian vegetation, development of riverside land, and changes in basinwide hydrology. Since the settlement of the Red River Valley in the mid- to late 1800s, the riparian galley forests have been cleared to make use of the rich soils and provide lumber for construction and fuel for steamboats. In recent years, expansion of the valley's urban centers for residential and commercial development has led to the removal of some of the remaining band of forest and native vegetation. Developers and homeowners will often remove or thin the riverbank vegetation for a clearer view or easier access to the river.

Enhanced soil moisture, a critical factor of slope stability, is frequently the result of development. Removal of the moisture-loving native plants and trees, installation of lawn sprinkler systems, and septic drain fields may double or triple the amount of moisture these soils typically receive annually. Local hydrology is altered as homeowners seek to move water away from their houses and, logically, toward the river. However, water from gutters, sumps, and yard drains only helps to saturate the already wet slopes (fig. CS11–4 (UND Energy and Environmental Research Center)).

Regional hydrologic changes may be contributing to the problem, as well. Increased drainage of agricultural land during the last 50 years and record precipitation during the past decade have led to frequent and significant flooding in the valley. The precipitation

Figure CS11–3 Types of slope failures

(a) Rotational failure near Reynolds, ND

(b) Flow slump failure near Wild Rice, ND

Figure CS11–4 Slump flows caused by septic drain fields near Fargo, ND

and flooding, especially during frost-free months, has drastically increased ground water levels and soil moisture. These changes in discharge and flooding have caused river channels to downcut and widen as they adjust to new flow regimes. The combined effect of the factors described above has been a rapid increase in the number of slope failures across the Red River Valley.

Project area: Grand Forks Country Club restoration site

Efforts to stabilize a rotational failure along Cole Creek in Grand Forks County, North Dakota, make a good case study of the difficulties posed by soils and hydrology in the Red River Valley. Project staff and cooperators worked with the Grand Forks Country Club (GFCC) Board of Directors, Club Manager, Golf Course Manager; two engineers; and three contractors to address a 2-acre slump that had damaged one of the club's golf cart bridges over the small creek. Two attempts using a variety of techniques were made during a 3-year period to stabilize the failure, some working better than others. Throughout the process, the goal of the GFCC was to find an effective, relatively inexpensive, and aesthetically pleasing way to stabilize the slope failure and protect a new cart bridge. The project entered the effort with the main goal of influencing riparian management along Cole Creek to improve water quality. Project staff and cooperators felt that the stabilization site provided an opportunity to demonstrate soil bioengineering and riparian restoration techniques to club members and residents of the greater Grand Forks area.

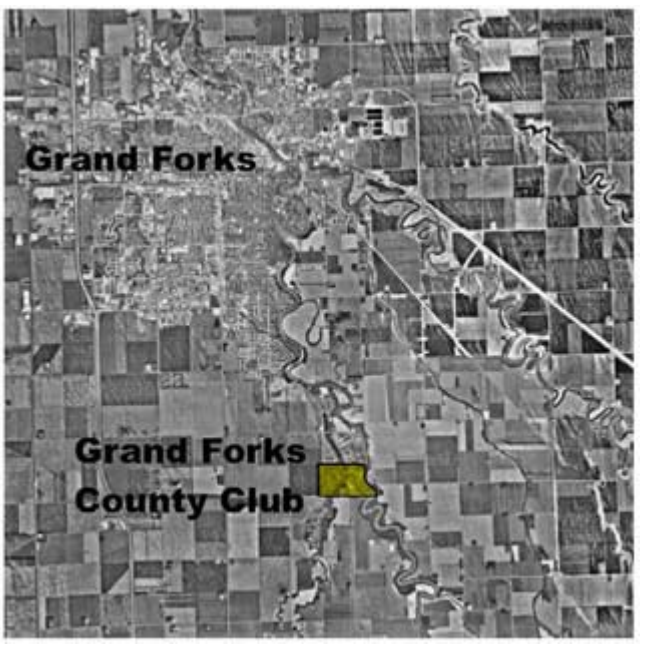
Watershed conditions

GFCC is located approximately 2 miles south of Grand Forks, North Dakota, where Cole Creek confluences with the Red River (fig. CS11–5). Cole Creek is a small stream draining nearly 300 square miles of agricultural land. The stream is intermittent in its headwaters, to perennial at its mouth, flowing regularly in the spring and during summer rainfall events. It is impaired along much of its length by a high sediment load, lack of riparian vegetation, low flows, and extreme summer water temperatures. When the golf course was built in 1963, trees, shrubs, and native vegetation were removed from what was previously a cattle pasture to create fairways and rough. In addition to vegetation and land use changes, Cole Creek has been adjusting to increased discharge from a legal county drain that expanded the watershed by nearly a third. This change in hydrology caused the channel to downcut and become incised throughout most of the GFCC. Wet weather and backwater from Red River flooding in the 1990s saturated the unstable soils adjacent to the entrenched channel and triggered slope failures throughout the course.

Stream problems

Evidence of the slumping that had damaged the golf cart bridge was visible in aerial photographs as early as 1997 (fig. CS11–6). As the 330-foot-long rotational failure settled toward Cole Creek, it unearthed the wooden bridge pilings, narrowed the creek channel, and raised the channel bottom (fig. CS11–7).

Figure CS11–5 Location of GFCC site in proximity to Grand Forks, ND



The narrowing of the channel at this point increased flow velocities, causing accelerated bank and channel scouring downstream (fig. CS11–8). A survey of the slump area showed the channel banks to be as steep as 2H:1V or greater and the slope grade to average 5H:1V (fig. CS11–9). A geotechnical report produced for the design for the new golf cart bridge suggested that the rotational failure could be 100 feet wide and 45 feet deep (CPS 2000).

Figure CS11–6 1997 aerial photo of Grand Forks Country Club slump, marked by arrow

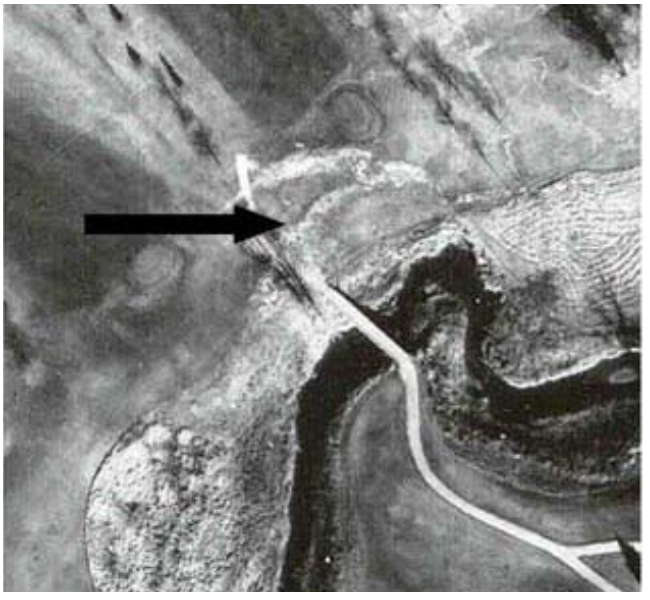
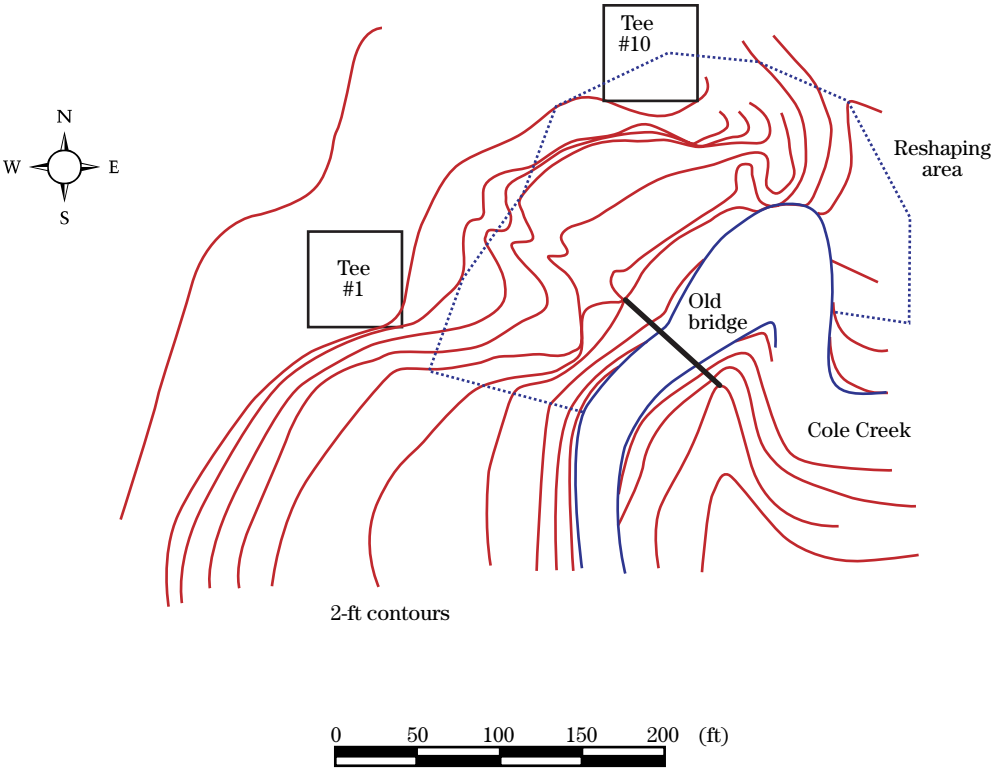


Figure CS11–7 Looking upstream on Cole Creek at the GFCC Cart Bridge (old)

Figure CS11–8 Looking downstream on Cole Creek at the scour erosion caused by the narrowed channel

Figure CS11–9 Topographic map of the slope failure prior to restoration



Planning and design for solutions

Typical efforts to stabilize this type of failure may call for keying significant quantities of rock riprap into the slope toe and channel bottom, balancing the forces causing the rotation. This method has been effective, but can be expensive, not as aesthetically pleasing, and may exacerbate downstream erosion problems. To address these concerns and balance the slump block, the project engineer called for reshaping the existing slope to a 7H:1V or 8H:1V grade and removing an estimated 10,000 cubic yards of soil from the top of the slump, reducing the loading weight. See the grading and restoration plan in figure CS11–10 and also the soil bioengineering and planting plan in figure CS11–11. Toe protection was also an essential aspect of keeping the reshaped slope in place. Rootwads were selected to be installed within a band of rock armor (105 yd^3). It was expected that the rootwads would deflect energy away from the bank and that the rock would add weight and protect the toe during above-bankfull flows.

Moisture management was another factor that was considered as a solution to stop the slumping. Both natural and human sources were supplying water to the slope. Floodwaters, record precipitation during the last decade, and irrigation for the tee box and fairway above the slope combined to saturate the soils, add weight, and lubricate the slickensides in the clay soils. The project sought to solve this issue through irrigation management and the installation of deep-rooted, moisture-wicking vegetation. The plan developed by a North Dakota Forest Service riparian forester called for 2,400 dormant live sandbar willow stakes to be installed over a 7,200-square-foot area (fig. CS11–11).

Figure CS11–10 Design contours for grading and stabilizing slope failure

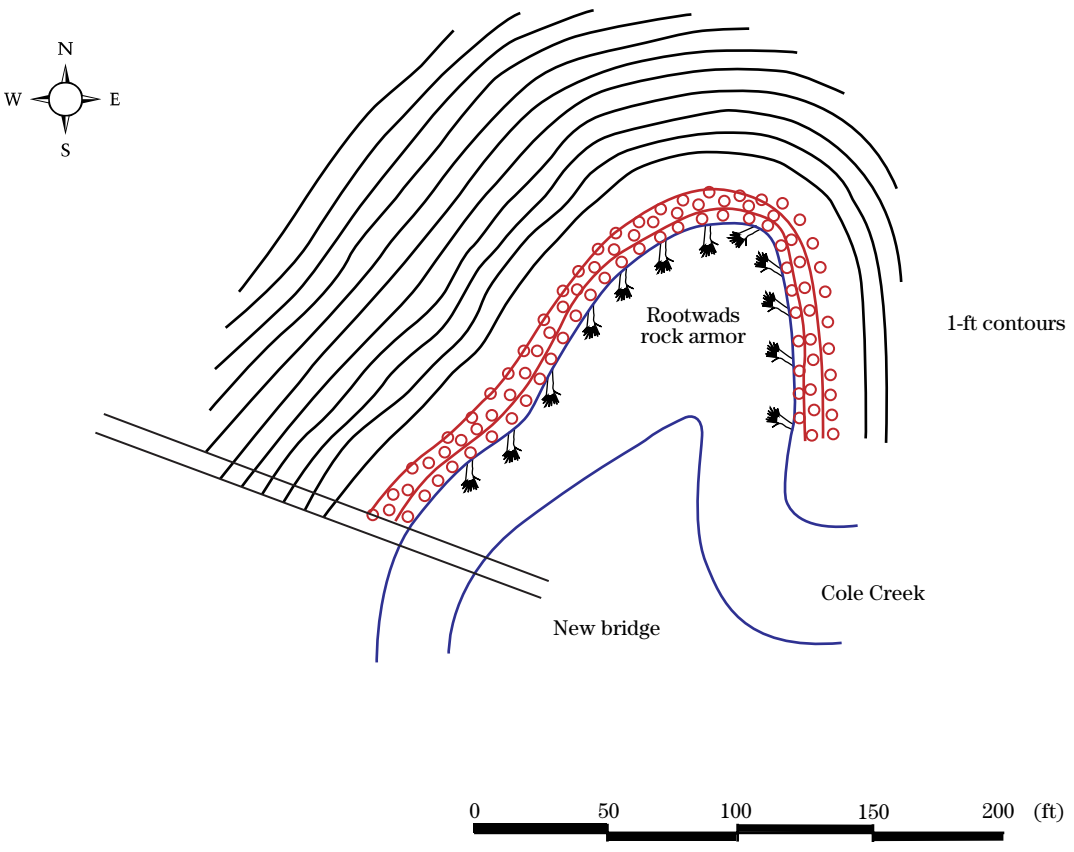
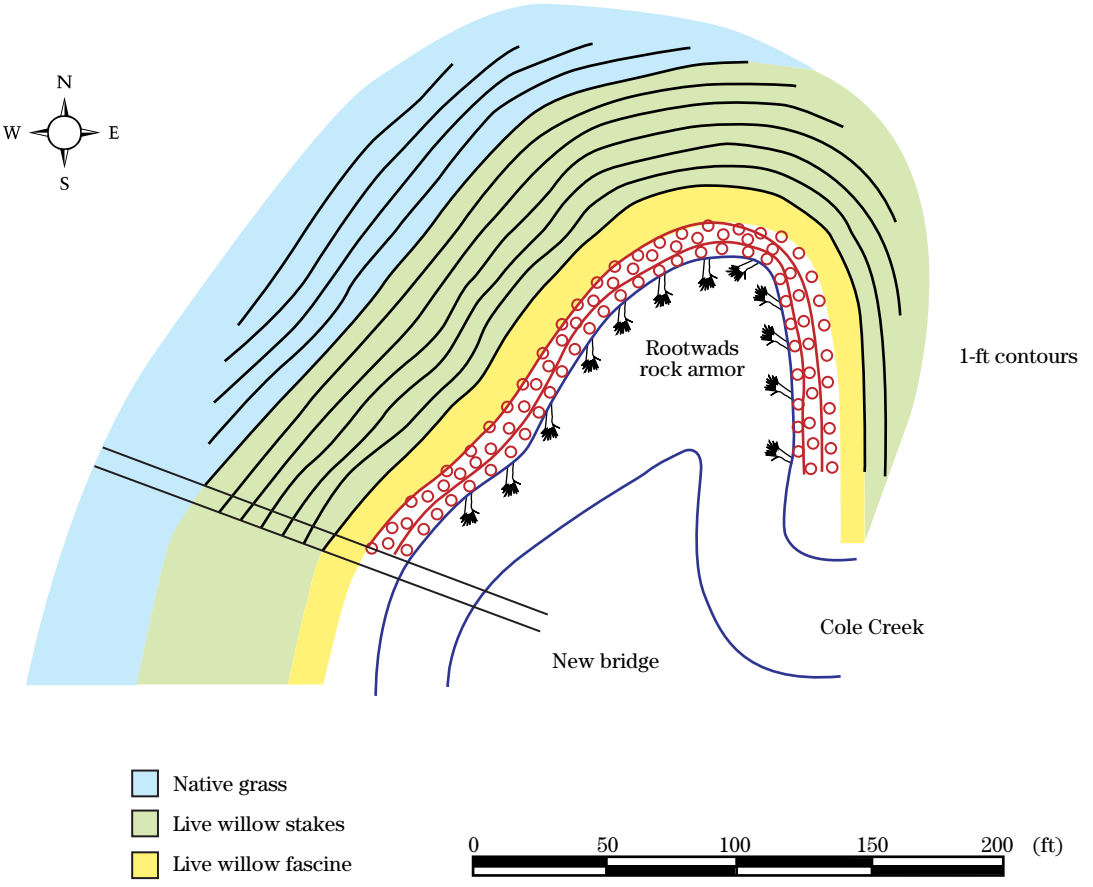


Figure CS11–11 Soil bioengineering and planting plan



Below the willow stakes, 450 feet of live willow fascine would be installed for additional root mass and flow energy deflection from the slope toe. To maintain access to the fairway, the top of the slope was to be planted with a mix of deep-rooted native grasses, including switchgrass, buffalograss, and big and little bluestem grasses. These grasses would take up more soil moisture with their larger and deeper root network and require little to no additional watering.

The design was not immediately accepted when presented to the GFCC Board of Directors. Both the earthmoving and revegetation plans indicated that a portion of a tee box would need to be removed. After negotiations, it was agreed that the tee box could stay, but that the final slope grading below the tee would be closer to 5H:1V and, therefore, would be less stable. It was explained to the GFCC Board that careful irrigation management or ceasing irrigation at the tee would be critical to success of the project, given the steeper final slope.

Restoration solutions: Construction and storm repairs

In March of 2001, the slope reshaping and rock and rootwad installation had been completed during the construction of the new golf cart bridge (fig. CS11–12). The plant materials were installed in June and by August were growing vigorously (fig. CS11–13). Total cost for the work was $33,400, with the excavation accounting for over half of the expense. With the exception of the native grasses being replaced with Kentucky bluegrass and irrigation continuing as before, the stabilization appeared to be holding.

However, by late fall, some minor slumping had occurred adjacent to the bridge where fairway drainage had not been diverted from the slope. The project engineer recommended repairs in the winter of 2002 that included stopping irrigation above the slope, diverting all drainage away, and replacing the Kentucky bluegrass with deep-rooted vegetation such as alfalfa. The project engineer also recommended that rock or fill be placed at the toe of the recent slumping to balance the downward forces and that additional rock be placed along the toe of the entire repaired reach. The weather during summer 2002 prevented the repairs from being completed.

Frequent widespread, severe storms between May and August 2002 brought torrential downpours and summer flooding not seen in the past 50 years. The Cole Creek Watershed was struck by two storms that dumped over 10 inches of rain in each event. The torrents that flushed through the creek were followed by backwater flooding from the Red River. At four times during the summer, the repaired slump was inundated to the bridge deck. Complete sections of the rock toe

Figure CS11–12 Downstream view of Cole Creek, May 2002, prior to planting

Figure CS11–13 Downstream view of Cole Creek, August 2002, after planting

had been washed away or slipped into the channel (fig. CS11–14). Portions of the fascine and entire rootwads had been pulled from the bank. Without toe protection and sufficient counter-balancing weight, the saturated slope began to move, rotating 2 feet and narrowing the channel by 4 feet. The force of the rotation had even bent the bridge piers that were set 80 feet into the clay.

Restoration attempt number 2

Project staff and the project engineer met with the GFCC Board that winter to discuss new alternatives to stabilize the failure. The engineer recommended that the slope be graded to at least 8H:1V and possibly 10H:1V, based on stable grades observed up and downstream of the site. It was also recommended that 325 cubic yards of rock riprap be placed over geotextile along the entire reach (fig. CS11–15). The rock was to have a D_{50} of 10 inches and would be keyed into the toe, extending 12 to 15 feet up the slope. The wedge of rock would protect the toe to bankfull events and above and add counter-balancing weight.

An aggressive revegetation and soil bioengineering plan was also developed for the site (fig. CS11–11). To deflect energy from flood flows, increase the root mass at the toe, and improve the aesthetics, a live wil-

Figure CS11–14 Views of Cole Creek looking downstream from the new cart bridge

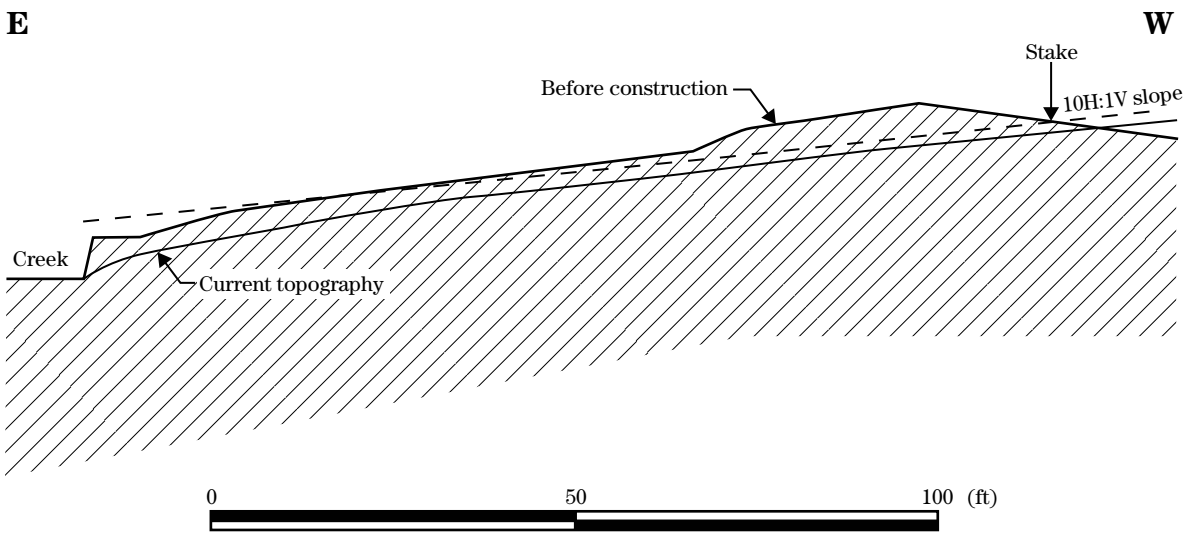
(a) Arrows indicate the same rocks in the stream

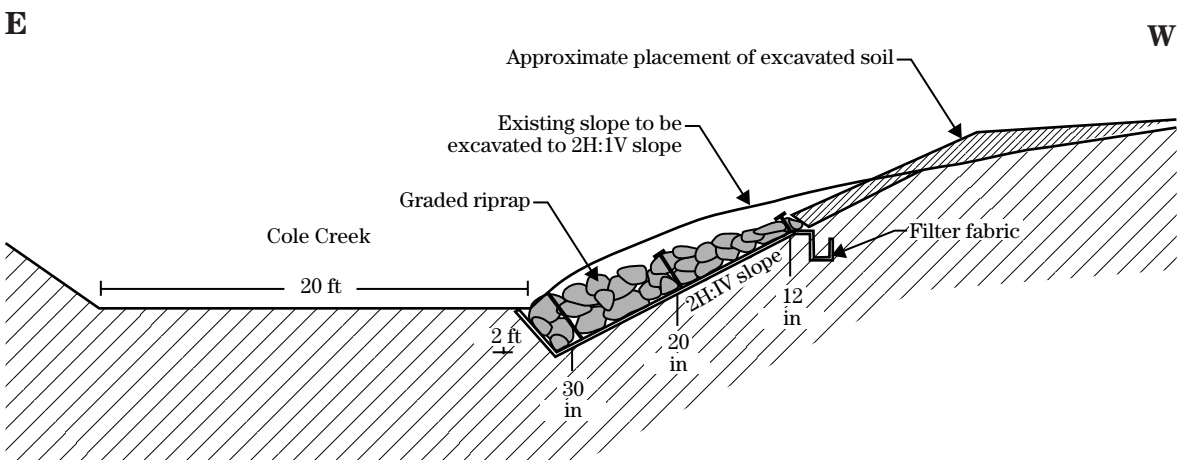
(b) Views of Cole Creek looking back upstream towards the bridge

low brush layer installed within the rock was planned for the section between the bridge and the meander. Excess moisture within the slump was addressed by installing three live willow pole drains among a dense planting of live willow stakes, rooted sandbar willow, and false indigo conservation stock. It was expected that the willow drains would intercept surface and shallow through-flows and direct them away from the slump, while the deep-rooted shrubs and trees would pull moisture from the clays. The plan again called for the upper portions of the slope to be planted with a deep-rooted grass and forb mixture to include prairie cordgrass, Canada wildrye, and switchgrass. All species planned for the site were flood tolerant, given the expectation of future flooding events.

The GFCC Board agreed to the plan, even though it required the removal of a tee box to achieve a 10H:1V slope. In fact, the club carried the plan a step further by planning the installation of a 6-foot-deep French

Figure CS11–15 Cross section of the slope showing the slope grade before construction and during repairs. The planned 10H:1V grade is shown for reference. Note that fill from the excavation will be placed directly above the rock.





drain that would divert surface and subsurface water from the fairway to either side of the slope failure.

Construction began in early June 2003 and was completed within 10 days (figs. CS11–16 through CS11–18). A nearly 10H:1V grade was achieved by removing material from the top of the slope, as well as placing fill above the rock toe. Total cost for the repairs was nearly $26,000, bringing the grand total for the site to $59,400. Table CS11–1 shows a breakdown of the costs. Repairs to the bridge pier had been completed earlier by installing four 16-inch, concrete-filled steel pipes 100 feet into the clay. Construction of the French drain and the native grass seeding was completed during drier weather in August 2003.

Repairs to the site appear to be functioning well; the vegetation is flourishing, channel erosion is limited, and the rotational slope failure is stable (fig. CS11–19).

Figure CS11–16 Aerial view of slope grading and earthwork to place rock toe

Figure CS11–17 Rock toe being placed over geotextile. A layer of live willow branches is visible within the rock toe.

Table CS11–1 Costs of the slope failure repairs by practice

Practice description	Cost
Earthwork for slope grading	$ 5,265
Installation of rock riprap at slope toe	$14,400
Live willow brush packing	$ 1,250
Installation of willow stakes and rooted tree and shrub stock	$ 3,464
Installation of live willow pole drains	$ 1,200
Native grass seeding	$ 240
Total cost	$25,819

Figure CS11–18 Completed stabilization

(a) Looking upstream toward the cart bridge

(b) Looking downstream from the cart bridge

Figure CS11–19 Slope stabilization and soil bioengineering project in May of 2004

(a) Looking downstream into the meander

(b) Looking upstream toward the new golf cart bridge

Case Study 12

Grade Control Structures in Western Iowa Streams

Issued August 2007

Cover photo: Grade control structure on David's Creek in western Iowa

Advisory Note

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Case Study 12 Grade Control in Western Iowa Streams

By John T. Thomas, Hungry Canyons Alliance

This case study summarizes 18 different methods of grade control for western Iowa alluvial streams in deep loess watersheds. Both successful and unsuccessful features of these designs are briefly summarized, along with lessons learned. Pictures of each technique are provided. The goals for each project are clearly stated. This information should be helpful to designers considering similar projects on degrading streams. Design of grade control is described in NEH654 TS14G.

Definitions of project description terms

Funding agency:

- Hungry Canyons Alliance (HCA) provides 80 percent of the total cost to install grade control including construction, design, contracting, and inspection up to $120,000.
- Emergency Watershed Protection (EWP) program projects are administered by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS). The EWP program provides cost share to cover the cost of the construction. The counties provide matching funds in the form of labor for design, contracting, and inspection.

Grade control achieved: amount of stream gradient taken out or controlled by the structure

Total cost: equal to the total cost of construction, design, contracting, and inspection for HCA projects; equal to the cost of construction for EWP projects

Grade control cost: total cost to install grade control, considered to be 100 percent for weir projects, but usually about 20 to 40 percent of the total cost for flume bridge projects

HCA/EWP/NRCS/DOT cost share: total cost share provided to the county/landowner for project completion (NRCS—a cost share program other than EWP, such as EQIP; DOT—Iowa Department of Transportation)

Estimated value of property protected: estimated value of property (bridges, utilities, farmland) protected by a project (which could include construction of multiple structures)

Ecology of reach: little to no fishery (poor stream habitat), questionable fishery (possible stream habitat), probable fishery (good stream habitat)

Stream processes: processes active in channel at time of construction

Stream stage: stage of channel evolution (six-stage Simon model (Simon 1989)) at the time of construction

Design criteria: If a bridge is present upstream, the grade control structure design discharge is equal to that of the bridge design discharge.

Major lessons learned

General notes:

- Each structure must be designed for its unique situation.
- There is a trade-off between design cost and maintenance. Grade control may be accomplished at less expense with a very basic design, but it may require more long-term maintenance.
- Make sure surveys run far enough downstream so that any future streambed degradation that may reach the outlet of the structure may be taken into consideration. If long surveys are not feasible, general information about the stream channel, such as an understanding of channel evolution, bridge inspection reports, and aerial photographs, may be used to estimate future erosion potential.
- The timing of construction is very important in determining the final cost, especially if dewatering of the construction site is required.

- The streambeds of small tributaries near their confluence with larger streams may be prone to fluctuations. The tributary may frequently serve as a backwater for the larger channel, causing aggradation. It will also be prone to channel erosion during infrequent locally heavy runoff events.
- Vertical drops or steep slopes (<4H:1V) will not allow catfish migration. However, shallow slopes (>20H:1V) have shown to allow catfish migration. Catfish cannot swim in flows higher than 2 feet per second (0.61 m/s) or in flows shallower than 1 foot (0.3 m). Research is in progress to determine the steepest weir slope where velocities are less than 2 feet per second, when flow is 1 foot in depth. Following the completion of this research, weirs will be built with slopes shallower than this critical slope.

What works

- Sheet pile is almost always used in HCA and EWP weirs. It is typically pounded into the streambed to a depth of about 20 feet. It represents "the last line of defense," should channel degradation continue or other components of the weir, such as the riprap and concrete, experience instability or failure. It also provides a cutoff wall, preventing flow through the weir and forcing water over the surface of the weir, which is especially important for fish passage.
- Although grout may crack over time, especially near the water line, it still has a longer life expectancy than riprap, considering that loose riprap tends to move around and the quality of quarried rock in western Iowa makes it very susceptible to freeze-thaw weathering. Grouted riprap also helps to force flow over the weir slope when trying to promote fish passage.
- Scour may be prevented by extending bank protection farther downstream than the end of the stilling basin, especially if the outlet of the stilling basin has a different geometry than the rest of the stilling basin. According to an analysis of HCA structure performance, bank sloughing because of scour may be limited if the unprotected banks immediately downstream from the weir have slopes that are 2H:1V or less. Also, if the channel at the weir is wider than the channel at the outlet, rock movement may be decreased and scour inhibited as flow is forced to the center of the channel.
- During construction of weirs, especially when dewatering is required, the channel is typically split into halves or thirds, and work is done on only one part of the stream at a time.
- A central grouted fish passage with baffles appears to allow fish migration. Tagged fish have been able to swim upstream over the structure. Tests of water velocities in the fish passage are in an acceptable range to allow the targeted fish species to migrate. Also, because the channel is divided into three different sections, dewatering is easier during construction or repair.

What does not work

- Barrier rails are no longer used due to their instability under high-flow conditions.
- Concrete blocks are difficult to work with during construction, mainly because of their size, and are prone to undermining by piping.
- The grouted riprap H-pile crib design is prone to leaking and piping.
- Unique energy dissipation concrete forms are difficult for a contractor to form during construction and are also difficult to repair.
- Energy dissipaters are prone to collecting trash, which then focuses water toward the banks.
- Riprap from western Iowa quarries will weather quickly. When placed on a weir slope, riprap will weather and gradually move downstream, eventually creating a vertical face at the weir, a long flat section in the middle, and steep tongues of moved rock at the weir outlet. When designing weirs for fish passage, riprap should be grouted to prevent movement and to minimize maintenance.

Hungry Canyons Alliance (HCA) Structure

Project number: 69–6114–2–1(4, 5)

Type of grade control: Double sheet pile weir

Grade control achieved: 7.7 ft

Other features: 4H:1V riprap downstream slope, 2H:1V riprap side slopes to full bank height, barrier rails cabled together and set in riprap pointing downstream and angled toward the center of the channel

Construction date: November 1993

Total cost: $150,124

HCA cost share: $150,124

Estimated value of property protected: $280,000

Participants: Pottawattamie County, IA; HCA

Stream name: Little Walnut Creek

Soil type: Loess alluvium

Drainage area: 8.05 mi^2

Channel width (top of bank): 80 ft

Channel depth (top of bank): 22 ft

Stream gradient: 16.9 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Limited to no fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Active channel downcutting and widening, active nick point/headcut migration

Stream stage (Simon): 3

Data collected: Topographic survey

Design criteria: Design discharge–Q_{50}–3,100 ft^3/s

Goals: To prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: Barrier rails and loose riprap have experienced significant movement because of high-flow events.

Lessons learned: This project was one of the first funded by the HCA and was the precursor of the basic structure design used today. Sheet pile is almost always used in HCA weirs. It is typically pounded into the streambed to a depth of about 20 ft. It represents the last line of defense should channel degradation continue or other components of the structure, such as the riprap, concrete, etc., experience instability or failure. Barrier rails are no longer used due to their instability under high-flow conditions. Unless it is large riprap, it is no longer placed loose, but is grouted to prevent movement.

Emergency Watershed Protection (EWP)

Project Number: 73–52

Type of grade control: Sheet pile weir

Grade control achieved: 4 ft

Other features: Vertical drop at sheet pile weir, a row of six precast 3- by 3- by 1-ft concrete blocks 1 ft above the rest of the stilling basin, precast 3- by 3- by 1-ft concrete blocks lining bottom of stilling basin; precast 3- by 3- by 1-ft concrete blocks at a 2H:1V slope around the sheet pile weir and on banks to a fifth bank height; 2H:1V loose riprap side slopes to full bank height downstream and half bank height upstream

Construction date: February 1994

Total cost: $246,952

EWP cost share: $246,952

Estimated value of property protected: $400,000

Participants: Page County, IA; EWP

Stream name: West Tarkio Creek

Soil type: Loess alluvium

Drainage area: 46.4 mi^2

Channel width (top of bank): 70 ft

Channel depth (top of bank): 16 ft

Stream gradient: 8 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{100}–9,400 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: The precast concrete blocks have had settling issues on this and many other projects. The vertical sheet pile is bowing in the center and has sprung a leak in the same location.

Lessons learned: Piping undermines some concrete blocks. A vertical drop will not allow fish migration and is now prohibited as a design on most streams by the Iowa Department of Natural Resources.

Emergency Watershed Protection (EWP)

Project Number: 307–07

Type of grade control: Grouted riprap chute

Grade control achieved: 30.8 ft

Other features: Grouted riprap chute with 3 ft channel depth; 4 ft straight drop sheet pile weir at end of chute; precast 3- by 3-ft concrete blocks (1–3 ft in depth) placed in stilling basin; 2H:1V grouted riprap side slopes to a third bank height

Construction date: September 1994

Total cost: $149,898.50

EWP cost share: $119,918.80

DOT cost share: $29,979.70

Estimated value of property protected: $300,000

Participants: Monona County, IA; EWP; DOT

Stream name: Soldier River Tributary

Soil type: Loess alluvium

Drainage area: 0.61 mi^2

Channel width (top of bank): 75 ft

Channel depth (top of bank): 31 ft

Stream gradient: 42 ft/mi

Channelization: Channelization downstream in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Little to no fishery

Site controls: Culvert upstream, agricultural fields on both sides

Stream processes: Active channel downcutting and widening, active nick point/headcut migration

Stream stage (Simon): 3

Data collected: Topographic survey

Design criteria: Design discharge–Q_{100}–800 ft^3/s

Goals: To prevent further streambed degradation with grade control. A 15 to 20-ft headcut had advanced 115 ft upstream in just 3 months toward a 10- by 10-ft RCB culvert under a major state highway.

Goals met: All goals met

Project performance: Water moving downward through the grout has entrained some of the sand bedding underneath the chute, flowed through some weepholes in the sheet pile, and deposited the sand in the stilling basin.

Lessons learned: Site has not experienced further degradation, but rather net aggradation; this tributary serves as a backwater for a major river 600 ft downstream more often than the channel is scoured out by flow coming down the chute.

Emergency Watershed Protection (EWP)

Project Number: 322–01

Type of grade control: Concrete block weir

Grade control achieved: 2 ft

Other features: Precast cubic yard concrete blocks and grouted riprap weir; seven H-piles protruding upward from stilling basin; 2H:1V grouted riprap side slopes to two-thirds bank height (modified to full bank height; loose riprap upstream from weir after new bridge construction)

Construction date: December 1994

Total cost: $132,317

EWP cost share: $109,840

Estimated value of property protected: $270,000

Participants: Audubon County, IA; EWP

Stream name: East Nishnabotna River

Soil type: Loess alluvium

Drainage area: 52 mi^2

Channel width (top of bank): 80 ft

Channel depth (top of bank): 16 ft

Stream gradient: 10.4 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Questionable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{100}–10,000 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: During construction, the concrete blocks had problems with shifting. Downstream from energy dissipaters, a sandbar needed to be removed that had been forcing waterflow toward the opposite bank.

Lessons learned: H-pile energy dissipaters are prone to collecting trash, which then focuses water toward the banks. Concrete blocks were difficult to work with during construction.

Emergency Watershed Protection (EWP)

Project Number: 315–10

Type of grade control: Grouted H-pile crib

Grade control achieved: 5 ft

Other features: Two-step, vertical drop, grouted riprap crib; grouted riprap stilling basin; 2H:1V grouted riprap side slopes to full bank height

Construction date: June 1995

Total cost: $212,646.08

EWP cost share: $212,646.08

Estimated value of property protected: $400,000

Participants: Page County, IA; EWP

Stream name: Tarkio River

Soil type: Loess alluvium

Drainage area: 154 mi^2

Channel width (top of bank): 200 ft

Channel depth (top of bank): 32 ft

Stream gradient: 8.5 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Active channel downcutting and widening, active nick point/headcut migration

Stream stage (Simon): 3

Design criteria: Design discharge–Q_{100}–15,000 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: The lack of a stilling basin has caused scour downstream and continued streambed degradation has undercut the end of the stilling basin. Another structure is planned immediately downstream to resolve this problem.

Lessons learned: The grouted riprap H-pile crib design is prone to leaking and piping. Concrete grout and riprap are prone to freeze-thaw weathering and cracking, especially near the water line.

Hungry Canyons Alliance (HCA) Structure

Project Number: 69–6114–7–6

Type of grade control: Five concrete block weirs

Grade control achieved: 3 ft each

Other features: One row of precast cubic yard concrete blocks laid across channel and cabled together; loose riprap 4H:1V downstream slope; 1.5H:1V loose riprap side slopes to two-thirds bank height

Construction date: July 1998

Total cost: $41,460.87

HCA cost share: $33,168.70

Estimated value of property protected: $524,000

Participants: Montgomery County, IA; HCA

Stream name: West Nodaway River Tributaries

Soil type: Loess alluvium

Drainage area: 2.4 to 9.8 mi^2

Channel width (top of bank): 34 to 78 ft

Channel depth (top of bank): 13.5 to 22 ft

Stream gradient: ~10 to 25 ft/mi

Channelization: Channelization downstream in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Limited to no fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: No hydraulic analysis performed, design based on experience

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: Rock movement has been caused by continued streambed degradation and/or high-flow events. The site furthest downstream, site 5, is experiencing erosion as the stream is trying to go around the blocks. This has been noted on several other concrete block sites, possibly because the blocks were not entrenched far enough into the banks or because of the lack of a cutoff wall.

Lessons learned: Grade control may be accomplished cheaply with this very basic design, but it may require more long-term maintenance.

Hungry Canyons Alliance (HCA) Structure

Project Number: 69–6114–9–6

Type of grade control: Sheet pile weir

Grade control achieved: 4 ft

Other features: 4H:1V grouted riprap downstream slope modified to 20H:1V by adding loose riprap to downstream slope; 1.5H:1V riprap side slopes to full bank height

Construction date: April 2000

Total cost: $75,305.89

HCA cost share: $62,305.89

Estimated value of property protected: $354,333.56

Participants: Montgomery County, IA; HCA

Stream name: Walnut Creek

Soil type: Loess alluvium

Drainage area: 78 mi^2

Channel width (top of bank): 80 ft

Channel depth (top of bank): 18 ft

Stream gradient: 7 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{50}–9,500 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control

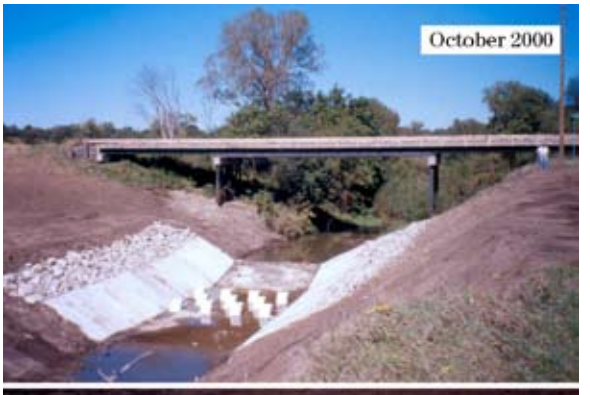
Goals met: All goals met

Project performance: Compared to five other projects, upstream and downstream of this site, that were modified at the same time to 20H:1V downstream slopes by adding loose riprap, there has been little rock movement at this site.

Lessons learned: The design for all six projects was very similar with the only noticeable difference being that at this site; the channel at the weir is wider than the channel at the outlet. In this way, flow was forced to the center of the channel. All six projects are being modified by adding grout to the weir slope to prevent any future rock movement and to maintain the 20H:1V slope.

Hungry Canyons Alliance (HCA) Structure

Project Number: 98–8



Type of grade control: Sheet pile weir

Grade control achieved: 4 ft

Other features: Reinforced concrete 4H:1V downstream slope; reinforced concrete basin with second row of sheet pile at downstream end; 11 reinforced concrete energy dissipaters in basin; 2H:1V reinforced concrete banks to a third bank height and riprap extending to half bank height

Construction date: October 2000

Total cost: $79,978.33

HCA cost share: $63,982.66

Estimated value of property protected: $239,312

Participants: Page County, IA; HCA

Stream name: Buchanan Creek

Soil type: Silty clay

Drainage area: 10 mi^2

Channel width (top of bank): 150 ft

Channel depth (top of bank): 27 ft

Stream gradient: 18 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Questionable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Active channel downcutting and widening, active nick point/headcut migration

Stream stage (Simon): 3

Design criteria: Design discharge–Q_{100}–2,884 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: One of the energy dissipating concrete hooks was damaged after being hit by a tree or ice.

Lessons learned: The energy dissipating concrete “hooks” are difficult for a contractor to form during construction and are also difficult to repair.

Hungry Canyons Alliance (HCA) Structure

Project Number: 69–6114–9–7

Type of grade control: Sheet pile weir

Grade control achieved: 3.4 ft

Other features: Reinforced concrete 4H:1V downstream slope; short (10 ft) reinforced concrete basin with second row of sheet pile at downstream end; 13 steel energy dissipaters in basin; 3H:1V reinforced concrete banks to a third bank height; grouted riprap to two-thirds bank height; and loose riprap extending to five-sixths bank height

Construction date: October 2000

Total cost: $81,810.55

HCA cost share: $65,448.44

Estimated value of property protected: $303,665.20

Participants: Page County, IA; HCA

Stream name: West Tarkio Creek

Soil type: Silty clay

Drainage area: 53 mi^2

Channel width (top of bank): 160 ft

Channel depth (top of bank): 29 ft

Stream gradient: 8 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{100}–10,080 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: The lack of a stilling basin has caused scour downstream. The streambed has continued to degrade, and if the situation continues to worsen, another grade control structure may be built downstream.

Lessons learned: A longer stilling basin would help to control stream energy and reduce scour.

Hungry Canyons Alliance (HCA)|Structure

Project Number: 99–8

Type of grade control: Sheet pile weir

Grade control achieved: 3.5 ft

Other features: 4H:1V grouted riprap downstream slope; grouted internal stilling basin with 1 ft rise at downstream end; 2H:1V riprap side slopes to two-thirds bank height (grouted to one-third bank height)

Construction date: July 2001

Total cost: $71,771.83

HCA cost share: $57,256.95

Estimated value of property protected: $241,820

Participants: Audubon County, IA; HCA

Stream name: David's Creek

Soil type: Sandy clay

Drainage area: 48.2 mi^2

Channel width (top of bank): 80 ft

Channel depth (top of bank): 18 ft

Stream gradient: 6 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{10}–4,805 ft^3/s (based on bridge design discharge)

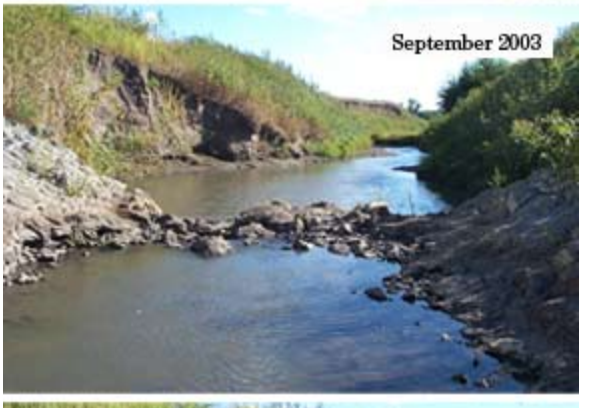
Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control and prevent scour

Goals met: Scour has not been prevented.

Project performance: Even with an internal stilling basin, significant bank scour has occurred downstream because banks were not protected past end of stilling basin.

Lessons learned: Scour may be prevented by extending bank protection farther downstream than the end of the stilling basin, especially if the outlet of the stilling basin has a different geometry than the rest of the stilling basin. According to an analysis of HCA structure performance, bank sloughing because of scour may be limited if the unprotected banks immediately downstream from the structure have slopes that are 2H:1V or less.



Hungry Canyons Alliance (HCA) Structure

Project Number: 69–6114–1–12

Type of grade control: Sheet pile weir

Grade control achieved: 6 ft

Other features: V-notch in sheet pile weir (outside 1 ft higher than center); 4H:1V lightly grouted riprap downstream slope; 2H:1V lightly grouted riprap side slopes to two-thirds bank height

Construction date: October 2001

Total cost: $62,100.93

HCA cost share: $49,680.74

Estimated value of property protected: $136,468

Participants: Monona County, IA; HCA

Stream name: McCleerey Creek Tributary

Soil type: Loess alluvium

Drainage area: 5.1 mi^2

Channel width (top of bank): 95 ft

Channel depth (top of bank): 25 ft

Stream gradient: 13.2 ft/mi

Channelization: Channelization downstream in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Limited to no fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{100}–3,163 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control and protect undermined flume bridge

Goals met: All goals met

Project performance: Performing well

Lessons learned: This is the standard Monona County design for streams with drainage areas of less than 10 mi^2. The Iowa DNR will closely regulate what can be built on streams with drainage areas of greater than 10 mi^2, because of fish migration concerns, and, hence, would not permit a structure design like this because of the steep downstream slope.

Hungry Canyons Alliance (HCA) Structure

Project Number: 99–14

Type of grade control: Sheet pile weir

Grade control achieved: 4 ft

Other features: 5H:1V grouted riprap downstream slope, grouted riprap stilling basin; 2H:1V grouted riprap side slopes to full bank height near weir (loose riprap side slopes up and downstream of weir to either a third or full bank height)

Construction date: October 2001

Total cost: $124,051.88

HCA cost share: $99,241.50

Estimated value of property protected: $254,312

Participants: Crawford County, IA; HCA

Stream name: East Boyer River

Soil type: Silty sand

Drainage area: 131 mi^2

Channel width (top of bank): 210 ft

Channel depth (top of bank): 18 ft

Stream gradient: 8.1 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Questionable fishery

Site controls: 24-in sewer main running across river, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{50}–13,513 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control and protect an important 24-in sewer main, exposed by streambed degradation, for the City of Denison

Goals met: All goals met

Project performance: Performing well

Lessons learned: The structure type must fit the situation. In this situation, absolute stability was needed to protect the sewer main, so the weir was heavily grouted. During construction of grade control structures, especially when dewatering is required because of grout application, the channel is typically split and work is done on only one half of the stream at a time. This structure may need to be modified in the future with a 20H:1V weir slope to allow fish passage.

Hungry Canyons Alliance (HCA) Structure

Project Number: 69–6114–1–06

Type of grade control: Grouted riprap weir

Grade control achieved: 3 ft

Other features: 20H:1V grouted riprap downstream slope; grouted and loose riprap stilling basin; 2H:1V grouted and loose riprap side slopes to a third bank height

Construction date: August 2002

Total cost: $107,060.05

HCA cost share: $85,647.95

Estimated value of property protected: $423,234

Participants: Crawford County, IA; HCA

Stream name: Boyer River

Soil type: Silty sandy clay

Drainage area: 222 mi^2

Channel width (top of bank): 137.5 ft

Channel depth (top of bank): 26 ft

Stream gradient: 4.2 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{100}–14,911 ft^3/s

Data collected: Topographic survey

Goals: To protect bridge pilings and banks, allow for fish migration over the structure, and prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: Performing well

Lessons learned: The structure type must fit the situation. In this situation, a low head dam was needed to restore some stability to the bridge pilings and the surrounding streambanks. Further streambed degradation is unlikely, so sheet pile was not needed. The timing of construction is also very important on streams with large drainages to have a low-cost, finished product.

Hungry Canyons Alliance (HCA) Structure

Project Number: 02–4–F

Type of grade control: Flume bridge replacement

Grade control achieved: 16.5 ft

Other features: Reinforced concrete flume outlet and basin on a new box culvert

Construction date: September 2002

Total cost: $71,422.80

Grade control cost: $16,672.85

HCA cost share: $12,800

Estimated value of property protected: $63,465

Participants: Fremont County, IA; HCA

Stream name: Walnut Creek Tributary

Soil type: Loess alluvium

Drainage area: 0.9 mi^2

Channel width (top of bank): 40 ft

Channel depth (top of bank): 23 ft

Stream gradient: 53 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Limited to no fishery

Site controls: Bridge part of structure, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge Q_{25}–470 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control

Goals met: All goals met

Project performance: Performing well

Lessons learned: Make sure surveys are run far enough downstream to identify and design for any future streambed degradation that may reach the outlet of the structure. If long surveys are not feasible, general information about the stream channel, such as an understanding of channel evolution, bridge inspection reports, and aerial photographs, may be used to estimate future erosion potential.

Hungry Canyons Alliance (HCA) Small Structure

Project Number: SS–00–13–Mike Schomers

Type of grade control: Low-water crossing

Grade control achieved: 3 ft

Other features: 2- to 30-in-diameter corrugated metal pipes on a 20H:1V slope, one with alternating 3-in steel baffles; grouted riprap upstream (2H:1V) and downstream (5H:1V) slopes; sheet pile toewalls at upstream and downstream ends; 20-ft-wide concrete top

Construction date: September 2002

Total cost: $38,156.05

HCA cost share: $8,500

NRCS cost share: $19,078.03

Estimated value of property protected: $150,000

Participants: Landowner; HCA; NRCS

Stream name: Keg Creek

Soil type: Loess alluvium

Drainage area: 30.4 mi^2

Channel width (top of bank): 50 ft

Channel depth (top of bank): 14 ft

Stream gradient: 10.4 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{100}–7,500 ft^3/s

Data collected: Topographic survey

Goals: To provide the landowner access to his property on the other side of the stream, prevent further streambed degradation with grade control, and allow for fish migration through the structure via the culvert w/fish baffles

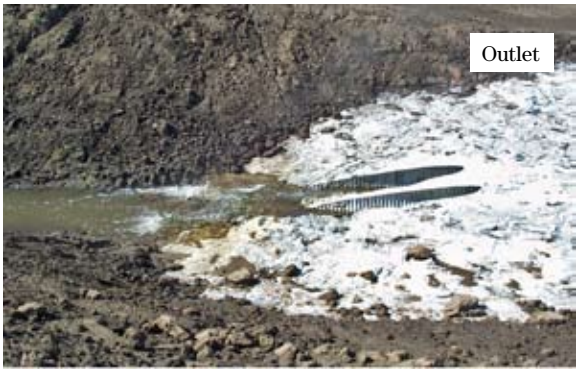
Goals met: All goals met

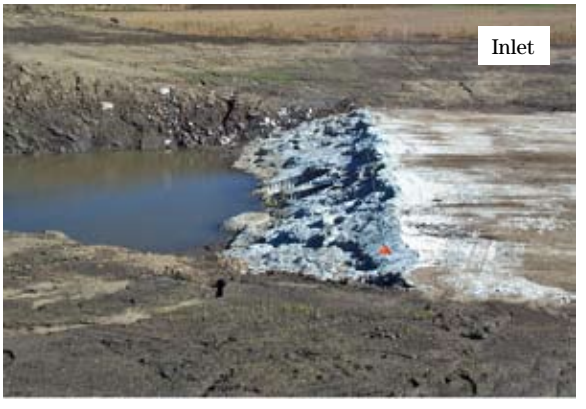
Project performance: Performing well and has been overtopped several times by high flows

Lessons learned: Even though the structure has been overtopped several times, no bed scour has occurred. When the structure is overtopped during high flows, the flow depth may be so deep that scour cannot erode the streambed at the outlet.

September 2002

Up culvert with fish baffles


Outlet


Inlet

Hungry Canyons Alliance (HCA) Structure

Project Number: 69–6114–1–23

Type of grade control: Sheet pile weir

Grade control achieved: 3 ft

Other features: Reinforced sheet pile weir and concrete floor with second row of sheet pile at downstream end; central 20H:1V grouted riprap fish passage with baffles; 2H:1V reinforced concrete side slopes to a third bank height; grouted riprap to half bank height

Construction date: June 2003

Total cost: $73,613.09

HCA cost share: $58,890.47

Estimated value of property protected: $414,234

Participants: Page County, IA; HCA

Stream name: East Tarkio Creek

Soil type: Loess alluvium

Drainage area: 38 mi^2

Channel width (top of bank): 92 ft

Channel depth (top of bank): 23 ft

Stream gradient: 10 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{100}–8,100 ft^3/s

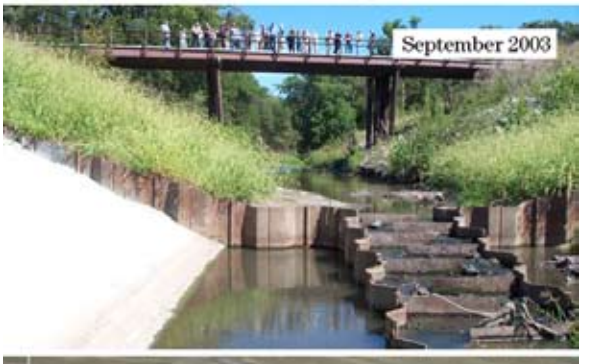
Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control and allow fish to migrate upstream over the structure

Goals met: All goals met

Project performance: Debris is occasionally lodged in the baffles.

Lessons learned: The central grouted fish passage w/baffles appears to allow fish migration. Tagged fish have been able to swim upstream over the structure. Tests of water velocities in the fish passage are in an acceptable range to allow the targeted fish species to migrate. Also, because the channel is divided into three different sections, dewatering is easier during construction or repair.



Hungry Canyons Alliance (HCA) Structure

Project Number: 02–1–F

Type of grade control: Sheet pile weir

Grade control achieved: 4 ft

Other features: 20H:1V riprap downstream slope; cubic yard rocks placed in channel; 2H:1V riprap side slopes to full bank height (grouted to half bank height)

Construction date: September 2003

Total cost: $111,422.98

HCA cost share: $89,121.15

Estimated value of property protected: $332,700

Participants: Audubon County, IA; HCA

Stream name: David's Creek

Soil type: Loess alluvium

Drainage area: 27.96 mi^2

Channel width (top of bank): 100 ft

Channel depth (top of bank): 28 ft

Stream gradient: 5.19 ft/mi

Channelization: Stream channelized in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Questionable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{10}–3,563 ft^3/s (based on bridge design discharge)

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control and allow for fish migration over structure

Goals met: All goals met except cubic yard rocks not placed in riprap, but on top of—could lead to sliding of large rocks during high flows

Project performance: Performing well—the structure has experienced a bank full flow event, and none of the large boulders moved despite resting on an uneven base.

Lessons learned: Be more specific in explaining placement of large rocks to contractors. The downstream riprap slope should be grouted.

Hungry Canyons Alliance (HCA) Structure

Project Number: 02–16

Type of grade control: Sheet pile weir

Grade control achieved: 4 ft

Other features: V-notch sheet pile weir (outside 1 ft higher than center); 20H:1V grouted riprap downstream slope, grouted riprap stilling basin; 2.5H:1V grouted riprap side slopes to one-half bank height; loose riprap side slopes to three-fifths bank height

Construction date: December 2003

Total cost: $98,316.18

HCA cost share: $78,652.94

Estimated value of property protected: $304,312

Participants: Crawford County, IA; HCA

Stream name: Otter Creek

Soil type: Loess alluvium

Drainage area: 30 mi^2

Channel width (top of bank): 77 ft

Channel depth (top of bank): 12 ft

Stream gradient: 6.95 ft/mi

Channelization: Channelization downstream in past and now recovering

Watershed land use: Agricultural rural

Ecology of reach: Probable fishery

Site controls: Bridge upstream, agricultural fields on both sides

Stream processes: Channel widening and slowing streambed degradation

Stream stage (Simon): 4

Design criteria: Design discharge–Q_{50}–6,153 ft^3/s

Data collected: Topographic survey

Goals: To prevent further streambed degradation with grade control while allowing fish to migrate over the structure

Goals met: All goals met

Project performance: Performing well

Lessons learned: Degradation prevention and fish migration required the structure to be built with a sheet pile weir and a 20H:1V downstream slope. Although grout will crack over time, it still has a longer life than riprap. Loose riprap tends to move around, and the rock in western Iowa is very susceptible to freeze-thaw weathering.

Case Study 13

Owl Creek Farms, North Branch of the Kokosing River, Knox County, Ohio

Issued August 2007

Cover photo: Treated section of Owl Creek on Owl Creek Farms, Ohio

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 13 Owl Creek Farms, North Branch of the Kokosing River, Knox County, Ohio

By David Moore, Burgess & Niple, Inc.

Owl Creek Farms was fined in 2000 by the Ohio Environmental Protection Agency for unpermitted stream straightening activities that were constructed in 1997. They were then required to submit a stream mitigation plan, implement the plan, and monitor the site for 5 years. This case study is a short account of these mitigation activities.

Project overview

The Owl Creek Farms project site is on a working farm on the North Branch of the Kokosing River (approximately RM 15.0), in the northwest corner of Knox County, Ohio (near Fredericktown). The watershed is rural, actively agricultural, and in a temperate climatic zone. Mitigation features included buried riprap revetment used on outside meander bends, two instream vortex rock weirs installed in the realigned reach for grade control, and live willow stakings planted along streambanks and on top of point bars. Management support activities include cessation of mowing and pasturing adjacent to stream. Tree seedlings were planted in the riparian area.

The North Branch of the Kokosing River generally supports a high quality aquatic community due largely to ground water supplemented flows and the availability of coarse substrate material. Agricultural runoff and bank erosion are the main sources of sedimentation. The flow is perennial and uncontrolled (no dams or weirs) in this part of the watershed.

Design

The drainage area above the site is about 10 square miles, and the substrate is predominantly gravel. The stream slope is relatively low (0.4%). The original bank condition was fair to poor, with erosion concentrated on the outside meander bends. Little vegetation was present along the banks (<10-ft corridor width). The bankfull channel width at riffles was 20 to 25 feet, and the mean bankfull depth at riffles was about 2 feet.

Mitigation features were selected and designed primarily to meet aquatic community index (Index of Biotic Integrity (IBI) and Invertebrate Community Index (ICI) values), physical habitat potential evaluation (Qualitative Habitat Evaluation Index (QHEI)) scores, rather than hydraulic criteria. The data collected include fish and macroinvertebrate surveys, cross sections, profiles, pebble count data, and riparian vegetation surveys.

No channel reconstruction was undertaken for mitigation. Existing postchannelization alignment was stabilized in place, and vortex rock weirs and plantings were added. No in-depth hydraulic analyses were conducted. Weir locations were determined in the field.

Results

The mitigation plan was implemented in the spring of 2001 by Owl Creek Farms, with costs not quantified. The stream had been channelized and is now stabilized in the current alignment. The overall goals were to mitigate the stream straightening done in 1997 to protect a barn and county road from meander migration. Goals of this mitigation were expressed in terms of IBI and ICI values, QHEI scores, and percent survival and areal coverage values for vegetation. Project goals were optimistic, relative to the preconstruction conditions, but as of the third year of postconstruction monitoring, the goals of this mitigation are being met.

In the third year of monitoring, the project is meeting or exceeding stated goals for fish and macroinvertebrate community quality, habitat quality, and vegetation survival. The vortex rock weirs, bank stabilization measures, live willow plantings, and mowing and grazing restrictions all have performed better than expected. The least successful feature of this project was the survival of riparian tree seedlings. The tree seedlings should have been better protected from competition, or not planted at all, to allow natural succession to occur.

Lessons learned

The vortex rock weirs performed better than anticipated in creating scour pools and attracting fish and macroinvertebrates. Of key importance was the skillful installation of anchor rocks that were well keyed into the bank. The property owner was a skilled equipment operator. The live willow stakings were very successful in stabilizing banks, improving cover, and even providing some shading benefits in the first growing season. Simply ceasing mowing and precluding grazing next to the stream are effective aquatic habitat improvement strategies. Concerns about noxious weeds did not materialize, probably because the area was not actually disturbed, and the existing diverse seedbank successfully outcompeted weeds. Noxious weeds would be more of a concern if the ground were disturbed by construction or other activities. The inherent qualities of the stream, good substrate supply, ground water-supported flow, and a relatively low gradient make it resilient for recovering from impacts. This project has caused a reexamination of the value of tree plantings versus simply allowing natural succession to occur. Figures CS13–1 and CS13–2 show before and after photos of Owl Creek.

Figure CS13–1 Owl Creek—before

Figure CS13–2 Owl Creek—after

Case Study 14

Streambank Stabilization in the Merrimack River Basin, New Hampshire

Issued August 2007

Cover photo: Completed section of Merrimack River in New Hampshire

Advisory Note

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Case Study 14 Streambank Stabilization in the Merrimack River Basin, New Hampshire

By Ghislan Brunet, Maccaferri, Bioengineering and Ecological Systems, Maryland

Problem

The Intervale Country Club (ICC), a 9-hole golf club open to the public, celebrated its 100th anniversary in 2003. The original course was constructed in 1903 by the U.S. Army Corps of Engineers (USACE). Two fairways (6 and 9) abut the river. The course has maintained a cart path along the river bank in the past. Senior club members recall the time when they could have driven the carts on the river side of large pines that now stand at the edge of the bank. Some members of the ICC estimate that 15 to 20 feet of shoreline have been lost over the past 20 years.

The Merrimack River flows 115 miles across central New Hampshire and through eastern Massachusetts where it enters the Atlantic Ocean at Newburyport. The Merrimack River Watershed is more than 5,000 square miles in size.

Over the years, the lake-like section of the river has become both a popular place for building homes and for recreation. Today, boat waves are the main cause for bank destabilization. The waves easily detach exposed soils at the toe of slopes, causing undercutting and mass wasting. The soils are predominately medium to fine sands with very little cohesive strength, once exposed.

Property loss along this reach of the river is due to flooding, ice scouring, and mass wasting of large trees. However, most of the instability problems along this reach are caused by people. Many stabilization methods (concrete walls) have resulted in flows redirected to the downstream neighbor. Many approaches to bank stabilization have been used over the years, as bank erosion has increased due to increased boating activity, as well as development in general.

Solution

Bank stabilization using concrete walls was common prior to 1980. In the 1970s and 1980s, riprap covering the entire height of the slope was a common practice, and in the 1990s, a combination of riprap along the bottom half of the slope and vegetative stabilization along the top was used. ICC used a similar riprap and vegetative stabilization in 1997.

Broad support for the project came from the local, state and Federal levels. Securing project permit requirements was actually ahead of schedule until the site had a positive hit for an endangered species—Bald Eagles. Because of the open water generally associated with this section of the river and sections further south on the river, the Bald Eagle has used this area for winter roosting. The project team coordinated site walks with New Hampshire Fish and Game and contracted with New Hampshire Audubon to provide assistance in how best to construct the project and not interfere with the Bald Eagles. The final solution was that New Hampshire Fish and Game had to approve the cutting of any tree more than 6 inches in diameter on the site. In addition, work could not be done from November 15 to April 15 of the following year.

This decision had two effects on the project. First, to the use of dormant cuttings and brush layering was precluded because the site could not be constructed during the dormant season. Secondly, the limitation on tree cutting increased construction time because the contractor would have to walk the excavator back and forth from the bank to place materials.

The site was divided into two phases. Phase I was the 6th fairway, consisting of 1,000 linear feet of shoreline. Phase II, not yet constructed, is the 9th fairway and is approximately 1,400 feet long.

Design

To preserve the trees on site, a concept of construction from the riverside was evaluated. The riverbed adjacent to the project provided a 12-foot-wide (4 m) shelf before dropping off at a steeper angle. The use of a Portadam® system or water dam barrier was investigated. This option was not utilized, however, as the contractor was able to complete the project from the top of the bank with selective tree cutting.

Preliminary design analysis provided three distinct facts for slope stability along this stretch of the river:

- The angle of repose of the natural slopes could be very steep and still be stable. Visual evidence showed slopes of 1H:1V and steeper as being stable provided that the toe of the slope was not undercut. The parameter for the stability analysis was limited to site observation. The native soil used in the analysis was a silty sand.
- The main cause of the toe scour was boat waves. Even at high water elevation, water moved by the site at a fairly slow velocity.

  Although ice damage is not a major concern for this section of the river, significant ice has formed along the river in the past.
- The river has a normal flow between 3,000 and 7,000 cubic feet per second. Low flow is less than 1,000 cubic feet per second, while flood stage occurs at greater than 31,000 cubic feet per second. The historic recorded discharge of 150,000 cubic feet per second for the river occurred during a 1936 hurricane.

Design stability analysis

Analysis was performed to verify the bank's slope stability and erosion resistance. The slope stability analysis was conducted using MacStars 2000™, a software package using the Bishop Simplified Method (Bishop 1955) to determine the global and internal stability. The slope stability analysis determined the impact of the root system after installation and establishment of vegetation.

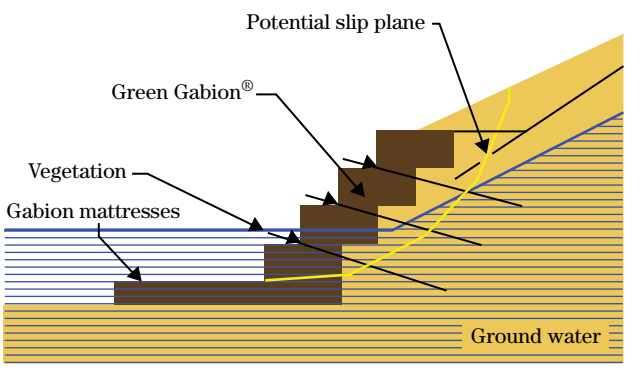
The slope stability analysis, done immediately after installation, resulted in a safety factor of 1.55 for global stability and 4.02 for internal stability (fig. CS14–1). The impact of the vegetation on slope stability was determined based on measured shear resistance of willow roots. The shear resistance of a 3-year-old willow can have an ultimate shear stress of 9.1 kilonewtons per square meter (kN/m^2) (191 lb/ft^2) (Goldsmith 1996). The reinforcing root length was considered to be 2 meters (6 ft) deep. To represent the root reinforcement in the structure, the density of the vegetation was considered at 30 centimeters (1 ft) spacing between each plant for each row of units.

The analysis of the impact of vegetation on slope stability was performed using the shear resistance of a willow after 3 years of growth. A reduction factor of 3 was applied to the ultimate strength to represent the uncertainties of the growing vegetation. A shear resistance of 3.0 kilonewtons per square meter (63 lb/ft^2) was used in the calculation. With the contribution of the vegetation, the internal stability of the structure (fig. CS14–1) gave a safety factor of 4.55 and a global safety factor of 1.55. The internal stability of the slope may increase 10 percent after the establishment of the vegetation. This last analysis was considered as information only and not for the design stability analysis. The global stability was not affected because the potential slip plane did not pass through the vegetation.

The material's erosion resistance was verified using MACRA 1™ software by Maccaferri. The analysis was performed to verify the resistance of erosion immediately after installation of the structure and after 3 years of vegetation establishment. The gabion revetment has a shear resistance of 336 newtons per square meter (N/m^2) (7.0 lb/ft^2) without vegetation and 450 newtons per square meter (9.4 lb/ft^2) with vegetation established.

The safety factor for erosion was calculated to be 9.9. Wave action from boats and potential ice gouging were not considered due to the limited budget for design analysis. Maccaferri literature shows that a gabion revetment can resist wave action up to 1.7 meters high (5.5 ft) on a slope of 1H:1.5V. Wave height due to boat traffic on the Merrimack River is estimated at 1 meter

Figure CS14–1 Slope stability analysis of design

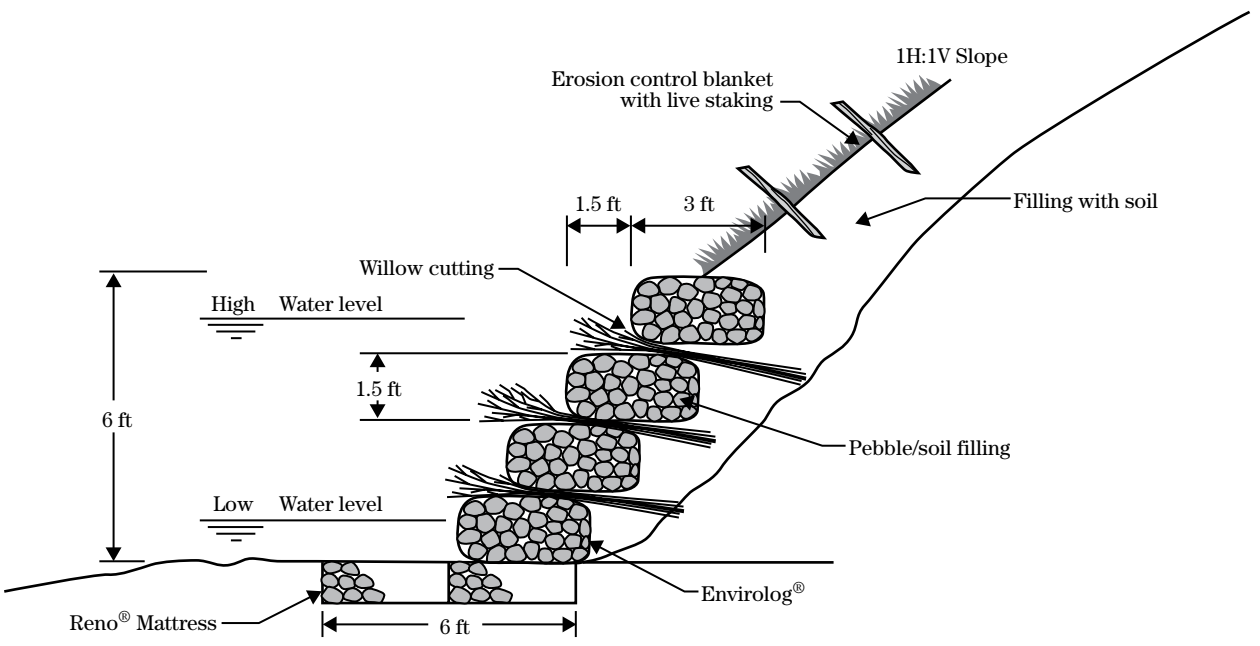


(3 ft), which is less than the maximum wave action allowed for stability.

Ice impacts can be analyzed best using scale models for laboratory testing. It was decided that this type of analysis would be too expensive. Additionally, ice has not been a problem in this reach of the Merrimack River for the past few years. The probability of local damage by ice impact was determined to be very limited based on the experience of the manufacturer and because of the added protection afforded by the vegetation on the external face of the gabion units.

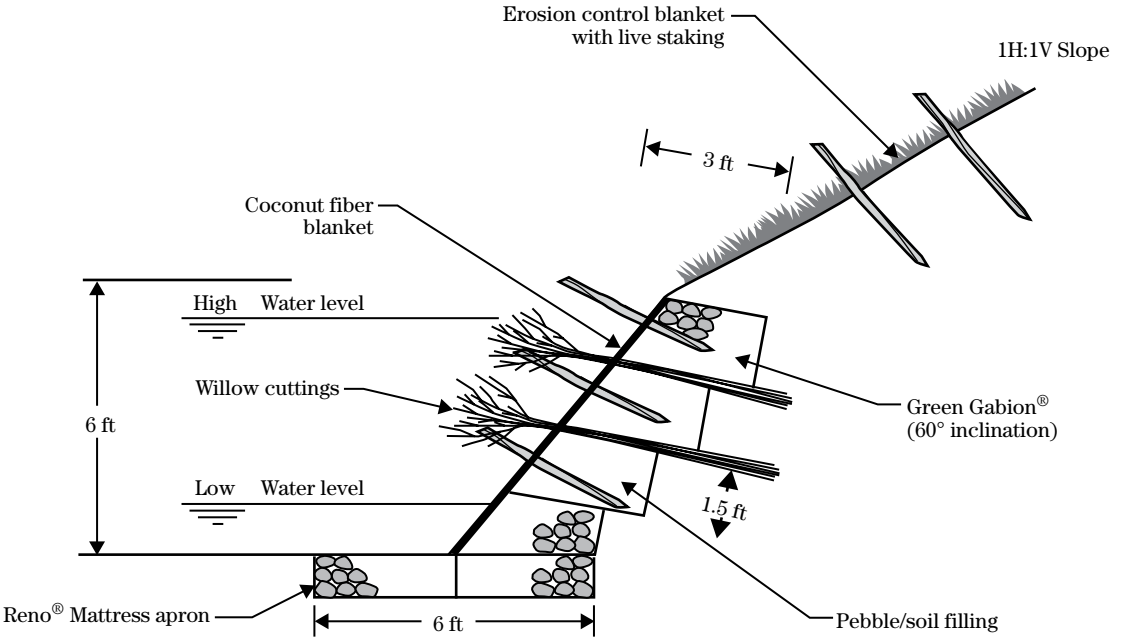
To prevent any potential scour under the structure, a Reno® Mattress of 30 centimeters (12 in) by 1.9 meters (6 ft) was installed at the base of the units, with an extension of 1 meter (3 ft) streamward to stop any erosion (figs. CS14–2 and CS14–3).

Figure CS14–2 Reno® Mattress, gabions, and vegetation form a stable streambank design



Typical cross section

Figure CS14–3 Reno® Mattress used with vegetation to create a stable slope



Material selection

The selection of appropriate materials was very important to the environmental aspects of this project so that the final installation would be integrated into the surrounding environment. Two new types of products were selected that offer strength, resistance, and natural integration. Green Gabion® (fig. CS14–4) is a trapezoidal-type of gabion with an inclination angle of 60º, lined with 900 grams per square meter (26.5 oz/yd^2) coconut mat in the facing. The unit is 2 meters (6.5 ft) long, 1 meter (3.25 ft) wide, and 0.5 meter (19 in) high. Envirolog® is a cylindrical type of gabion lined with 900 grams per square meter (26.5 oz/yd^2) coconut mat. Both types of product used in this project are made of PVC-coated woven wire mesh. To ensure durability of the structure, all wire is galvanized and PVC-coated before being transformed in mesh. The flexibility and strength of the double-twisted mesh are very important. The flexibility of the mesh allows the units to adapt to the profile of the bank even after any subsequent settlement.

The voids of a gabion structure (30 to 40%) are filled with topsoil. The topsoil helps to retain the moisture in the structure to promote growth of the vegetation and also acts as a substrate for the propagation of the root system between the stones. This forms a strong interlocking with stone and wire and roots.

To retain the topsoil in the units for vegetation, a thick layer of coconut mat is lined inside the wire mesh basket (figs. CS14–4 and CS14–5).

The coconut mat, with a durability of 3 to 5 years before its biodegradation, helps to maintain the topsoil's moisture for the vegetation.

Assembly and installation

Construction on the site began in June 2002. Don Wheeler Construction of Bedford, New Hampshire, was the contractor. Work began with cutting the approved trees, clearing an upper work area, and creating a temporary tee so the 6th fairway could still be used during the summer. Work was completed in September 2002. The project schedule was extended for numerous reasons including the very tight labor market at the time of construction. Ideally, this type of project works best with a six-person crew; however, there were times when only two workers were available for the project.

Figure CS14–4 Green Gabion® schematic

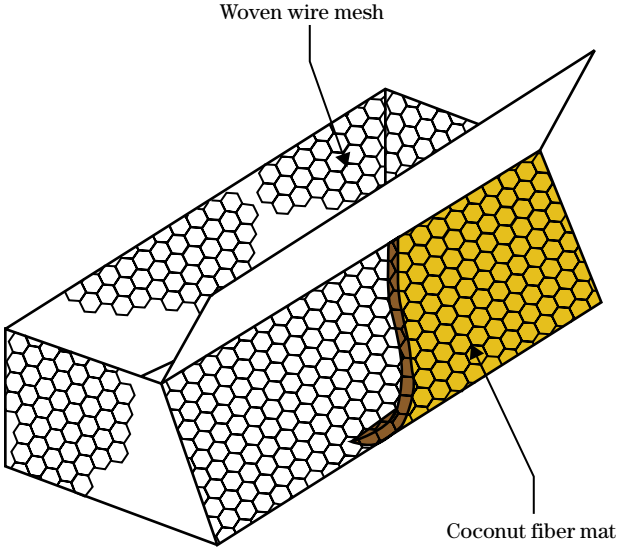
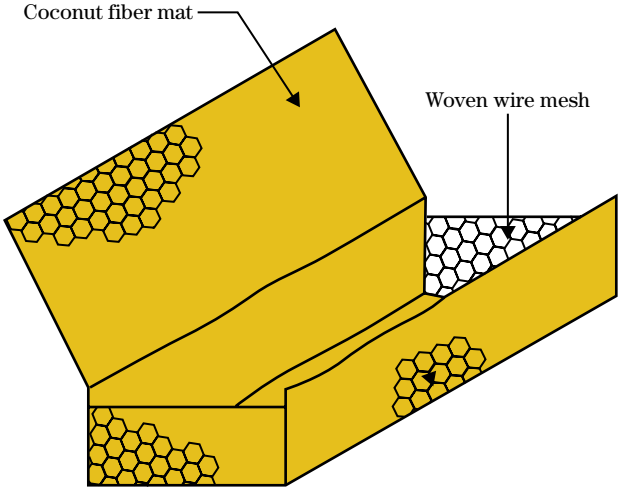


Figure CS14–5 Coconut fiber was used to line gabion baskets to retain soil for vegetation growth.



Because of the timing of the project, the original concept of dormant cutting was abandoned in favor of potted willow and dogwood shrubs. The revised specification called for one 1-gallon shrub per linear foot of lift, totaling approximately 4,000 shrubs.

The units were preassembled as a box and lined with the coconut mat before being placed on the site (figs. CS14–6 and CS14–7).

The coconut mat was secured to the wire mesh using lacing wire. The units were placed side by side and connected together with the adjacent units and with the upper and lower baskets. Between each row, willows and dogwood shrubs were placed at an interval of 30 centimeters (1 ft).

After being placed and secured together, the units were filled with stone and topsoil (fig. CS14–8). The lower sections, which are more exposed to the wave action, first were filled with stone followed by topsoil to fill the voids. Filled in this way, the bottom units were getting a higher percentage of stone to prevent loss of topsoil with time. The upper section was filled with stone and topsoil already mixed together before being placed in the units.

Figure CS14–6 Green Gabions® preassembled with coir fiber mat

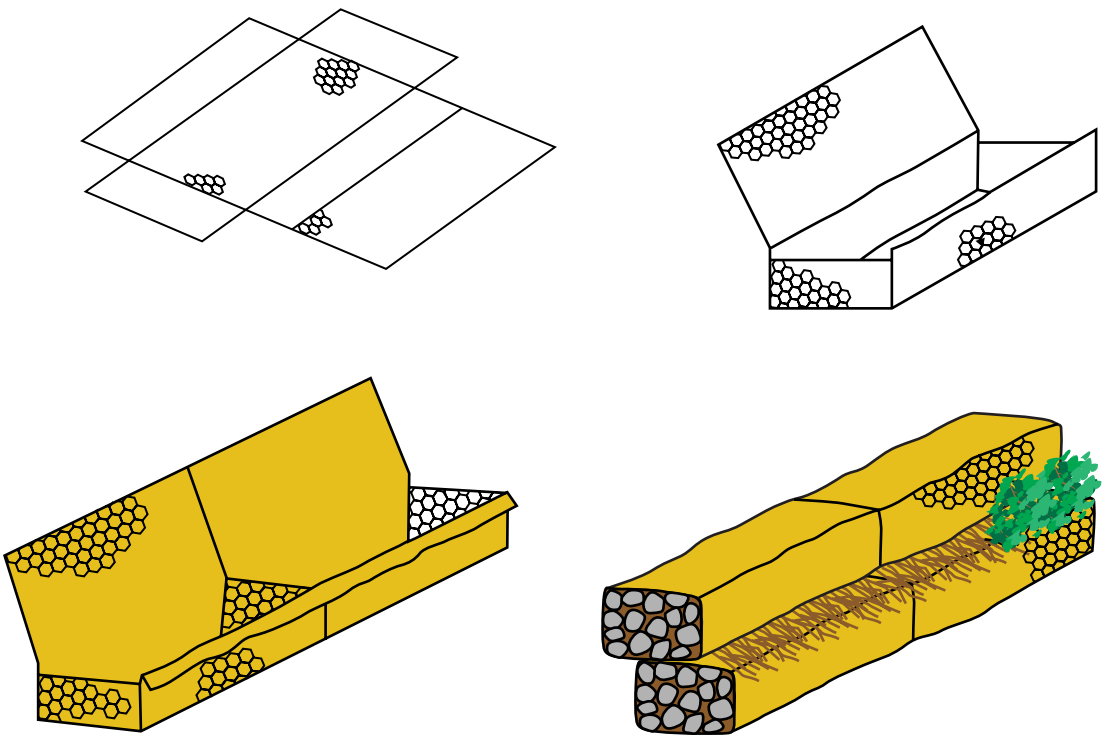


Figure CS14–7 Green Gabions® preassembled with coir fiber mat

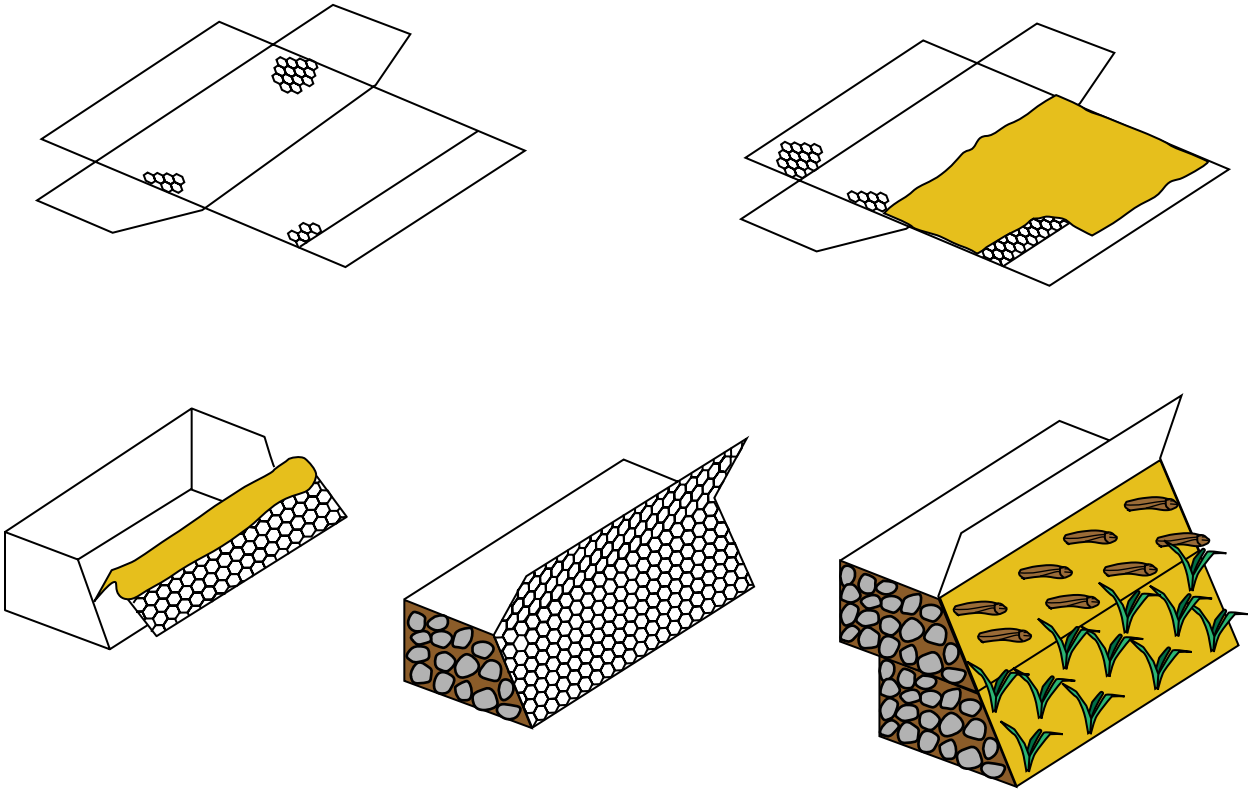


Figure CS14–8 Installed gabion baskets lined with coconut fiber mat and filled with stone and topsoil

Conclusion and recommendations

In general, the project work was done as per the specification for the assembly and installation of the units. Because of the limited budget, a full-time inspector on the project was not possible. Some small problems could have been prevented with an inspector on the site, considering that the contractor did not have a lot of experience with gabions. In particular, there was a problem with the size of the stone being too small in the Reno® Mattresses used for toe protection. Some stones have already come out of the mattress after 1 year. These mattresses should be opened and refilled with proper sized stone.

After only one growing season, the vegetation is generally well established where it was planted. Some areas where the plants were too small or not properly placed during the installation have some problems with establishment in the structure. Figures CS14–9 and CS14–10 show the progression of the growth of vegetation. Figure CS14–9 was taken in the spring, and figure CS14–10 in the midsummer of the first growing season. The modified gabions used on this project combined with thick coconut blanket can resist erosion for 3 to 5 years before seeing any degradation of the coconut blanket. Live staking was recommended by the designer to enhance vegetation where it was missing.

In general, the project is a success considering that few repairs or follow up will be necessary. The landowner now has a stabilized bank, and this green solution can certainly be applied elsewhere along the Merrimack River to fix eroding streambanks.

Figure CS14–9 Project in spring after initial installation

Figure CS14–10 Completed project in midsummer after installation

Case Study 15

Streambank Stabilization in the Guadalupe River Basin, Santa Clara County, California

Issued August 2007

Cover photo: Treated section of the Guadalupe River in Santa Clara County, California

Advisory Note

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Case Study 15

Streambank Stabilization in the Guadalupe River Basin, Santa Clara County, California

By John McCullah, Watershed Geologist, CPESC, and Kaila Dettman, Watershed Hydrologist of Salix Applied Earthcare

Introduction

As part of a mitigation plan, geotextile blankets were applied to a bank of the Guadalupe River in an attempt to stabilize the channel slope in the fall of 2002. The blankets were installed within the channel, perpendicular to the flow. This project failed during a storm event in December 2002. Material was washed from beneath, and the blankets were dislodged and torn. In an attempt to repair the damage and provide education regarding biotechnical stabilization solutions, the Santa Clara Valley Water District employed the authors to recommend practices that would include a vegetative component to help stabilize the bank and improve habitat and aesthetic benefits.

Site description

Located in Santa Clara County in central California, the Guadalupe River drains 170 square miles of watershed into the southern tip of the San Francisco Bay. The river begins at the confluence of Alamitos and Guadalupe creeks, flows north through the center of San Jose, then discharges into the bay at Alviso. The river is surrounded by urbanization, yet still provides habitat for many plants and animals including Steelhead and Chinook salmon, both protected by the Endangered Species Act.

In the 1970s and 1980s, the Santa Clara Valley Water District was required to replenish the aquifer and control flooding. The Guadalupe River Basin has been greatly affected by human activity including the urbanization of the surrounding areas, installation of dams and reservoirs, the channelization of streams, and construction of levees for flood protection. Many miles of streambank were armored with sack concrete walls and riprap. Due to environmental concerns, the district was required to shift the focus of stream projects to incorporate only “green” solutions in the mid-1990s. This included replacing many hard structures constructed during past projects. Flooding concerns, threats to bridge integrity, loss of wildlife habitat, water quality issues, and lack of aesthetic value have led to significant restoration efforts within the watershed.

One site targeted for restoration is located between Willow Glen Way and Alma Avenue on both the east and west banks of the Guadalupe River. It is known as Reach 9 within the Stream Maintenance Program (SMP) administered by the Santa Clara Valley Water District. The river is channelized, narrow, incised, and characterized by high velocities and peak flows during storm events. The channel substrate is primarily made up of fine gravels with cobbles and small boulders. Prior to restoration efforts, the channel was confined by a sack concrete wall. Many areas up- and downstream of the site have solid reinforced concrete banks. The river cannot migrate laterally due to the presence of a bridge, and houses are located close to the channel.

Original project

The original restoration work involved removing the damaged concrete access ramp and the sack concrete wall (fig. CS15–1) and replacing the structures with more habitat-friendly treatments. Restoration of the streambank included complete removal of the access ramp. Biotechnical techniques were installed on

Figure CS15–1 Damaged access ramp and wall

the bank including vegetated log cribwall structures, geotextile erosion control blankets, and native vegetation. The primary goal was to protect the integrity of the downstream bridge while providing shade, wildlife habitat, and local bank stability.

Installation proceeded as planned in fall of 2002 (fig. CS15–2). Log structures were keyed into the toe of the slope to provide anchoring and stability. Blankets made from coir netting (900 g/m^2) were laid out vertically along the slope (or perpendicular to the flow, which was consistent with manufacturer recommendations for slope protection), with coir rolls installed on the top and toe edges of the blanket layer. The blanket was then stapled along the length of each section.

2002 storm event

During December 2002, the first winter following project completion, a significant storm event occurred with a 5-year return frequency. The coir netting was severely damaged, and the underlying fill on the toe and midslope of the streambank was undercut and washed away (fig. CS15–3). The logs at the toe of the slope remained in place, yet extensive scour occurred around the structures (fig. CS15–4).

Hydraulic analysis indicated that the river reached velocities of 6 feet per second, developed shear forces of approximately 1 pound per square foot, and was at least 17 feet deep during the storm. The question was whether the failure was due to shear forces too great for the geotextile material or due to improper installation.

Figure CS15–3 Damaged streambank following a 5-year frequency event in December 2002

Figure CS15–2 Finished original project in fall of 2002

Figure CS15–4 Evidence of scour and exposure of the log structures

Biotechnical project installation

In May 2003, a field trip was conducted along the Guadalupe River with staff from various Federal, state, and local agencies to come up with alternatives and designs to repair the damaged bank. The authors were asked to visit the site and recommend potential techniques that could stabilize the slope while providing wildlife habitat and aesthetic value. Recommendations, designs, and specific installation criteria were supplied for live siltation, cobble placement, and a redesigned coir blanket layout. Installation commenced in July 2003, and work began with the removal of the torn and dislodged blankets and regrading of the damaged bank. Maintenance crews of the Santa Clara Valley Water District performed the construction and application with guidance and training by the authors.

Live siltation

Live siltation is a technique in which willow cuttings are arranged in bunches to provide roughness and encourage sediment deposition. Willow cuttings were staked into the soil and rock above and within the log structures at the toe of the bank (fig. CS15–5). Deep, voided areas from the previous erosion allowed the stakes to be planted without digging. Willows are capable of withstanding periodic inundation and high velocity flows, and the roots help anchor the toe, while surface vegetation provides shading and habitat.

Cobble placement

The use of stone was prohibited due to new policies that allowed only “green” solutions within the watershed. The recommendation that cobble be installed was met with resistance; however, regulatory personnel were reassured that cobble would provide natural habitat and aesthetic value while improving channel integrity. Following the installation of the willows, cobble was placed on the toe of the slope to armor the bank and provide initial support for the willows during establishment (fig. CS15–6). Rounded cobble was used and fit in with the appearance of the river while providing the stability needed at the toe of the bank.

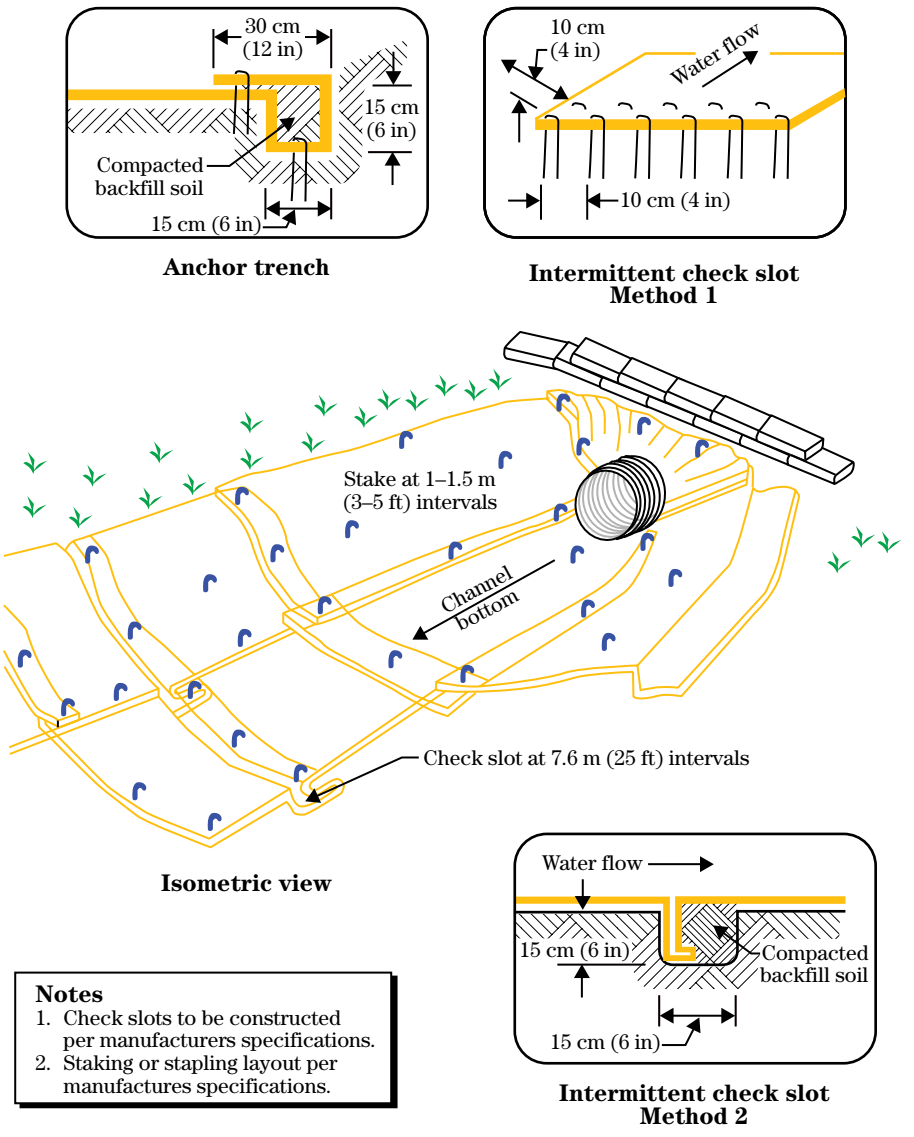
Figure CS15–5 Live siltation installation

Figure CS15–6 Cobble placement following the live siltation installation

Coir blanket installation

The authors determined that the blankets had been installed incorrectly because the application had been applied based on the principles of slope stabilization, not channel stabilization. Channel installation requires that blankets be oriented parallel to the flow and include anchor slots (fig. CS15–7). Prior to coir blanket application, the slope was regraded, top soil was placed on terraces and compacted to 85 percent of optimum density, and mulch and seed were applied hydraulically to the slope. Longitudinal slots (horizontal) and check slots (vertical) were dug into the fill material on the middle and upper portions of the bank to serve as footings for anchoring the blankets and coir logs.

Figure CS15–7 Erosion control blankets channel installation



Notes
1. Check slots to be constructed per manufacturers specifications.
2. Staking or stapling layout per manufactures specifications.

Materials included 55 coir logs at 6 inches in diameter and 10 feet in length, 7 rolls of GEOCOIR®/DeKoWe® 900 geotextile blanket, and one 100 #3 rebar J-hook staples (with 18-in legs and 6-in hooks). The coir blankets were rated to withstand velocities of 10 feet per second by the manufacturer. The blankets were installed loosely to allow all fibers to make contact with the soil surface and were laid horizontally along the slope (parallel to the river) to reduce the potential for undermining and dislodging by high flows. The coir logs were installed over the blankets in the previously dug trenches and anchored with the hooked staples on both sides of the coir log. The edges of the blankets were incorporated into small trenches and backfilled to secure the material to the slope (fig. CS15–8). Additional cobble was brought in and incorporated on the toe of the slope following blanket installation, and the project was completed (fig. CS15–9). Container plants were installed later with DriWater™ packets to provide necessary moisture during the establishment period.

Biotechnical project performance

During the winters of 2003 and 2004, the slope remained stable, and the blankets, cobble, and willows stayed in place. Photographs taken during storm events demonstrated the inundation experienced by the slope during high flows (fig. CS15–10). Following a storm in January 2004, the flow line reached high elevations on the slope and indicated that the restoration work had been successful (fig. CS15–11).

Conclusion

The restoration work performed in 2003 was extremely successful in stabilizing the slope. In April 2004, the bank was becoming vegetated (fig. CS15–12), and the willows were successfully established at the toe (fig. CS15–13). For future projects, plantings should be performed with minimal disturbance to the geotextile blanket (holes cut for the vegetation were quite large and could provide a potential point of undermining). The project emphasized the importance of installing geotextile blankets parallel to the flow of water within channels where inundation will occur. Combining techniques such as cobble revetment, live staking, pole planting, live siltation, hydroseeding, and coir blankets provided the stability, habitat, and aesthetic value needed for the site.

Figure CS15–8 Slope preparation and blanket installation

Figure CS15–9 Finished project

Figure CS15–10 Inundation of the restored slope

Figure CS15–11 Flow line after a storm in January 2004

Figure CS15–12 Revegetated slope 1 year later

Figure CS15–13 Willow establishment at the toe

Case Study 16 Coffee Creek, Edmond, Oklahoma

Issued August 2007

Cover photo: Treated section of Coffee Creek in Edmond, Oklahoma

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 16 Coffee Creek, Edmond, Oklahoma

Robert P. Armstrong, P.E., Senior Project Manager, Huitt-Zollars, Inc., Dallas, Texas; William D. Armstrong, P.E., RPLS, Senior Director of Development and Marketing, BWR Corporation Inc., Bartlesville, Oklahoma; Mark L. Johnston, P.E., Project Manager, BWR Corporation Inc., Oklahoma City, Oklahoma; Nancy L. Kennedy, Flood Plain Administrator, City of Edmond, Oklahoma

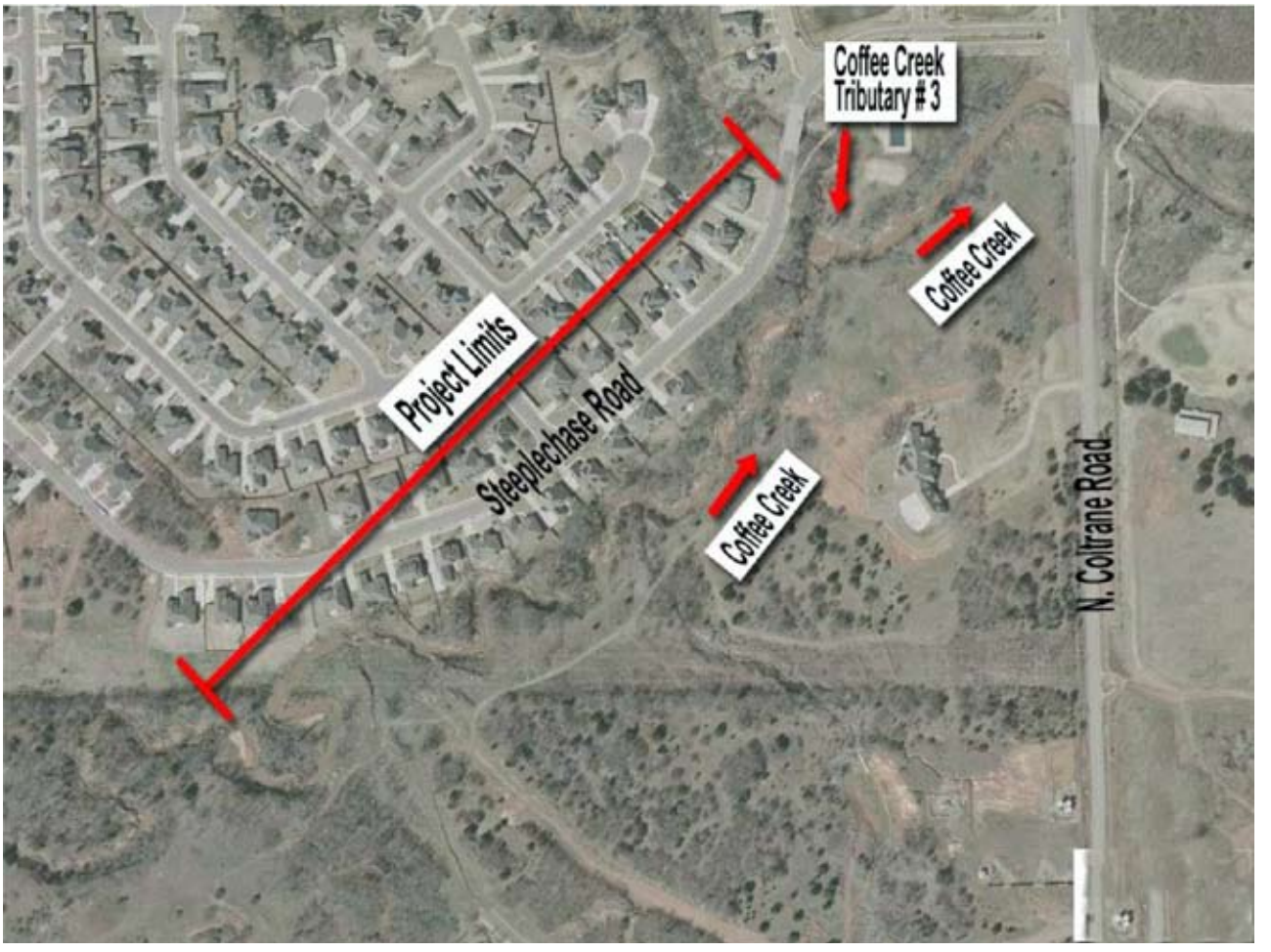
Project background

In October 2000, an extreme flood event caused severe erosion and lateral instability of a portion of Coffee Creek located along the southeastern edge of the Steeplechase subdivision in Edmond, Oklahoma (fig. CS16–1).

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) provided grant funding through the Emergency Watershed Protection (EWP) program, covering 75 percent of the costs for the design and construction of channel improvements to protect nearby homes and public utilities. The project area is comprised of the reach of Coffee Creek that begins upstream of Coltrane Road at the confluence between Coffee Creek and Coffee Creek Tributary No. 3 and extends upstream approximately 2,300 feet (fig. CS16–2). Coffee Creek lies within a Federal Emergency Management Agency (FEMA)-regulated Zone AE flood plain.

Figure CS16–1 Location of project

Figure CS16–2 Aerial photo of project area



Existing conditions

Coffee Creek is a deeply incised channel with a width/depth ratio of less than 5H:1V. The entrenchment ratio for this reach of Coffee Creek, the ratio of flood plain width to channel width, is approximately 2H:1V. Coffee Creek is a highly meandering stream and has undergone active lateral migration leading to a high degree of instability (figs. CS16–3, CS16–4, and CS16–5.) The channel cross section is highly irregular. Near vertical banks have developed in several locations. Previous attempts to prevent lateral migration through the use of riprap and retaining walls have been unsuccessful.

The channel bed slope is relatively flat at 0.0038 and appears to be relatively stable downstream of and within the project reach. There are no apparent signs of significant active channel bed degradation throughout most of the project reach. However, there are indications of previous bed degradation and headcutting. The headcutting was prevented from migrating farther upstream by an existing gabion grade control structure that had been installed as protection for a high-pressure gas line. The drop in channel flowline elevation across the gabion structure is approximately 4 feet, and the structure was slightly undermined and being flanked.

The soils throughout most of the project reach are highly erodible. Bank slopes are comprised typically of sandy clay and sandy loam. Bed material is mainly a sandy loam. A few areas of the bed are comprised of fat clay (highly plastic) with low erodibility. The ordinary high water depth is approximately 2 to 3 feet. Vegetation above the ordinary high water mark and within the channel is not well established due to the high degree of lateral instability. Limited riparian habitat exists outside of the main channel.

Figure CS16–4 Streambank erosion prior to the project

Figure CS16–3 Streambank erosion prior to the project

Figure CS16–5 Proximity to infrastructure imposed restrictions on design approaches

Hydrology and hydraulics

The contributing drainage area of Coffee Creek at the downstream end of the project reach is approximately 3.4 square miles comprised mostly of residential and rangeland areas. Just downstream of the project reach, Coffee Creek Tributary No. 3, a left bank tributary with a drainage area of approximately 2.8 square miles, discharges into Coffee Creek. FEMA discharges for Coffee Creek are published in the revised Flood Insurance Study (FIS) for the City of Edmond, Oklahoma, dated April 16, 1990. The 50-, 100-, and 500-year discharges are estimated by FEMA to be 2,465 cubic feet per second, 3,115 cubic feet per second, and 4,676 cubic feet per second, respectively. The existing channel banks are overtopped by discharges between the 50-year and 100-year event. The FEMA 100-year discharge was used for determining regulatory compliance for the project. The FEMA 500-year discharge, which is approximately 50 percent greater than the 100-year discharge, was used as the design discharge for the restoration project.

The current regulatory FEMA model for Coffee Creek was provided by the city. The U.S. Geological Society (USGS) Stepbackwater Program, E431, was used in the FEMA modeling. Data from the FEMA regulatory model was translated into a HEC–RAS model (U.S. Army Corps of Engineers (USACE) 1995a) for this study. Cross sections of existing conditions were then established throughout the project reach using current topography data and were added to the hydraulic model to supplement description of existing conditions.

Average channel velocities for the existing conditions channel generally ranged between 4 and 9 feet per second for the 500-year discharge. The maximum average shear stress for the existing channel was estimated to be approximately 7 pounds per square foot for the 500-year discharge.

Project objectives and design constraints

The primary objective of the project was to eliminate the excessive erosion and protect public and private infrastructure along the project reach. Threatened infrastructure included:

- two sanitary sewer lines
- several homes
- a high-pressure gas line
- a high-power electric transmission line
- a domestic water line
- an oil field operation in the area

A secondary objective was to restore the channel using soil bioengineering methods to the greatest extent practicable to minimize potential adverse environmental impacts of the project and maximize aesthetic value.

Several significant project constraints were addressed. Project management constraints included:

- coordinating and securing the 25 percent matching funds from multiple sources including the municipal stormwater utility, adjacent homeowners, and a private utility company
- a single property owner of most of the land along this reach of Coffee Creek requiring significant project coordination regarding the project approach, alignment, and easement issues
- scheduling issues related to project funding and the securing of easements placed a significant constraint on design activities, forcing the majority of the project design to occur over a 2-week period

Project design constraints included:

- using soil bioengineering techniques, while being confined to a less than desirable cross-sectional width
- protecting existing infrastructure and avoiding adverse impacts to other adjacent infrastructure

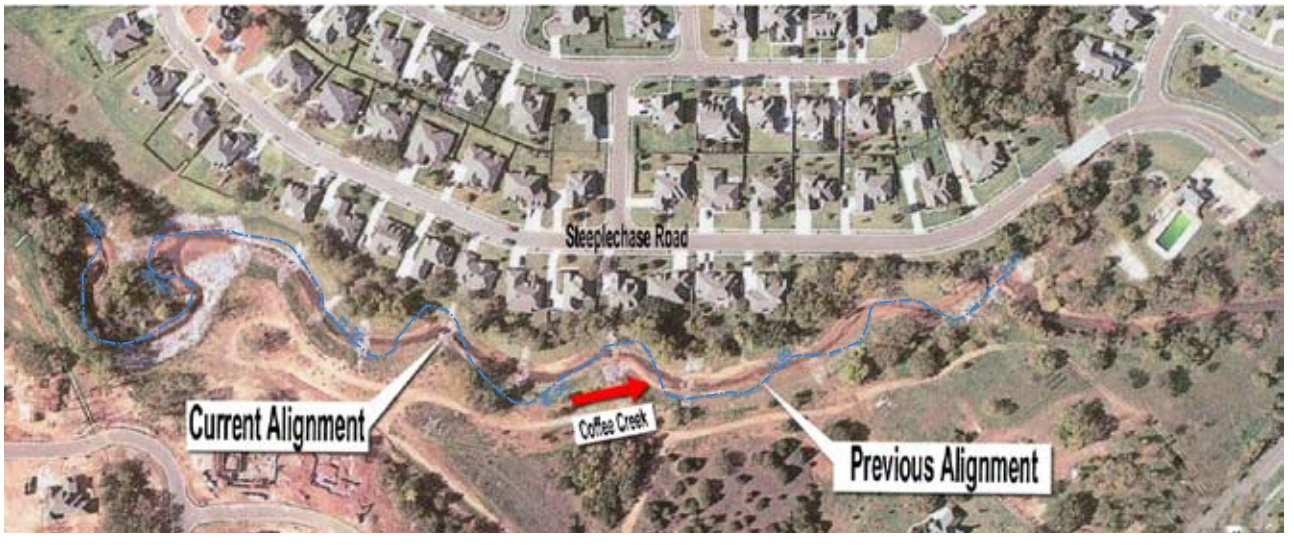
The project is located within a FEMA Zone AE flood plain and required fill within the regulatory floodway. Due to project scheduling constraints, there was not enough time to secure a Conditional Letter of Map Revision from FEMA. Therefore, a zero-rise condition for the 100-year flood event was necessary to meet local and Federal flood plain requirements.

Design elements

Consideration of the project objectives and constraints resulted in the use of several specific design elements including:

- A horizontal alignment was developed to move the creek away from the threatened infrastructure to the greatest extent practicable, while also minimizing encroachment of the creek farther onto the adjacent primary property owner, and minimizing changes to the existing meander pattern. A comparison of existing conditions and proposed channel alignments is provided in figure CS16–6.
- Grade control structures were needed to maintain a stable channel bed and to compensate, through the use of small drop structures, for the reduced channel length of the proposed alignment.
- A uniform typical section was developed to provide a low-flow channel capable of conveying the 2-year discharge. The channel was also designed to maximize conveyance, while minimizing encroachment onto the adjacent property.
- Stone riprap was used to secure a fixed channel alignment at the upstream and downstream ends of the project.
- A three-tiered approach was developed for controlling the horizontal alignment of the creek. The approach provided hard armoring to protect the nearby infrastructure and also maintained a natural section to the greatest extent practicable. The main elements of the three-tiered approach:
  - Grade control structures were designed to double as horizontal alignment control structures. The structures were designed as sheet pile weirs with concrete caps. These structures provide hard armoring stability at select locations within the project reach. Structure locations were selected to provide protection for threatened infrastructure, as well as provide overall stability to the channel alignment by creating fixed points along

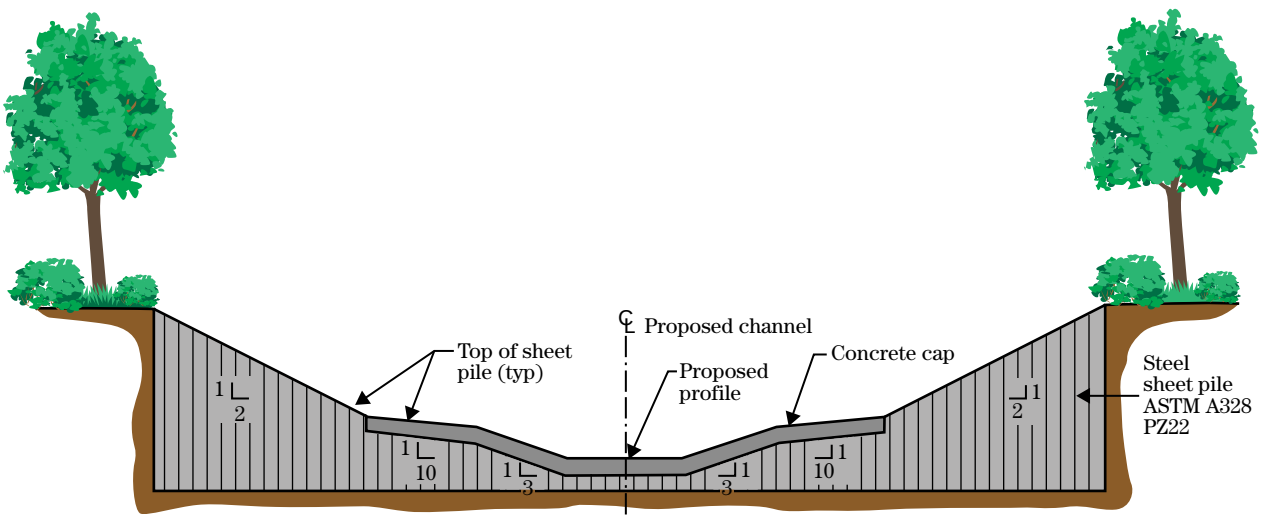
Figure CS16–6 Aerial view of project channel alignments



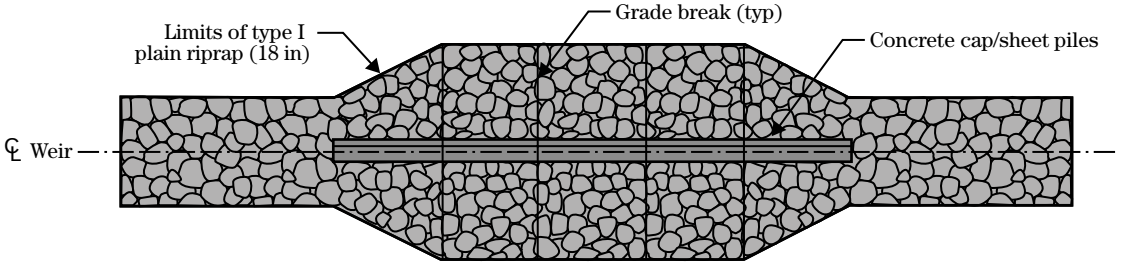
the alignment. A typical section and plan view of the weir structures is provided in figure CS16–7.

— Native grasses and legumes were designed for the flood plain benches and channel side slopes between the weir control structures. The vegetation is supported by a combination of turf reinforcement mat and degradable erosion control blankets. A variety of trees were placed along the top of the channel bank to replace the riparian and upland habitat removed in the channel realignment. This design element enhances channel stability between the hard-armored weir structures, while maintaining a natural section to maximize the environmental benefits and aesthetic values of the project. The low-flow channel was left unreinforced to allow the creek freedom to meander and form a natural aquatic habitat. The small flood plain bench was reinforced with vegetation and a degradable erosion control mat. The lower portion of the 2H:1V main channel bank slope was reinforced with turf reinforcement mat to help maintain a stable toe of slope. A degradable erosion control mat was also used for the upper portion of the main channel bank slope where shear stresses are lower, and the additional protection afforded

Figure CS16–7 Weir structure, plan view, and cross section



Channel weir typical cross section



Channel weir plan view

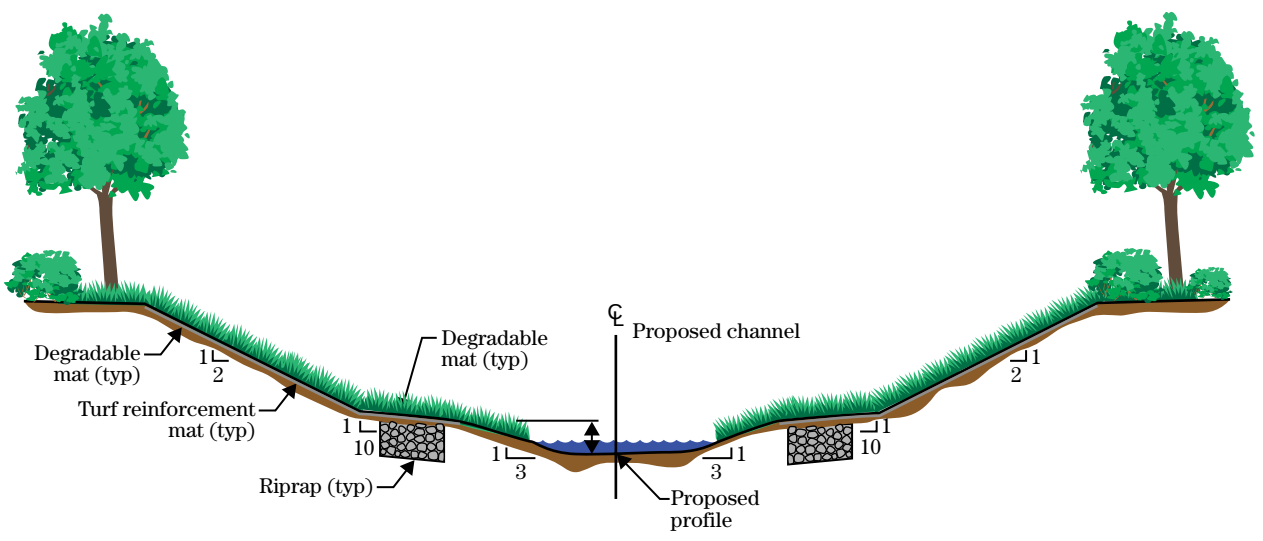
by the turf reinforcement mat was not necessary. A typical section of the channel is provided in figure CS16–8.

— It is recognized that due to the proximity of adjacent infrastructure, adequate space was not available to re-create what may be considered a stable channel through application of traditional geomorphologic principles. Therefore, since the low-flow channel may, over time, begin to change size and alignment and place the stable 2H:1V main channel banks at risk, buried riprap was placed under the main channel toe of slope along the entire length of the project between the weir structures. While the low-flow channel and flood plain benches were allowed to remain natural, and hence deformable, the riprap was added to provide some assurance of a stable toe for the main channel bank.

Results

Project construction began in October 2002 and was substantially complete in August 2003. The project has been subjected to several significant postconstruction runoff events including an event that inundated the flood plain bench. Vegetation has begun to establish, and the channel is stable. As expected, the low-flow channel has begun to change shape and alignment. In a few locations, the low-flow channel has migrated to, and exposed, the buried riprap toe. The excessive erosion and scour, lateral migration, and general widening of the channel have ceased. The channel banks are stable, and the infrastructure has been protected. The project will continue to be monitored to help assess and document long-term performance of the design elements. Figures CS16–9 through CS16–11 show a comparison of preconstruction photos and photos taken after construction was substantially complete.

Figure CS16–8 Typical channel cross section with toe slope protection



Channel typical section

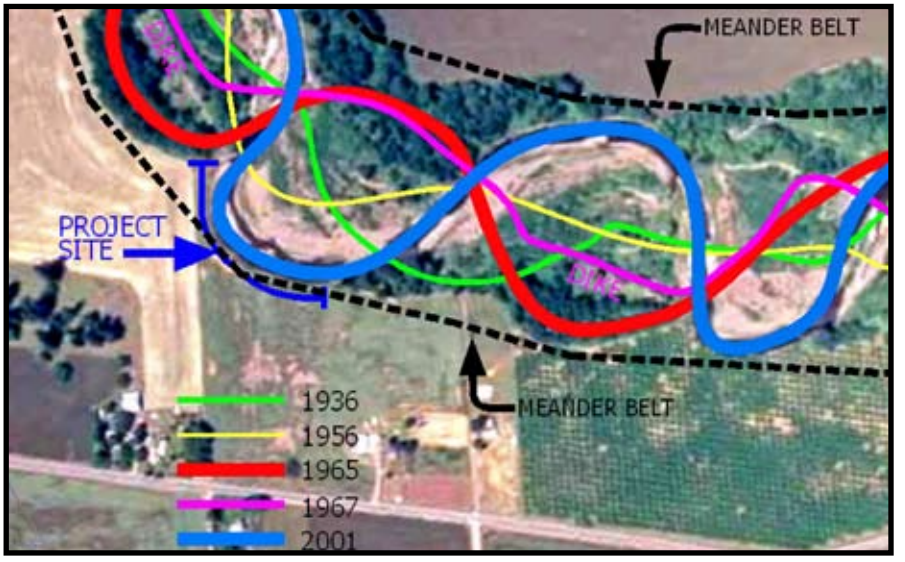
Figure CS16–9 Eroding channel prior to construction, and after project implementation; house and property lines imposed restrictions on the design

Figure CS16–10 Eroding channel prior to construction, and after project implementation; house and property lines imposed restrictions on the design

Figure CS16–11 Eroding channel prior to construction and after project implementation

Case Study 17

Stream Barbs on the Calapooia River, Oregon



Issued August 2007

Cover photo: Historic channel traces of the Calapooia River in Oregon

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Case Study 17 Stream Barbs on the Calapooia River, Oregon

By Scott Wright, P.E., area engineer, U.S. Department of Agriculture, Natural Resources Conservation Service, Tangent, Oregon

Introduction

In January 2002, the Carbajal Streambank Stabilization project was funded through the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) Environmental Quality Incentives Program (EQIP). The project was approved to provide design and technical assistance to stabilize approximately 1,000 feet of riverbank (fig. CS17–1) along the Calapooia River in Linn County, Oregon. The river had 10-foot vertical banks and was eroding laterally at an average rate of 10 feet per year, with localized areas in excess of 20 feet per year. The Calapooia also has several salmonid species including the threatened Chinook salmon and winter steelhead. The district conservationist, engineer, and landowner developed the following project objectives:

- reduce bank erosion and loss of productive agricultural lands
- provide fish habitat and habitat diversity for endangered species
- not impact upstream and downstream landowners
- establish a stable riparian buffer strip

Design options were developed in accordance with NRCS standards, project objectives, and statewide programmatic biological opinion for endangered species. The final design was four rock barbs incorporating large wood, two engineered log structures, bank shaping, and vegetative planting. The project experienced a 5- to 10-year flow event 3 months after completion, and no noticeable erosion was observed along the riverbanks. In addition, significant areas of biodiversity were developed as a result of scour around the barb structures and proliferation of vegetation along the enhanced riverbanks. Total project cost was approximately $70 per foot of streambank stabilized.

Figure CS17–1 Calapooia River project (*Photos courtesy of Scott Wright*)

(a) Preproject conditions looking downstream at outside bank during summer low flows

(b) Postproject conditions looking downstream at outside bank during summer low flows

Fluvial geomorphology

The Calapooia River drains a 366-square-mile watershed area on the western foothills of the Cascade Range, with a mean annual precipitation of 60 inches. The river is more than 70 miles long with headwaters at an elevation of approximately 5,200 feet and a confluence elevation of 200 feet at the Willamette River. The river system contains several anadromous salmonid species including spring Chinook and winter steelhead that are listed as threatened under the Federal Endangered Species Act (ESA) of 1999.

Based on the Calapooia Watershed Assessment by the local watershed council, significant channel alterations had been performed from 1900 to 1980. Figure CS17–2 illustrates typical work in the watershed.

An aerial photo from 1966 (fig. CS17–3 (modified from U.S. Army Corps of Engineers (USACE) photo)) shows a dike just downstream of the project site constructed from excavated instream materials placed to cut off a meander bend. In addition to the channel realignment at the project site, another cutoff dike was constructed two meander bends upstream from the project site to cut off another meander.

To document historic channel alterations and natural changes, a composite picture of channel alignments was assembled. Figure CS17–4 shows the historic channel alignments from 1936, 1956, 1965, 1967, and 2001, superimposed on the 2001 aerial photo. The river's response to the 1966 meander cutoff dikes is readily visible as the meander phase shifted 180 degrees based on a sine curve relationship. The current river location mirrors the predike conditions in 1965. Analysis of traces of the historic channel highlights the heavily altered state of the river and the dynamic response to stream modifications. Based on nearly 70 years of channel traces, the meander belt width measures approximately 1,000 feet.

It is clear from the analysis of the historic channel that the project area is located near the outer edge of the historic meander migration zone. This allows for more streambank stabilization options because the stabilization will not have an impact on overall planform, nor would it affect flood plain connectivity since the project would not change top-of-bank elevations.

Figure CS17–2 Channel alterations in the Calapooia River in 1950s

Figure CS17–3 Aerial view of project area in 1966 showing dikes used to cut off meanders

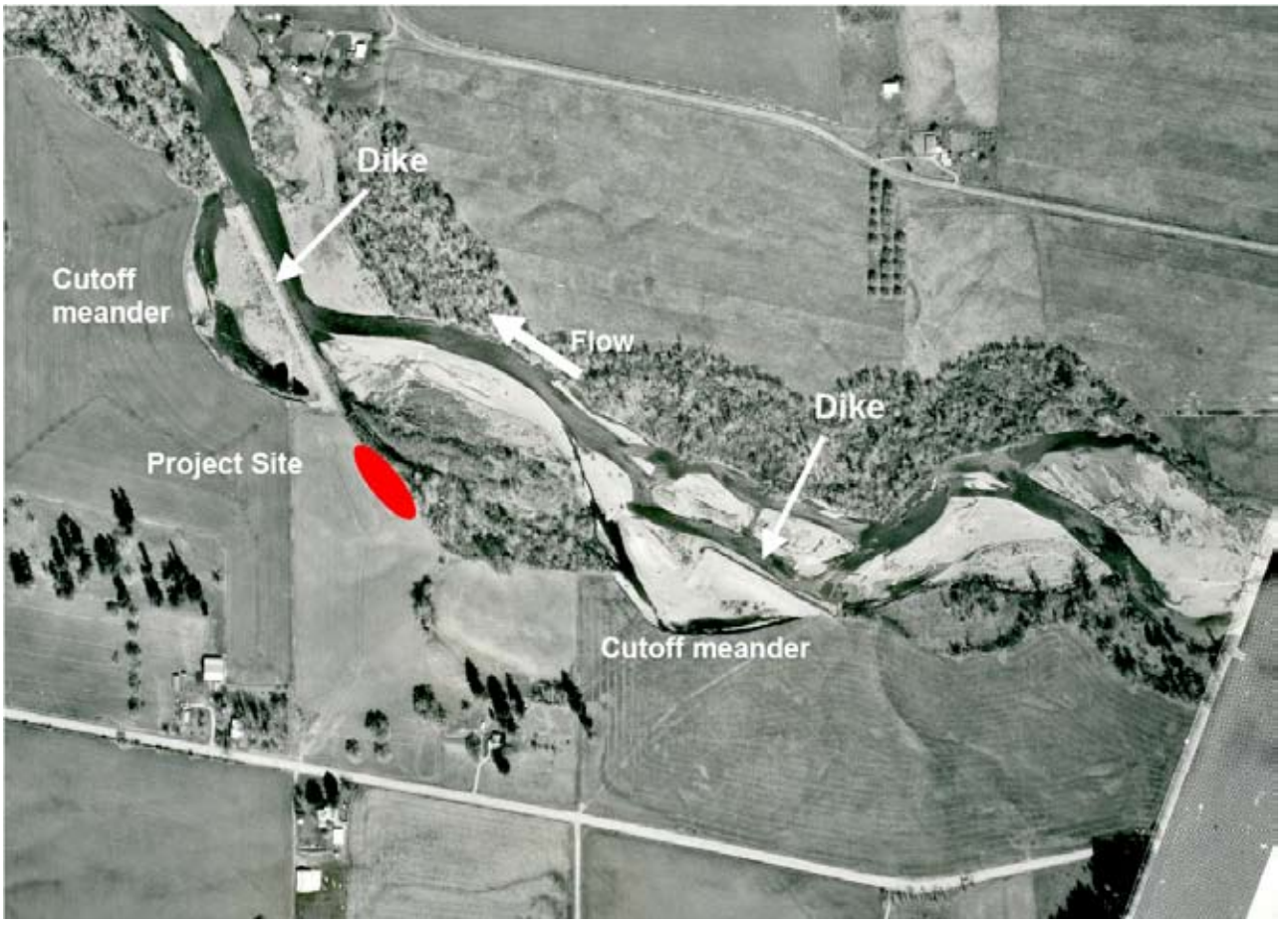
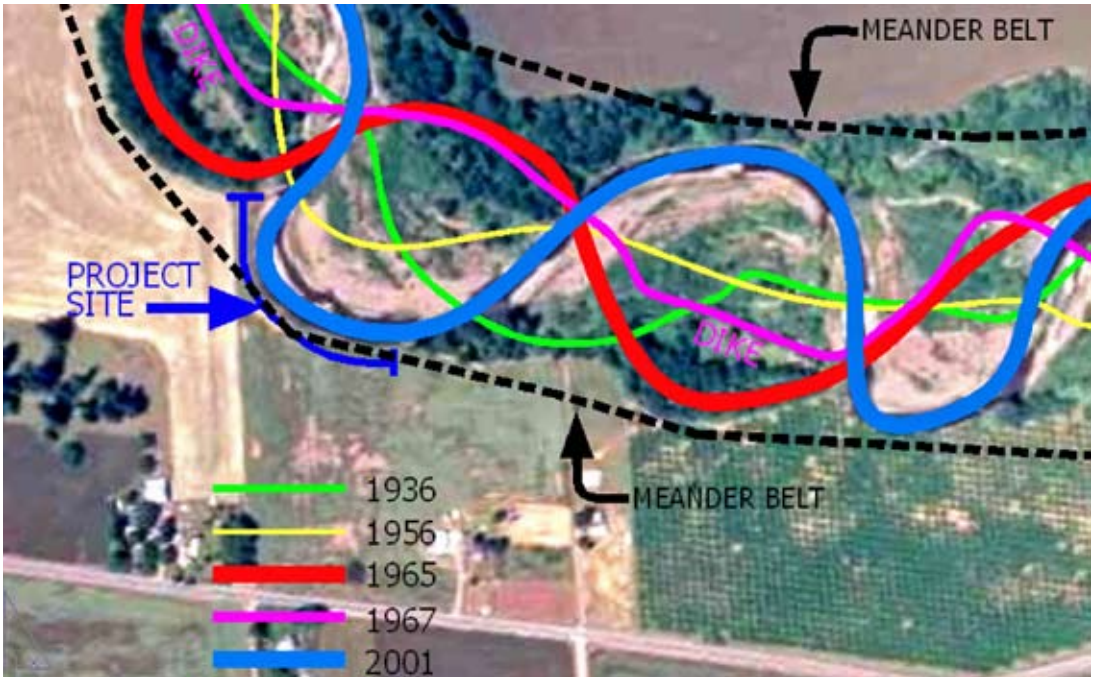


Figure CS17–4 Historic channel traces with corresponding year designated by color



Hydrology and hydraulics

The project site is located in a compound meander that starts with a radius of 450 feet and tightens at the downstream end of the project to a radius of 175 feet. The radius tightening causes high shear forces and scour on the streambank, eroding the bank toe. As the toe material is eroded, the cohesionless soil above the gravel-sand-silt mixture is unable to resist additional shear forces, and the weight of the soil causes mass-block failures on vertical planes. Channel migration, human alterations, and farming practices have left the existing stream corridor void of vegetation to help resist additional erosion. As a result, lateral channel migration at the project site was 10 to 20 feet per year.

A thorough topographic survey of the project reach was performed with a Topcon GTS–211D total station, equipped with a handheld HP–48 data collector. Survey points were downloaded from the data collector into Eagle Point Civil Design software. The data points and breaklines were used in the CAD environment to generate contours and a base map (fig. CS17–5) used for design and construction drawings. River cross sections were exported to HEC–RAS (USACE 1995a) to create a hydraulic model of the site.

Based on field data and a reach analysis, table CS17–1 lists the physical characteristics of the project site.

The drainage basin for the project site was delineated using U.S. Geological Survey (USGS) maps in ArcGIS. The drainage area was proportioned to a stream gage, located 5 miles upstream of the site, to develop peak discharge flows and recurrence intervals. Based on the gage records, the flows were developed (table CS17–2).

A steady-state HEC–RAS model was developed based on topographic site survey and hydrologic conditions. The model was used to generate hydraulic characteristics of the site, as well as velocity distributions. In addition, the bankfull flow was determined based on physical features from the site survey combined with the HEC–RAS model and peak flow records. A typical velocity distribution cross section from HEC–RAS is shown in figure CS17–6 at bankfull stage.

HEC–RAS is a one-dimensional hydraulic model that does not account for meander mechanics that result from curvature and channel width. The 1991 USACE Engineering Manual (EM) 1110–2–1601 (Engineering and Design – Hydraulic Design of Flood Control Channels) summarizes research showing that the vertical (or spiraling) velocity can exceed the longitudinal stream velocity by more than 35 percent. Therefore, average longitudinal velocities from HEC–RAS are multiplied by 1.5 or 2.0 for design of rock barb structures. This factor of safety accounts for meander effects and turbulent burst velocities.

Knighton (1998) identifies a consistent relationship between meander parameters and channel width (w) where the latter operates as a scale variable of the channel system. The term tortuosity is introduced as an index of the effect of meander geometries on these forces and is defined as the radius of meander curvature (Rc) divided by the channel top width (Rc/w). The channel radius is measured through the meander bend along the thalweg, and the width is taken as the water surface top width at bankfull stage in the uniform section upstream of the meander. Due to the compound nature of the meander bend, the tortuosity of the upstream portion of the project was 3.3, while the downstream end was 1.3. This is significant because when tortuosity is below 3, cross-stream flows become an important consideration for design, in addition to the spiraling, meander-caused flows. This means that flow can impinge on the bank in between barbs, and additional bank protection may be required.

Design

Alternatives such as streambank soil bioengineering with plants and geosynthetics, bank roughness with large wood, and rock structures were evaluated. Based on fluvial geomorphology, hydrology, hydraulics, site survey, and permitting considerations, it was determined that bank shaping, rock barbs, and engineered log structures, in concert with vegetation establishment, would meet project goals. Bank shaping and rock barbs provide immediate stability and reduce hydraulic forces on the bank, thereby allowing vegetation time to grow and establish a solid root system. The vegetal growth, in turn, helps secure long-term stability of the site and enhances the biodiversity of the riparian corridor. Engineered log structures provide immediate habitat for endangered salmonid species and help recruit additional large wood to enhance the stream corridor near the project site.

Figure CS17–5 Existing topographic drawing used for making HEC–RAS model and construction drawings

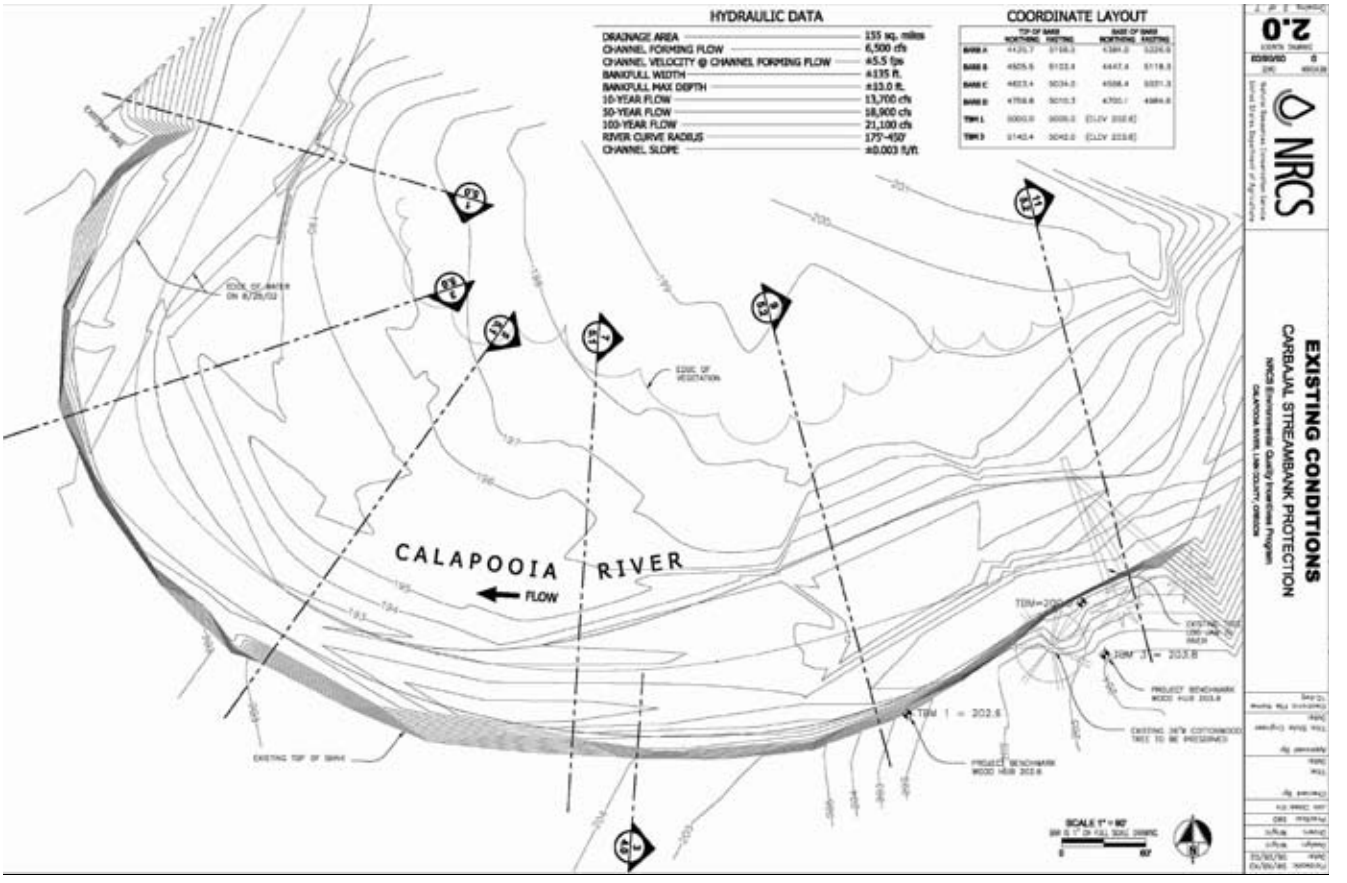


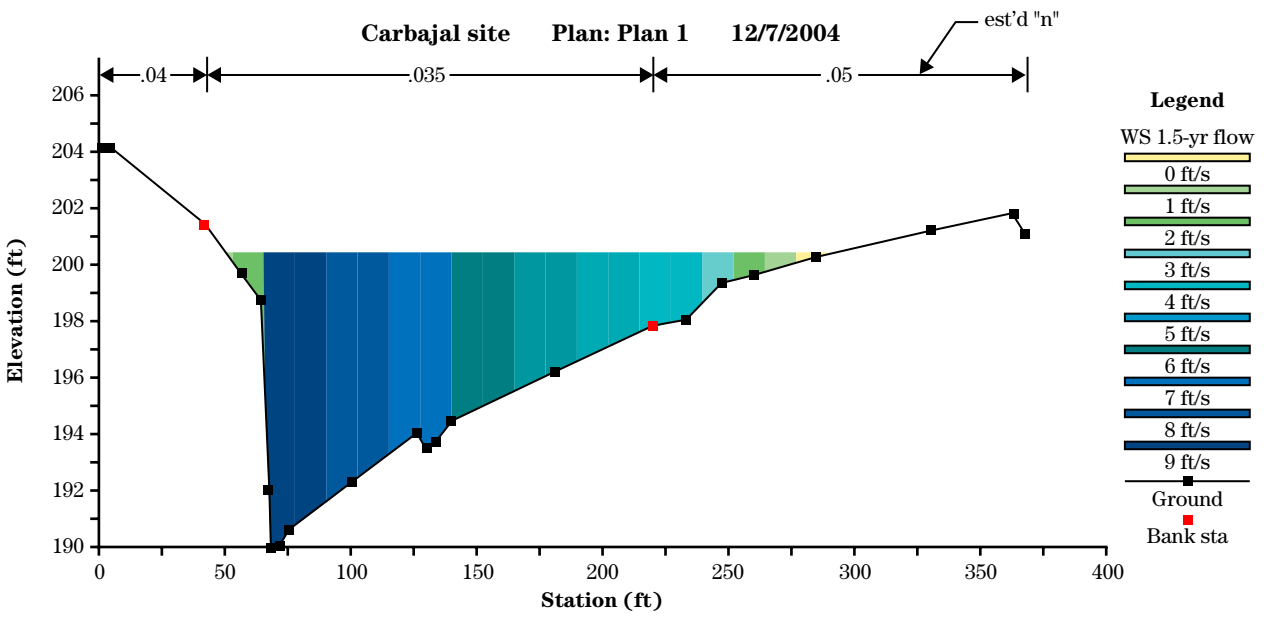
Table CS17–1 Project area characteristics

Reach characteristics	
Drainage area	155 mi²
Bankfull width	135 ft
Sinuosity	1.4
Channel slope	0.003 ft/ft
Historic meander belt width	900 ft
Typical curve radii	450 ft
Meander wavelength	1,600 ft
D_{50}	60 mm (2.4 in)

Table CS17–2 Peak discharge estimates and recurrence interval

Recurrence interval (years)	Peak flow (ft^3/s)
1.5-yr bankfull (estimate)	6,500
2-yr	7,900
5-yr	11,400
10-yr	13,700
25-yr	16,700
50-yr	18,900
100-yr	21,100

Figure CS17–6 HEC–RAS velocity distribution output for a typical cross section



Rock barbs

Barb geometry and rock sizing were done in accordance with Oregon NRCS Technical Note 23, Design of Stream Barbs, which is similar to NEH654 TS14H. Barb rock size and gradation used for construction are summarized in table CS17–3.

The rock barbs were the first component to be constructed. The barbs were staked out using steel "T" posts driven into the riverbed along the design alignment. This practice allowed the contractor to work on the streambank and have a constant view of the proper barb alignment. Figure CS17–7 shows the two downstream barbs immediately after construction and prior to any bank shaping.

Engineered log structures

Engineered log structures were installed at each end of the project to provide immediate fish habitat, provide a mechanism for catching large woody material, and act as anchor points to reduce the erosion potential and reduce the likelihood of flanking the barbs. Logs with rootwads were placed together to form a structure that was ballasted with large rock. The ballast rock was designed using D'Aoust and Millar's (2000) performance-based research which is similar to NEH654 TS14J. These authors state "lateral drag forces do not need to be considered explicitly and the factor of safety against buoyancy can be used as a simple design criterion" for multiple log structures that are tied together. Therefore, a buoyancy calculation was used as the design basis for the log structures (fig. CS17–8).

Figure CS17–7 Looking downstream at initial installation of two barbs before bank shaping, during low summer flows (*Photo courtesy of Scott Wright*)

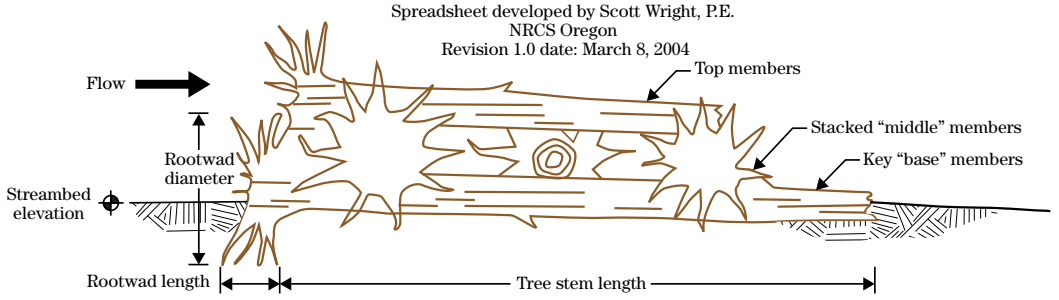
Table CS17–3 Summary of rock gradation used for barb construction

Average gradation blend						
Percent passing	**Diameter (in)**	**Weight (lb)**	**Percent of sample**	**Weight to make sample (lb)**	**Average rock weight (lb)**	**Number of rocks (ea)**
93	48	8,064				
			25.0	178,750	5,733	31.2
68	36	3,402				
			27.5	196,625	2,205	89.2
40	24	1,008				
			17.5	125,125	717	174.6
23	18	425				
			15.0	107,250	276	389.1
8	12	126				
			7.5	53,625	63	851.2
0	0	0				

Figure CS17–8 Typical spreadsheet for calculating factor of safety against log structure buoyancy

Buoyancy calculations for engineered log jam

Spreadsheet developed by Scott Wright, P.E.
NRCS Oregon
Revision 1.0 date: March 8, 2004



Key "base" members

Number of logs with rootwads	N_L =	4	
Specific gravity of large wood	S_L =	0.45	
Average rootwad diameter	D_{RW} =	8	feet
Average rootwad length	L_{RW} =	3	feet
Proportion of voids in rootwad	p =	0.45	decimal
Tree stem average diameter	D_{TS} =	2	feet
Tree stem average length	L_{TS} =	20	feet

$$\% \; F_{BL} = \left(\frac{\pi D_{TS}^2 L_{TS}}{4} + \frac{\pi D_{RW}^2 L_{RW}}{4} \times (1-p) \right) \times \rho_W g (1 - S_L) \times N_L$$

F_{BL} = 20,008 pounds

Stacked "middle" members

Number of logs with rootwads	N_L =	2	
Specific gravity of large wood	S_L =	0.45	
Average rootwad diameter	D_{RW} =	6	feet
Average rootwad length	L_{RW} =	2	feet
Proportion of voids in rootwad	p =	0.45	decimal
Tree stem average diameter	D_{TS} =	2	feet
Tree stem average length	L_{TS} =	30	feet

$$\% \; F_{BL} = \left(\frac{\pi D_{TS}^2 L_{TS}}{4} + \frac{\pi D_{RW}^2 L_{RW}}{4} \times (1-p) \right) \times \rho_W g (1 - S_L) \times N_L$$

F_{BL} = 8,602 pounds

Top members

Number of logs with rootwads	N_L =	2	
Specific gravity of large wood	S_L =	0.45	
Average rootwad diameter	D_{RW} =	6	feet
Average rootwad length	L_{RW} =	2	feet
Proportion of voids in rootwad	p =	0.45	decimal
Tree stem average diameter	D_{TS} =	2	feet
Tree stem average length	L_{TS} =	20	feet

$$\% \; F_{BL} = \left(\frac{\pi D_{TS}^2 L_{TS}}{4} + \frac{\pi D_{RW}^2 L_{RW}}{4} \times (1-p) \right) \times \rho_W g (1 - S_L) \times N_L$$

F_{BL} = 6,446 pounds

Boulder ballast

Specific gravity of boulders	S_S =	2.65	
Diameter of boulder	D_B =	3.5	feet
Number of boulders unsubmerged	N_{BU} =	0	
Number of boulders fully submerged	N_{BS} =	24	

$$W' = \frac{\pi D_B^3 L_{BS}}{6} \times \rho_W g (S_L - 1)$$

W = 3,712 pounds per boulder unsubmerged
W' = 2,311 pounds per boulder submerged

Total weight for all boulders (submerged and unsubmerged) = **55,469** pounds

Factor of safety: buoyancy

A simplified approach is used to estimate buoyancy where the logs and ballast boulders in the logjam are fully submerged. In addition, the logjam and boulders act as a composite structure and are assumed fully connected. Water velocity inside the logjam is highly turbulent and near zero, therefore, vertical uplift forces are assumed negligible.

$$FS_B = \frac{W + W'}{\sum F_{BL}}$$

= 1.58 **FS_B = 1.58**

D'Aoust and Millar (2000) recommend a minimum factor of safety of 1.5 against buoyancy for log structures. Based on experience using log structures in Oregon, this minimum factor of safety against buoyancy is an adequate design parameter. However, it is recommended that the factor of safety be closer to 2.0 and that the large wood be connected together to allow the structure to act as a single unit. These connections also provide better stability in the structure for placing ballast material. The higher factor of safety also allows for greater flexibility during construction when working with imperfect and irregular logs.

Bank shaping and vegetation

The existing bank consisted of noncohesive material and was geotechnically unstable. Therefore, the bank was shaped and excavated from the summer low-water elevation to the catch point of the existing ground at a 3H:1V slope. This slope creates a stable bank and provides an optimal surface to plant vegetation. Annual grass seed was planted, along with a 3-year, degradable erosion control blanket. The blanket provided immediate stabilization of the soil and exposed bank until the vegetation could establish (fig. CS17–9). The erosion control blanket had a permissible shear stress of 2.25 pounds per square foot that easily exceeded the 10-year flow maximum shear stress of 1.2 pounds per square foot predicted in the HEC–RAS model.

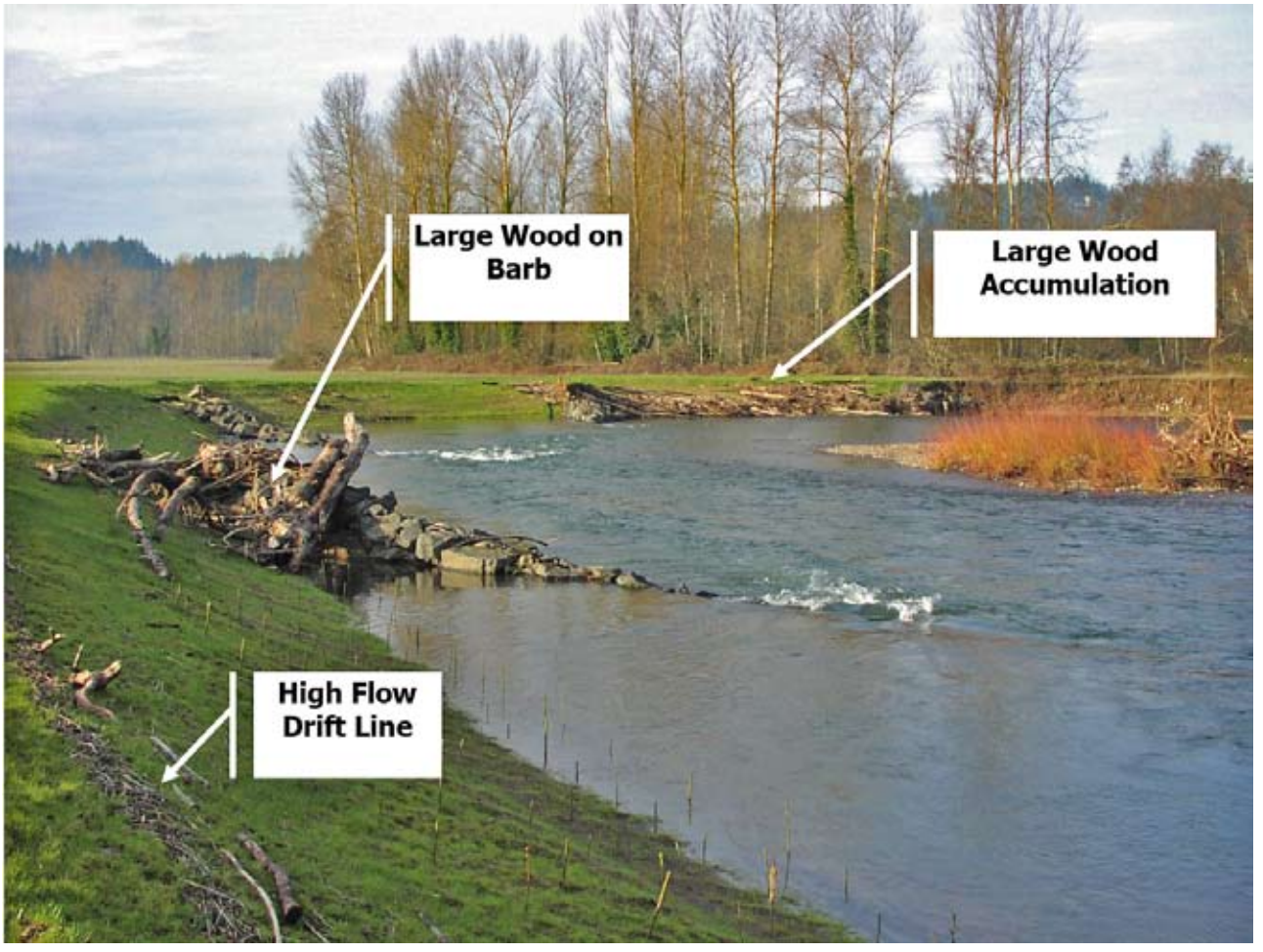
Performance

Just 3 months after project completion, a gaged storm event occurred that measured between a 5- and 10-year peak flow. The project withstood the storm event with no noticeable erosion or adverse effects to the surrounding area (fig. CS17–10). Large amounts of wood collected on top of each barb and especially near the downstream third of the meander bend—at the engineered log structure (fig. CS17–11).

Figure CS17–9 Looking downstream at completed project showing barbs and erosion control mat during summer flow (*Photo courtesy of Scott Wright*)

Figure CS17–10 Winter baseflow with planted willows and grass

Figure CS17–11 Looking downstream at completed project area after 10-year storm event (*Photo courtesy of Scott Wright*)



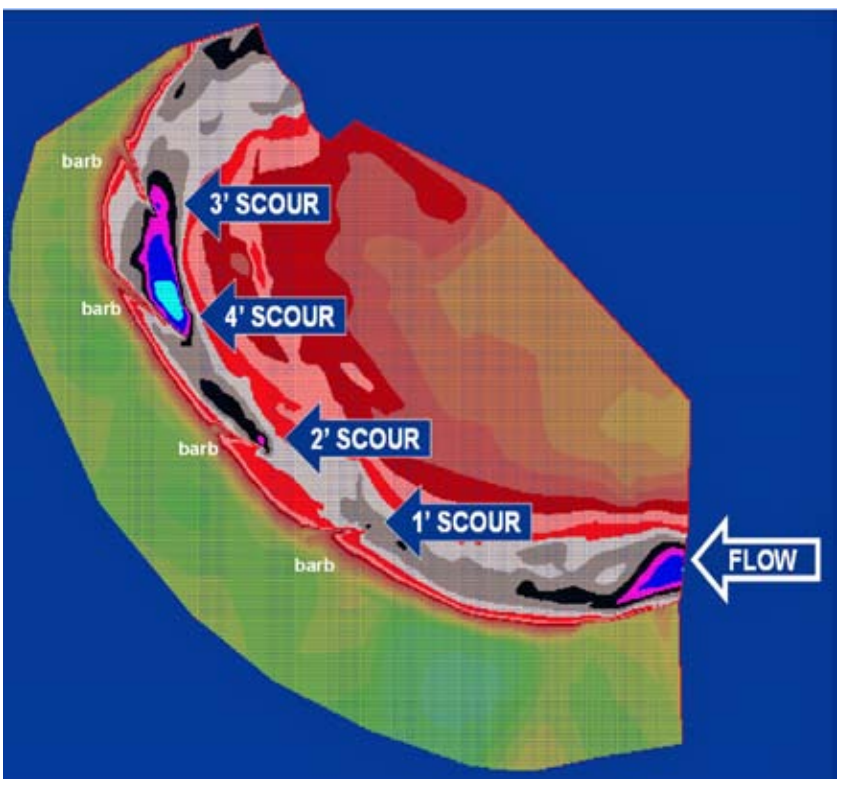
Detailed topographic information was collected pre- and postconstruction to identify actual geomorphic effects of the stream barbs and overall performance. Figure CS17–12 identifies the actual scour that occurred around each barb and the streambed.

Because of the barbs, no scour or erosion occurred along the outside bank of the meander. As illustrated in figure CS17–12, the hydraulic effect of the barbs caused local scour and constriction scour. The scour pattern begins around the tip of the barb and extends downstream in an elliptical shape. This pattern is similar to other observations made in Oregon around barb groups on C3 and C4 gravel-bed rivers.

Energy dissipation within the project reach is caused by scour and a hydraulic jump at each barb. Figure CS17–13 shows the distinct hydraulic jump as water flows over the barb. This jump is progressive with stage because of the crest slope of the barb weir. Based on this project and several other observations of barbs, a 15H:1V slope appears to be an optimal weir slope to enact the hydraulic jump throughout various discharge stages.

The barbs reduced near bank flow velocities, created scour areas that enhanced fish habitat, provided reach diversity, collected large wood, and dissipated hydraulic energy within the project reach without translating

Figure CS17–12 Actual scour around each barb following 5- to 10-year peak flow event. Each color band gradient represents a 1-foot elevation increment. (*Photo courtesy of Scott Wright*)



erosion problems downstream. Figure CS17–14 shows the completed project 3 years after construction. A significant number of willows now grow in the reach corridor and further reduce near bank flow velocities. The vegetation provides habitat to promote biodiversity that was not present in the preproject state.

Cost

The stabilization techniques included four rock stream barbs, two engineered log structures, bank excavation and shaping, and an erosion control blanket. The protected length of streambank was approximately 900 feet with a construction cost of $60,000. The cost included mobilization, materials, installation of all structures, and final clean up. All excess soil from bank shaping was disposed of onsite, and the project was easily accessible with machinery. Rock for the barbs was transported in standard dump trucks from a quarry 25 miles from the site. Large wood for the structures was purchased and transported to the site. The landowner provided all materials, labor, and supplies for the willow and riparian buffer plantings.

Summary

After three winters and a 5- to 10-year peak flow event, the project has performed well and exceeded landowner expectations. Biologists and regulatory agencies are pleased by project performance and the much improved habitat and species diversity. An ongoing research study will provide quantitative data on the biological impacts of stream barbs on the riverine environment.

Figure CS17–13 Example of energy dissipation from a hydraulic jump formed over one of the project barbs during winter baseflow (*Photo courtesy of Scott Wright*)

Figure CS17–14 Looking downstream 3 years after project completion with extensive vegetation growth (*Photo courtesy of Scott Wright*)

Case Study 18 Wiley Creek, Sweet Home, Oregon

Issued August 2007

Cover photo: Embankment treatment on Wiley Creek in Oregon

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.

Case Study 18 Wiley Creek, Sweet Home, Oregon

By Sean Welch, state hydraulic engineer, U.S. Department of Agriculture, Natural Resources Conservation Service, Portland, Oregon

Introduction

The Wiley Creek Streambank Protection Project in Linn County, Oregon, was designed in 2003 and 2004 by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), Oregon State Office. The project goals included the protection of two structures located 5 feet from the edge of a 23-foot-high vertical bank, bank stabilization, and fisheries habitat improvement.

The project was constructed in summer 2004 and consisted of a 180-foot-long reinforced earth embankment protected by three engineered log jams (ELJ) and two stream barbs. Bankfull discharge was determined at approximately 3,200 cubic feet per second with a 100-year discharge of more than 12,000 cubic feet per second. The project demonstrates the use of geosynthetic reinforced earth fills and soil bioengineering techniques for bank stabilization in a high-energy river system. Additionally, the project provides a demonstration of infrastructure and bank protection methods that achieve Endangered Species Act regulatory considerations through creation and enhancement of salmonid habitat. The project was constructed for $107,000 under the NRCS Environmental Quality Incentives Program (EQIP). Figures CS18–1 and CS18–2 show the preproject bank condition and 1 year following construction.

Background

The Wiley Creek Streambank Protection Project consisted of stabilizing and creating fish habitat along approximately 180 linear feet of streambank and the protection of two buildings. The project is located near Sweet Home, Oregon, along Wiley Creek, a tributary to the Santiam River, which flows to the Willamette River. Federally listed steelhead and Chinook salmon use the project reach of Wiley Creek for spawning and rearing habitat, which necessitated environmentally sensitive engineering design, more stringent permitting requirements, and additional implementation considerations. The preproject site consisted of a 23-foot-high vertical

Figure CS18–1 Preproject riverbank along Wiley Creek, December 2003. Note location of buildings at the top of bank.

Figure CS18–2 Completed reinforced earth embankment, stream barb, and bank vegetation 1 year following construction

bank with two structures approximately 5 feet from the top edge of the bank (fig. CS18–1). Anecdotal information from the landowner and analysis of historical photographs indicated that the river's left bank had eroded more than 40 feet since the rain-on-snow flood event of 1996 (fig. CS18–3).

Design objective

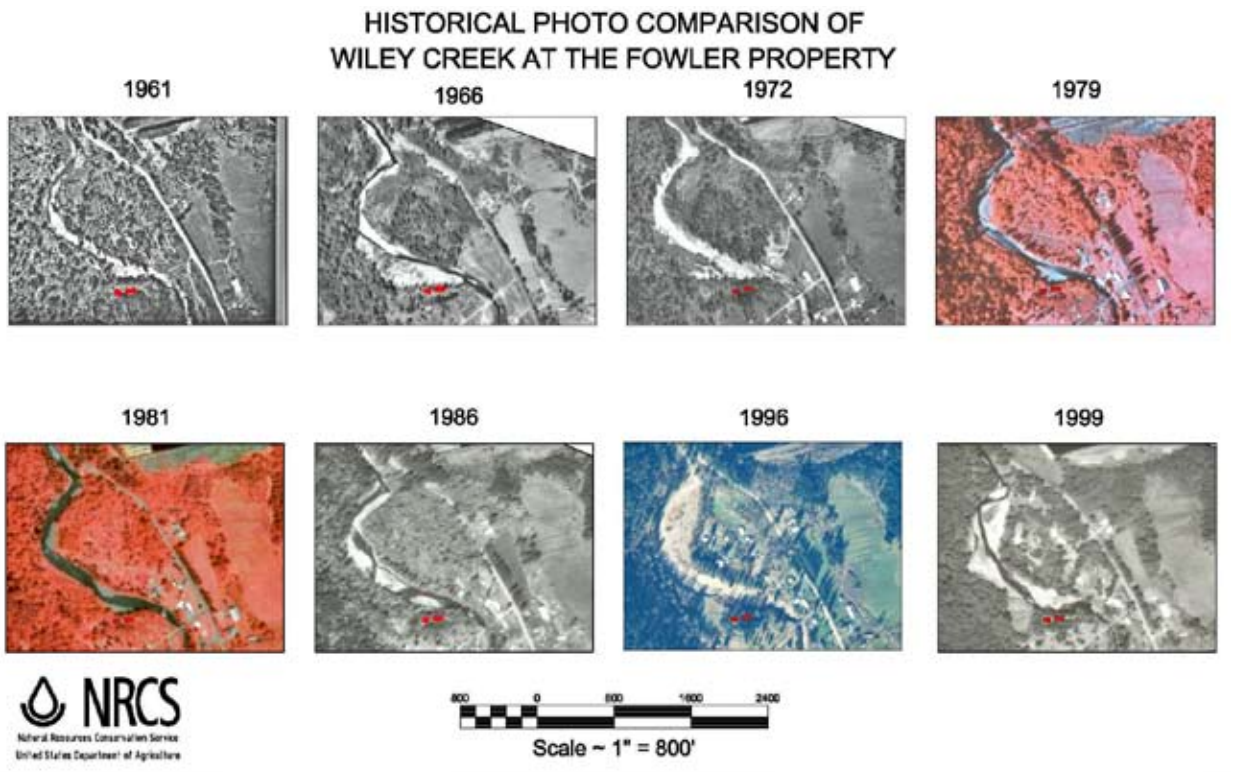
Design objectives included protection of two streamside structures, stabilization of the eroding left bank, and enhancement of salmonid habitat along Wiley Creek through the project reach. Additional considerations required no significant increase in the preproject flood elevations and implementation between July 15 to September 30 during the Oregon Department of Fish and Wildlife's instream work window.

The project design incorporated a reinforced earth embankment consisting of thirteen 2-foot soil lifts, encapsulated with geotextile-geogrid that extended from the toe of the eroded bank to the top of the bankline. Scour and erosion protection of the embankment was provided through the construction of two stream barbs and four ELJs.

Geomorphology

Watershed condition has changed dramatically within the Willamette Basin in the past century, and Wiley Creek is no exception. Many of the streams in the western Cascades were splash-dammed to transport logged timber downstream to receiving lumber mills. This activity had a significant effect on geomorphic condition of the rivers and streams and a severe im-

Figure CS18–3 Historical aerial photographs of Wiley Creek along the project reach showing morphologic changes over time



pact on instream habitat and biodiversity. The extensive timber-cutting in the watersheds also modified the magnitude, timing, and duration of the hydrograph, along with increased sediment production and transport processes. The contributing watershed area at the project location is 57 square miles, with nearly 3,700 feet of watershed relief.

The Wiley Creek project site is located within a transitional morphologic reach of Wiley Creek. The upstream reach is narrowly confined, has low sinuosity, and is bedrock controlled. The Rosgen stream classification (Rosgen 1994) for this reach appears to be B1c. Minimal sediment deposition occurs within this reach, except for a few areas along the active channel margins. The reach is hydraulically smooth and, with the exception of a few boulders, is scoured to bedrock (fig. CS18–4).

Wiley Creek transitions abruptly from this transport-dominated reach over an 8-foot-high bedrock overfall ledge to an over-widened depositional reach. This section of Wiley Creek is adjacent to the project and is characterized by distributary flow and a large mid-channel willow dominated bar. The excessive sediment deposition in this reach resulted in an anastomosed pattern, forcing the channel against the river's left streambank adjacent to the project. This reach was classified as a Rosgen D4 stream type (Rosgen 1994) (fig. CS18–5). The project bank is located in the trees on the right side of the photo. Note the variable pattern and excessive sediment deposition. The bedrock overfall is immediately upstream, just beyond the limits of the photo.

A topographic survey was performed through the composite stream reach and was used for the geomorphic analysis and as base information for the hydraulic and geotechnical modeling. Survey data were collected by transferring georeferenced control points to the project area with a Topcon Survey Grade Global Positioning System. The topographic survey was performed using a Topcon total station and reduced in Eagle Point software. The project site map is shown in figure CS18–6.

Hydrology

Hydrologic analysis of Wiley Creek was performed using the U.S. Army Corps of Engineers (USACE) software program, HEC–FFA CPD–59 (formerly known as HEC–WRC) (USACE 1992). The flood frequency analysis is based on the methods present within Bulletin 17B guidelines of the U.S. Water Resources Council. Two gages were analyzed including USGS# 14187100, Wiley Creek at Foster, Oregon, and USGS# 14187000, Wiley Creek near Foster, Oregon. The two gages did not contain sequent records, which necessitated the use of watershed areal weighting to adjust discharge values for a composite record. Results of the flood frequency analysis are provided in table CS18–1.

Table CS18–1 Flood frequency analyses

Return period (yr)	Flow rate (ft^3/s)
2	3,251
10	6,111
25	7,437
50	8,365
75	9,243
100	12,092

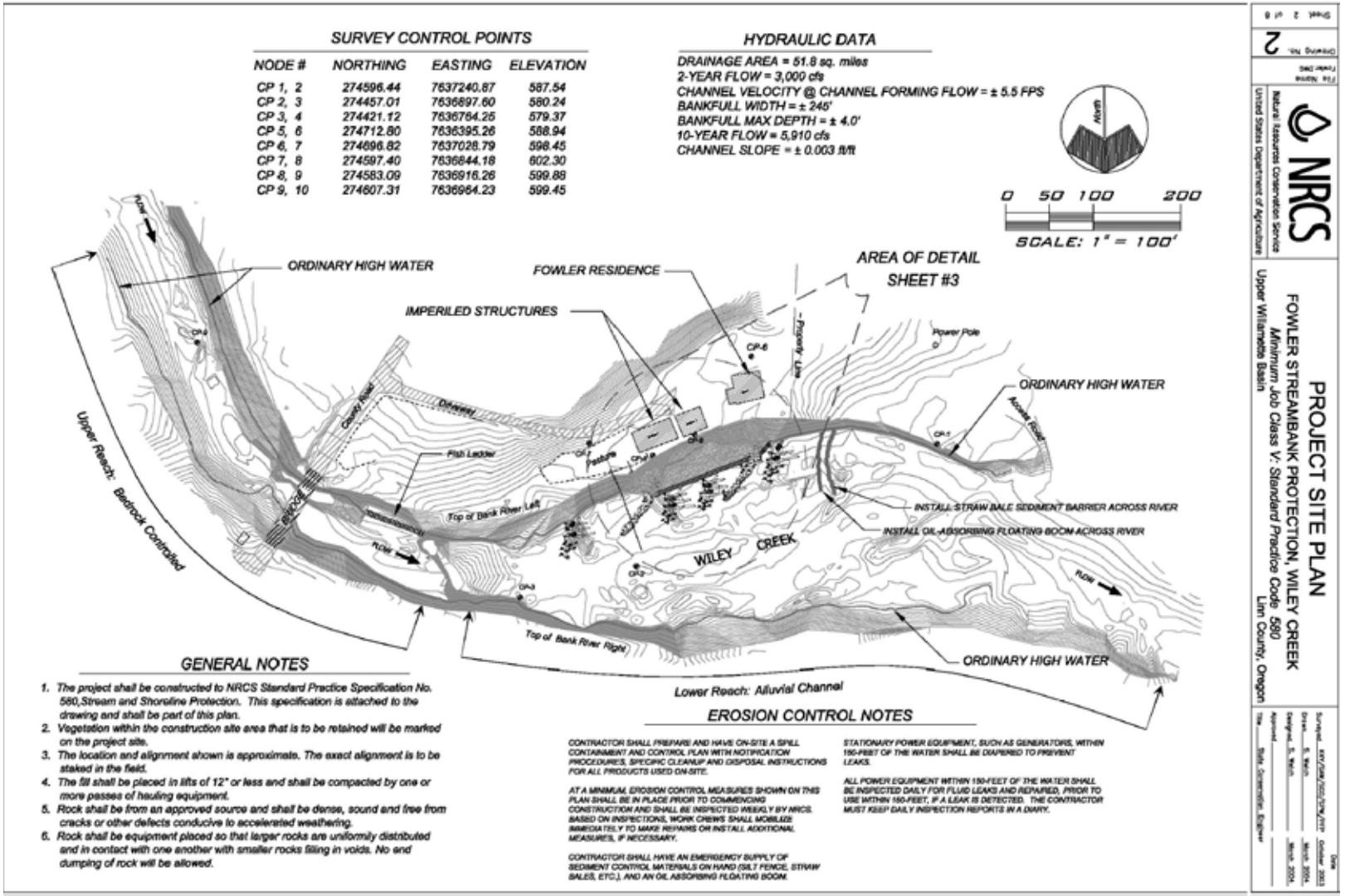
Figure CS18–4 (a) Looking upstream from the Wiley Creek Bridge at the B1c reach above the project; (b) looking upstream to the bedrock overfall ledge. The Wiley Creek Bridge can be seen in the background. This location marked the transition from the B1c to D4 reach.

(a)

(b)

Figure CS18–5 Looking upstream at the D4 reach

Figure CS18–6 Topographic site map of the Wiley Creek Project



Reach scale hydraulics

Wiley Creek was modeled with the USACE's River Analysis System (HEC–RAS) (USACE 1995a), using the topographic survey data as base information. The geometric data model included sections, reach lengths, and overbank stations and was developed in AutoCAD and exported to HEC–RAS for hydraulic analysis. Information obtained from the HEC–RAS model included average velocity, shear stress, stream power, and a reach length water surface and energy grade profiles at discharges ranging from the 2-year to 100-year flood. Velocity distribution output using the ArcView HEC-GeoRAS extension is shown in figure CS18–7.

The hydraulic model extended from the upstream-bedrock-dominated B1c channel, across the bedrock overfall, and through the high width-depth ratio D4 channel adjacent to the project. Model results were used to interpret reach-scale sediment transport processes by identifying areas of high hydraulic stress and depositional potential through the transitional channel morphology. Large energy losses were computed across the bedrock overfall that defined the break between the upstream transport dominated reach and the depositional project reach. The mixed flow regime was used to compute subcritical and supercritical water surface profiles including the large hydraulic jump at the bedrock overfall (fig. CS18–8).

Figure CS18–7 Quasi, two-dimensional velocity distribution for the 2-year flood computed by HEC–RAS. Contours and model cross sections (black lines) are also shown.

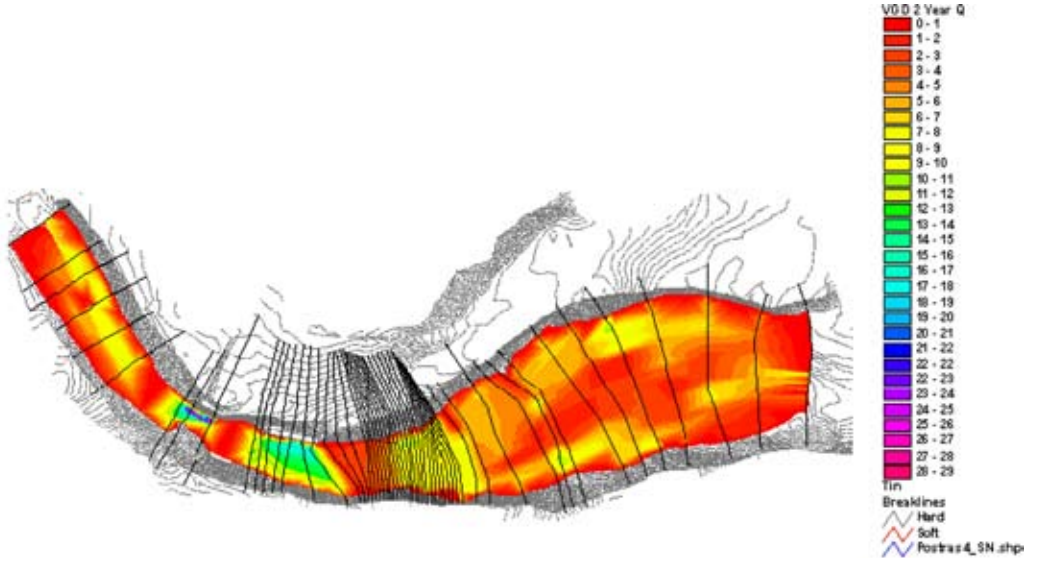
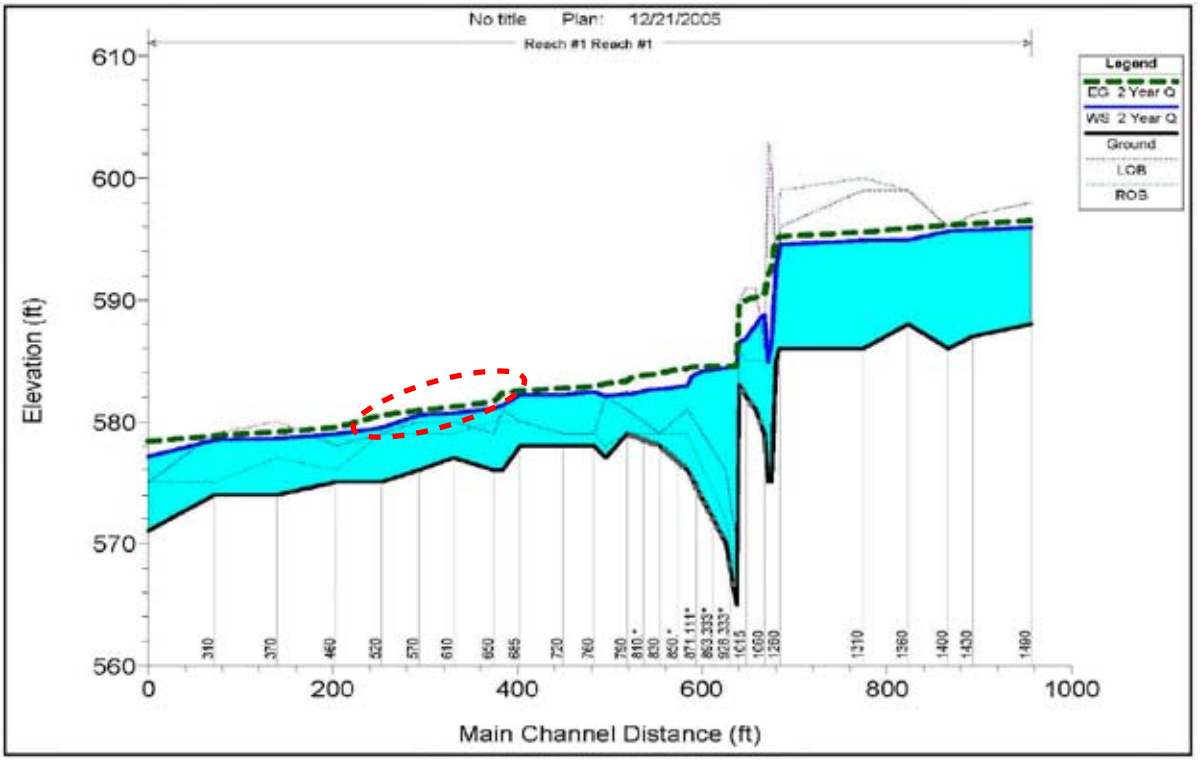


Figure CS18–8 Computed water surface profile and energy grade line for the 2-year flood. Note hydraulic jumps between sections 1260 and 928 across the bedrock overfall upstream of project reach. Project reach is defined by red oval.



Geotechnical design

During the initial site reconnaissance, the bank condition was evaluated for both hydraulic and geotechnical stability. The bank had eroded to a near vertical condition and was well beyond the stable angle of repose (fig. CS18–9). Bank stratigraphy consisted of poorly consolidated alluvium (fig. CS18–10). The buildings at the top of bank were an additional destabilizing factor as point loads. The dominant bank failure mechanism was hydraulic stress undercutting the bank with subsequent tension-block failure of the overburden. Rapid drawdown of the saturated soils and positive pore water pressure within the bank also contributed to instability.

The combination of hydraulic stress, low strength of the earth materials, and loading condition at the top of the bank required a design that would provide free-draining support to the bank, while resisting hydraulic stresses. Preliminary alternatives were identified that included an out-sloped embankment with a rock-reinforced toe or a structural fill section using cellular confinement or reinforced earth.

Reinforced earth combined with soil bioengineering techniques was chosen based on proven transportation applications, ease of permitting, and ability to incorporate habitat enhancement features. Two references provided the technical basis for the embankment design:

- Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines (U.S. Department of Transportation Federal Highway Administration (FHWA) (2001c)
- Forest Service Retaining Wall Design Guide (USDA Forest Service 1994)

These references provided two methods for determining the required geogrid, lift height, and tendon lengths for the reinforced earth embankment based on user-supplied geotechnical information. Additional information regarding these features is provided in technical supplements 14D and 14I of this handbook. Figures CS18–11 and CS18–12 show output from the FHWA RSSA (FHWA 2001c) program (companion software to Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines) that was used to analyze multiple water table and loading conditions for internal and global embankment stability.

The program solves the modified Bishop's method for bank stability for a user-provided factor of safety assuming both linear and rotational failure planes. The

Figure CS18–9 Looking upstream at the project bank. Less than 5 feet of bank remained from the building to the edge.

Figure CS18–10 Instream view of project bank. Note vertical condition and poorly consolidated sandy-silt alluvium bank material.

Figure CS18–11 RSSA model showing bank materials, loading, and computed tendon configuration for a mid-bank water table condition

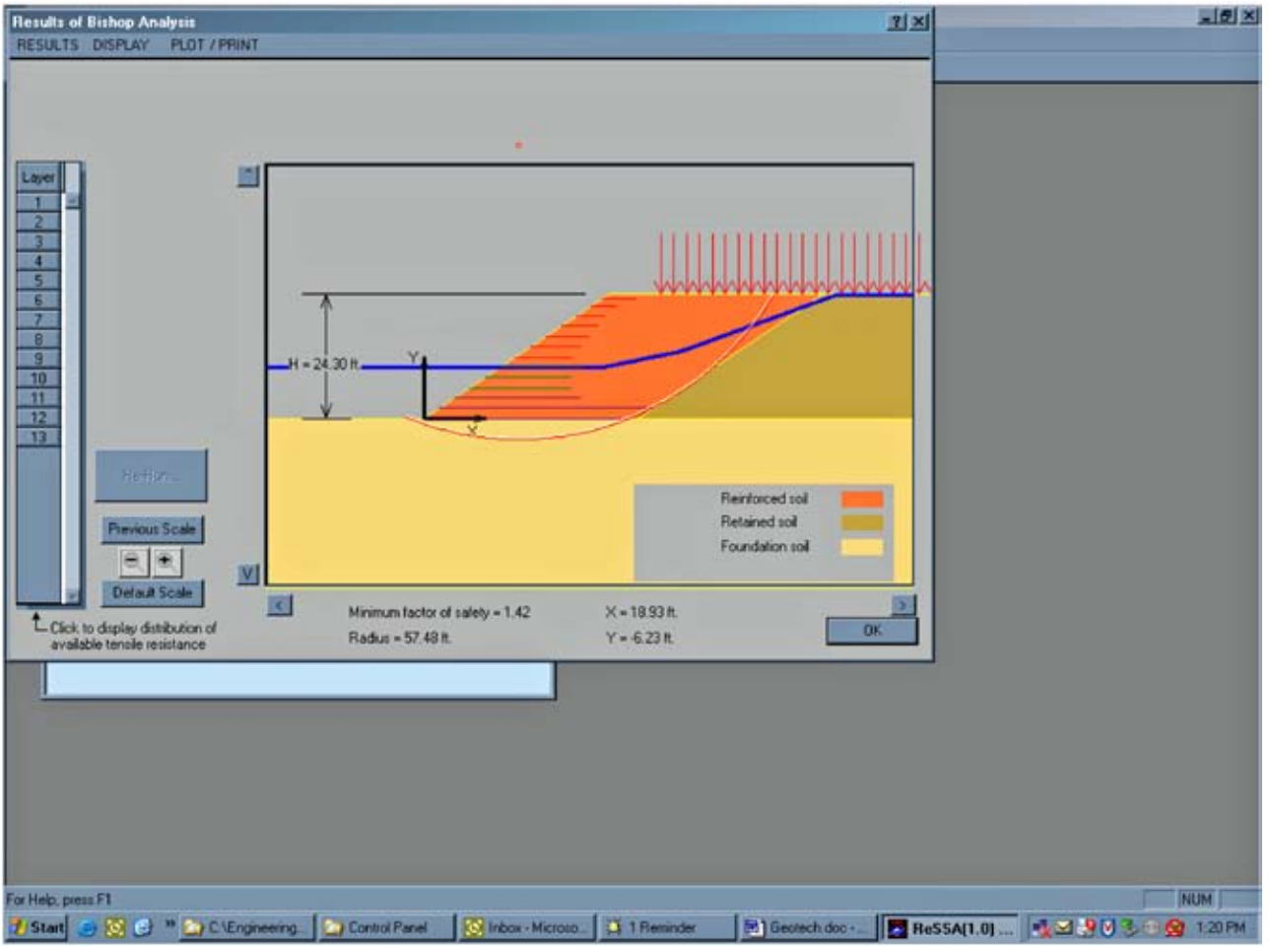
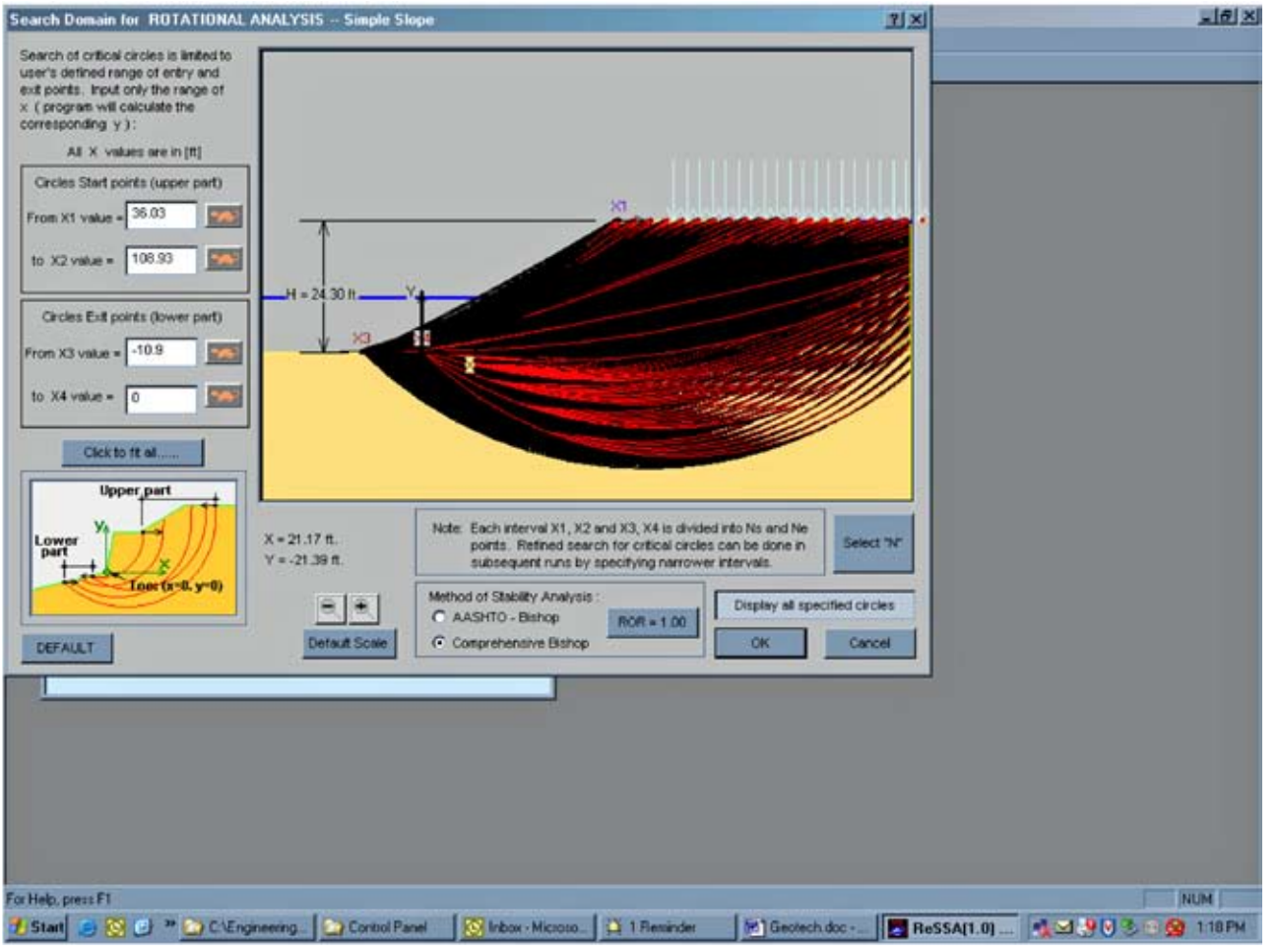


Figure CS18–12 Bishop slices showing optimization results for rotational bank failure



program optimizes on these two failure scenarios and computes required geogrid tendon lengths based on a user-provided elevation schedule.

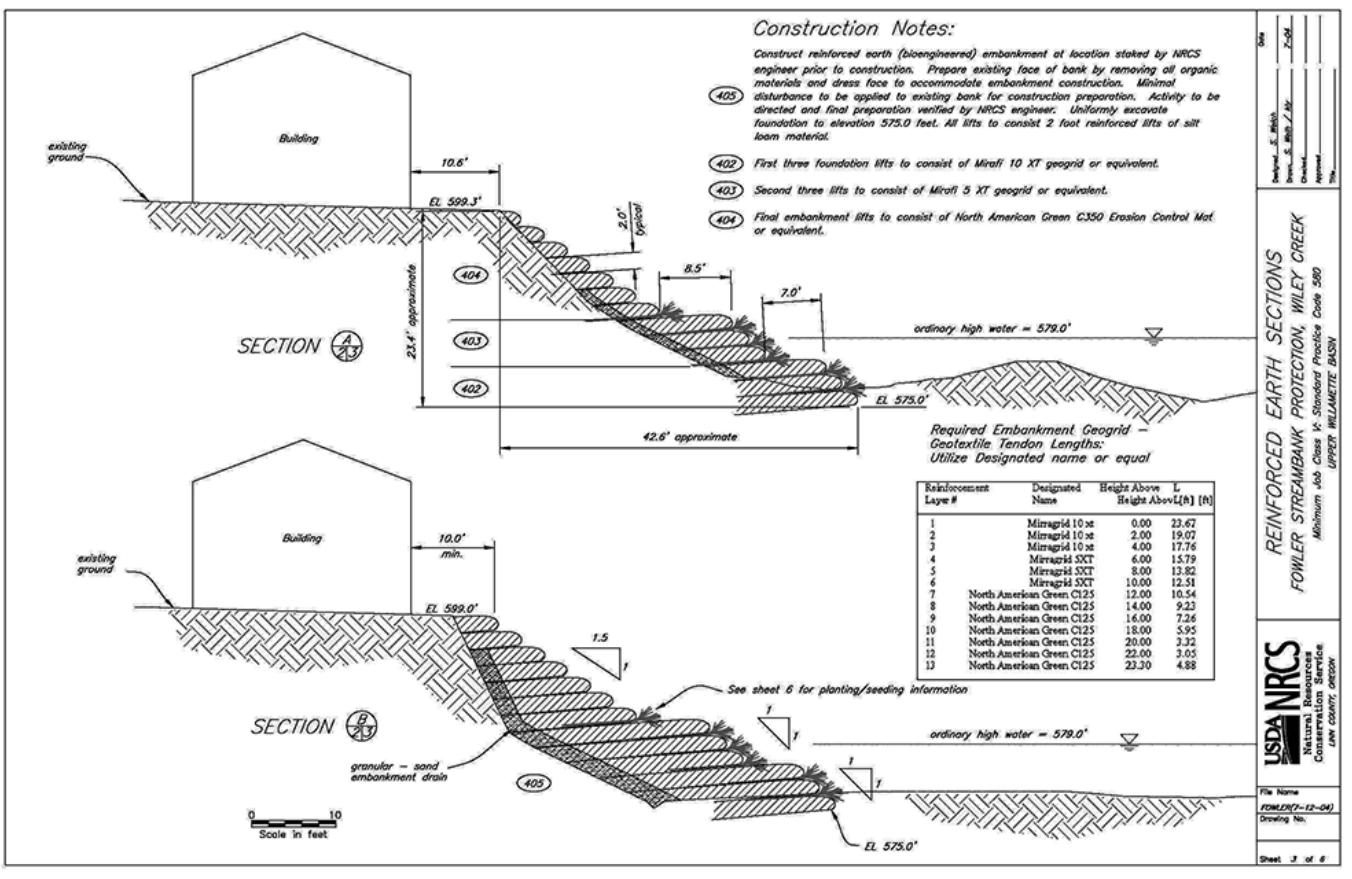
Tendon materials were chosen based on tensile strength, cost, and manufacturer's recommendations for the given condition. Lift design consisted of 2-foot compacted silt loam soil reinforced with a woven geogrid, and faced with a long-term erosion control fabric. The design also included a filter drain at the interface between the pre-project bankline and the imported material.

Vegetation design

The Aberdeen, Idaho, and Corvallis, Oregon, NRCS Plant Materials Centers were consulted for specifications on the appropriate vegetative components for the project. Increased boundary roughness using vegetation was critical for reduction in near-bank shear stress and velocity reduction along the face of the constructed embankment. Vegetation components were based on a hydric-to-mesic compositional transition from the base-flow elevation to the top of the top of bank. Native willow (*Salix lucida Muhl* ssp. *lasiandra*) was abundant at the project location and was harvested and placed between the embankment lifts. The embankment was protected by placing complete willow clumps along the toe-of-slope per NRCS PMC TN–42, Willow Clump Plantings and NRCS PMC TN–23, How to Plant Willows and Cottonwoods for Riparian Rehabilitation.

Figure CS18–13 shows construction documentation (section view) of the reinforced earth embankment, with the tendon schedule and willow placement within the lower lifts. The embankment drain is also shown at the original section-design section interface.

Figure CS18–13 Section drawings of reinforced earth embankment



Hydraulic design

Stability of the embankment required near-bank hydraulics to be controlled to threshold values less than the permissible maximum. The silt-loam embankment material could be readily entrained at velocities between 3 to 5 feet per second, even under optimum compaction. However, with appropriate measures, it was recognized that the geotechnical design was feasible. Methods used to reduce the near-bank shear stress included an increase in boundary roughness and large-scale roughness through the use of aggressive revegetation and ELJs, and flow redirection using stream barbs and ELJs.

Three ELJs were constructed using design methods presented by D'Aoust and Millar (2000). This information is similar to that presented in NEH654 TS14J. Their criteria are based on the systematic review and analysis of 90 constructed projects in western Canada, and they recommend a minimum factor of safety against buoyant forces on the ELJ structure of 1.5 or greater. Oregon NRCS uses this design analysis methodology, but does not use cable for connecting ballast to the log members. Based on experience and regulatory considerations, it is found that bolting the ELJ members together creates a composite structure and allows for competent framework for the rock ballast. Additional research in the Northwest has shown the habitat benefits of incorporating large wood in streambank protection projects for salmonid velocity refugia, cover, diversity complexity, and macroinvertebrate production.

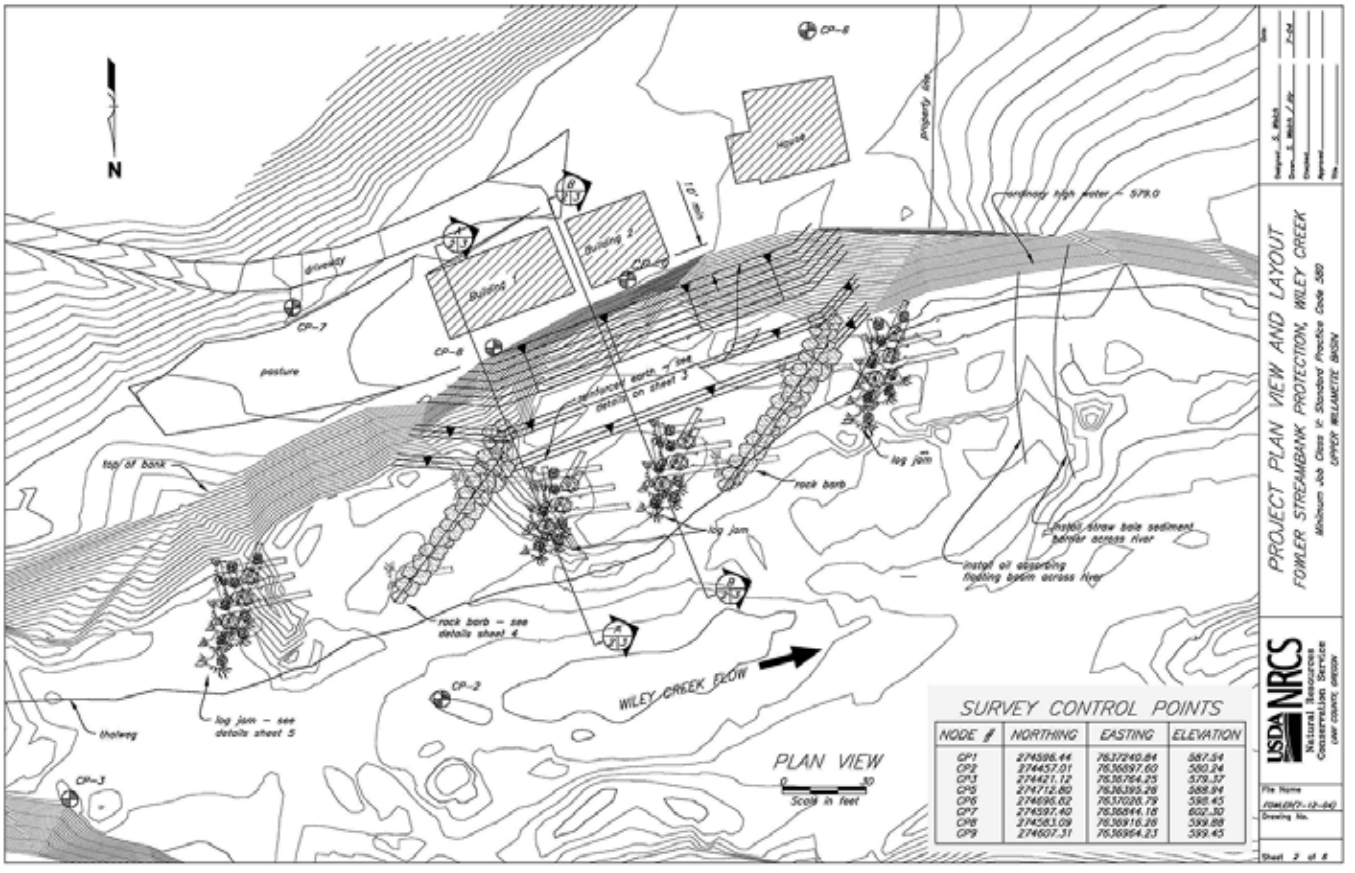
Two stream barbs incorporating large wood were used for hydraulic control at the upstream and downstream ends of the reinforced earth embankment. Barbs are a proven technology for near-bank velocity reduction and bank protection. NEH654 TS14H provides design guidance for these structures including geometric design, spacing-layout, and rock sizing criteria. Figure CS18–14 shows construction of an ELJ and stream barb, while figure CS18–15 shows the layout all of the project components.

Figure CS18–14 (a) Construction of the upstream ELJ. A temporary cofferdam was placed to dewater the construction area; (b) construction of one of the project stream barbs. Photo is taken looking up the axis of the barb structure.

(a)

(b)

Figure CS18–15 Plan view layout of reinforced earth embankment, ELJs, and stream barbs



Construction: project cost

Contractor selection was performed by the landowner and although the selected company had limited instream construction experience, implementation progressed on time and within schedule. Design, permitting, and construction management were provided by the NRCS. Due to the presence of threatened and endangered salmonid species within many Northwest rivers, most states, including Oregon, have designated periods when instream work can be performed. This process requires all state and Federal permits be acquired before the limited construction window including endangered species act consultation, if required.

All equipment that operated instream was required to be cleaned and leak free with a spill management plan available from the contractor. Project equipment included: one D6 bulldozer, one 130-horsepower excavator, a front–end loader, and three 12- to 14-cubic yard dump trucks hauling fill material on a constant rotation. Total project cost was $107,000 including all construction labor and materials. The cost estimate and quantities of materials are shown in figure CS18–16.

Construction began with an access road to the bottom of the project bank and placement of a temporary cofferdam to divert flow from the project site. With site preparation complete, materials were delivered including large wood, rock, geotextiles, and embankment fill. The following list identifies the progression of project elements during construction:

- construction of the downstream stream barb
- foundation preparation and construction of the reinforced earth embankment
- construction of the upstream stream barb
- completion of the embankment
- construction of the four engineered log jams
- vegetation planting at toe of embankment and around large wood structures.
- vegetation planting of the embankment

Figures CS18–17 through CS18–21 document construction of the primary project components.

Figure CS18–16 Engineer's cost estimate and materials estimate for the Wiley Creek project

ITEM	ESTIMATED QUANTITY	UNIT	UNIT COST	ESTIMATE	
EROSION CONTROL & GEOTEXTILES					
NAG "C350" Erosion Control Mat, installed	1800	yd^2	$ 5.50	$ 9,900	550 yds^2 lifts, 350 yds^2 caps (Cost from NAG)
Mirafi soil reinforcement grid 10XT, installed	1200	yd^2	$ 5.00	$ 6,000	180' long x (32+27+26) tendon, cap & wrap
Mirafi soil reinforcement grid 5XT, installed	900	yd^2	$ 3.00	$ 2,700	180' long x (24+22+20.5) tendon, cap & wrap
Temporary Erosion Control	1	lump sum	$ 1,500.00	$ 1,500	
				$ 20,100	
ROCK & LARGE WOOD					
Large boulders delivered					
(large ELJ = 3, 6' & 8, 4' - small ELJ = 3, 6' & 4, 4')	112.5	ton	$ 21.00	$ 2,363	mid-valley gravel pit (NRCS Basin Engineer)
Large boulders delivered & installed in barb (avg 3' diam)	600	yd^3	$ 45.00	$ 27,000	mid-valley gravel pit (NRCS Basin Engineer)
Large Wood log jams installed	4	each	$ 5,000.00	$ 20,000	
				$ 49,363	
EARTHWORK & VEGETATION					
Compacted earth fill delivered & installed	3250	yd^3	$ 10.00	$ 32,500	mid-valley gravel pit (Cost from S.Wright)
Vegetation					
Willow Avg. 5.5'	2800	ft.	$ 0.20	$ 3,080	Salem SWCD
Red Osier Dogwood 5'	230	each	$ 0.20	$ 230	Salem SWCD
Potted Plants	455	each	$ 3.50	$ 1,593	Salem SWCD
Columbia Sedge	920	each	$ 0.60	$ 552	Salem SWCD
				$ 5,455	
			TOTAL ESTIMATE =	$ 107,417	

Figure CS18–17 (a) Access road was constructed to allow haul trucks to drive onto each lift, dump fill material, and provide compaction; (b) fill material was spread uniformly with a dozer.

(a)

(b)

Figure CS18–18 (a) First soil lift on top of the base foundation geogrid. Portion of upstream stream barb is in foreground, and downstream barb is seen in distance. Geogrid extending from the soil is wrapped over to encapsulate the lift after compaction and grade have met specification; (b) Grade was checked at multiple locations on each soil lift. Base course geogrid can be seen underlying fill material.

(a)

(b)

Figure CS18–19 (a) First lift is complete for grade and compaction, and geogrid has been wrapped and staked. Lift is being faced with erosion control fabric to minimize soil piping and reduce photo-degradation of the geogrid tendon. (b) First lift is completed, and willow cuttings are being placed. Willows were harvested onsite and placed between the first three lifts to the bankfull elevation.

(a)

(b)

Figure CS18–20 (a) Embankment construction continues on lift #9. Note the terrace setback about midway up the bank. This feature provided a flat zone to facilitate shrub planting. Another terrace setback was placed at lift #9. (b) Embankment construction is complete and vegetation planting has started. The NRCS Plant Material Center provided guidance on native vegetation selection and appropriate species for the project.

(a)

(b)

Figure CS18–21 (a) Excavator used a chain to place log members in position for the construction of the engineered log jams. Individual log members were bolted together, and rock ballast was placed. Note pool in background. (b) The presence of salmon in the immediate vicinity of the construction area required careful management of turbidity and site runoff. Photo was taken at the pool noted in figure CS18–21a.

(a)

(b)

Project performance

High flows tested the project after construction was completed in August 2004. A late December storm brought significant snowfall to the Cascades which rapidly melted during a warming trend. This snowmelt-driven runoff, combined with rainfall, resulted in considerable discharge in many of the Cascade River systems. The project experienced a flow of approximately 2,500 cubic feet per second without any erosion (fig. CS18–22). The revegetation and plantings were in a dormant condition and offered little benefit of hydraulic resistance, which served as a testament to the effectiveness of the ELJ and stream barb design incorporated into the project. Currently, the growth of the vegetation components, including willow cuttings used in the embankment and the clump plantings placed along the toe, have provided an additional factor of safety against erosion (figs. CS18–23 through CS18–25).

The objectives of the project in providing bank stabilization and habitat improvement were met completely. The landowner was originally faced with imminent loss of property and now has a bank that is restored to a stable condition, and the buildings are protected. From a technical standpoint, the project has proven that earthen embankments can be used in a dynamic fluvial environment if appropriate hydraulic control is incorporated. Additionally, bank protection projects and fisheries habitat improvement are not mutually exclusive applications, but can be designed in concert to meet multiple engineering and ecosystem-based objectives.

Figure CS18–22 (a) Project nearing completion. All primary project components are complete except for embankment vegetation. (b) November 2004 flooding approximately 2 months after the completion of construction. Note high velocities deflected at the upstream log jam (on left of photo) and the subcritical, low-shear stress flow condition in the near bank region along the embankment toe.

(a)

(b)

Figure CS18–23 (a) Looking downstream along the embankment immediately after construction and before planting of vegetation (August 2004); (b) Same view of project in December 2005 showing vegetation establishment. Note location of buildings in both photos.

(a)

(b)

Figure CS18–24 (a) Looking upstream along the embankment immediately following placement of vegetation (August 2004); (b) Same view of project in December 2005 showing establishment of vegetation with vigorous willow growth along the embankment toe. Note location of buildings in both photos.

(a)

(b)

Figure CS18–25 (a) Preproject bank condition (June 2004); (b) Bank condition 1 year after project completion

(a)

(b)