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.

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

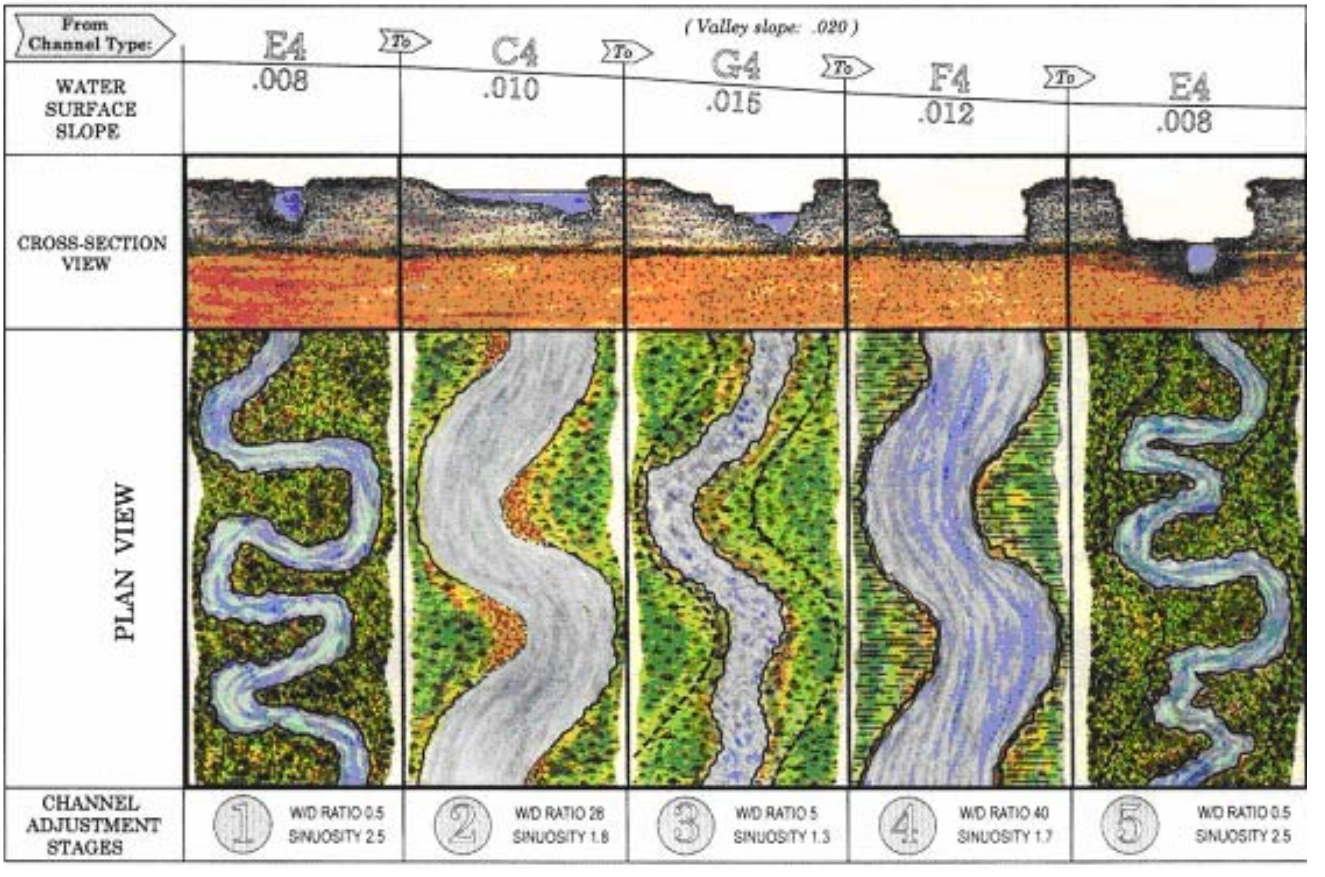

Table TS3E–6 Summary of characteristics of Rosgen stream types by watershed conditions

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.

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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.

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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 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.

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.

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

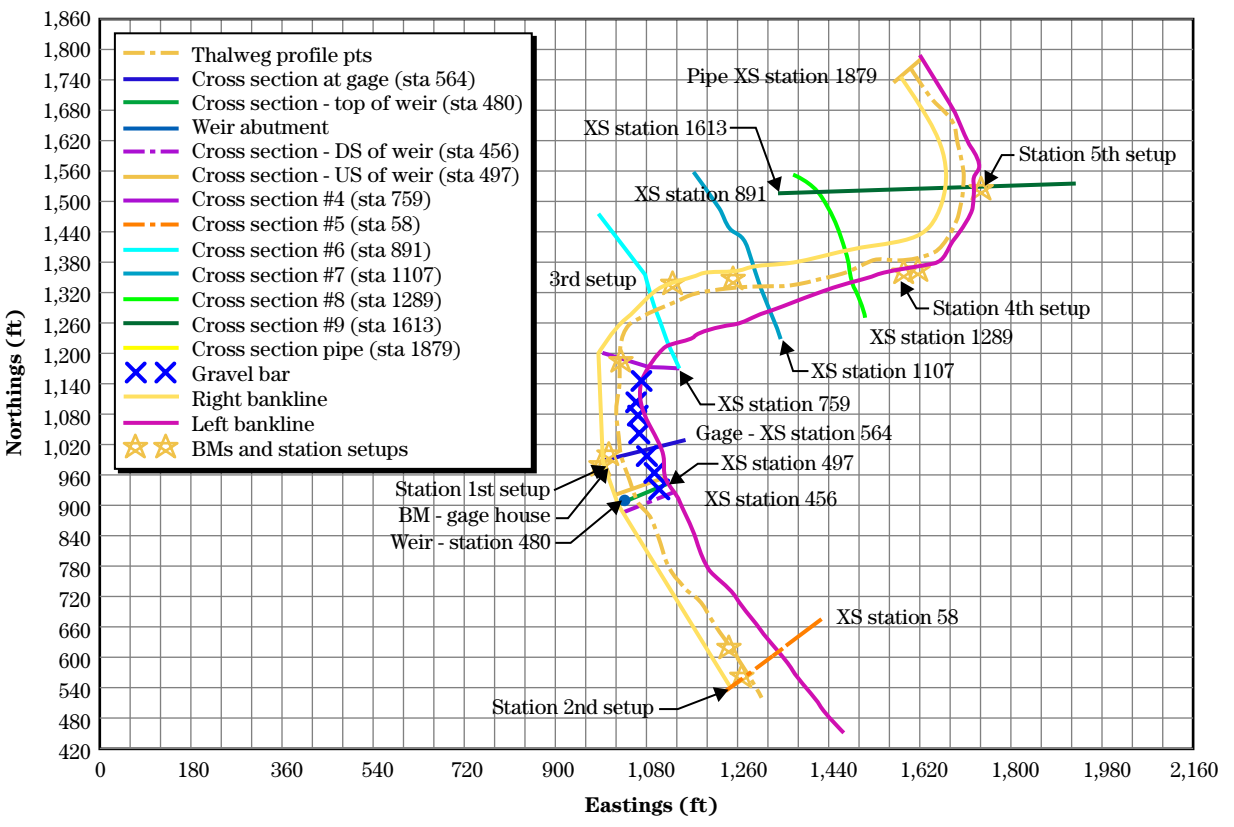
area/top width from the discharge measurement notes for a given discharge. Manning's n can vary considerably with depth of flow. Streams characteristically have high roughness at low flows and become hydraulically smoother as depth of flow increases. It is also important to note that this is a normal 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.

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.

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.

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

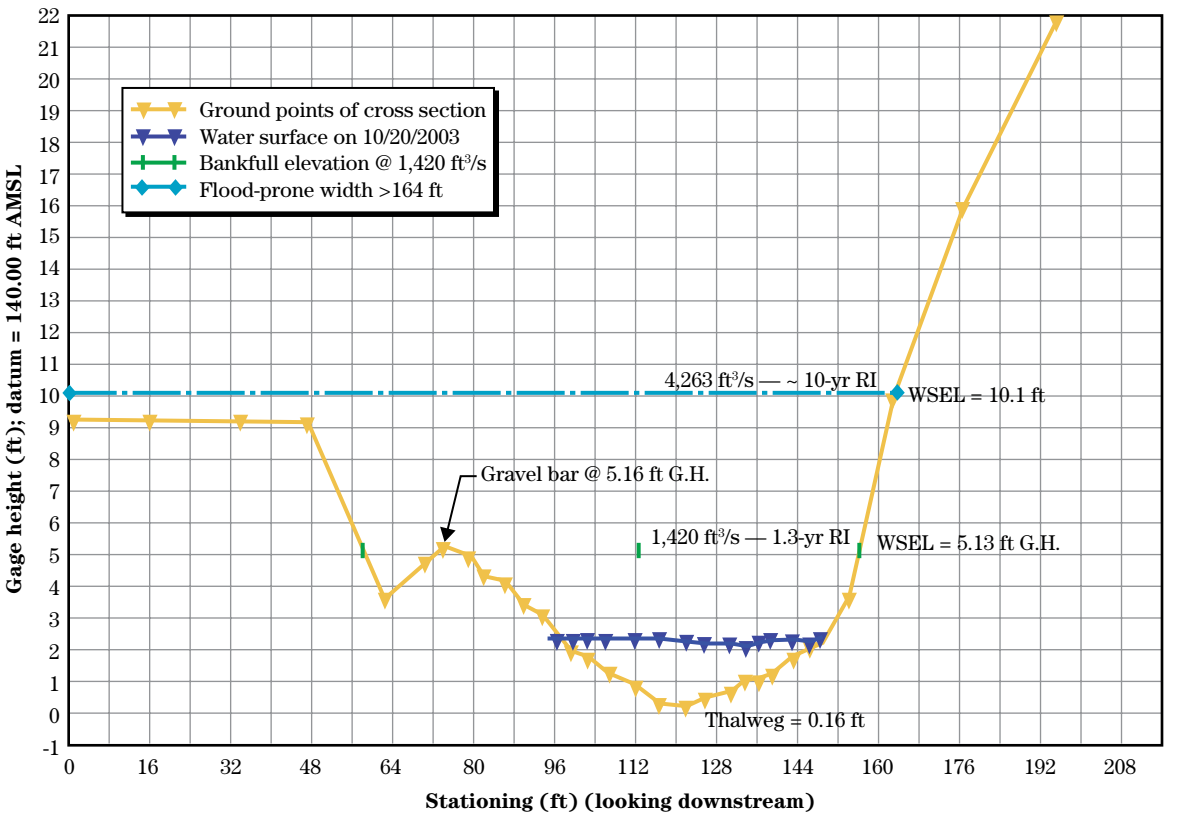
is normal protocol to define a cross section looking downstream with the stationing (in the cross section) increasing from left to right.

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.

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 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

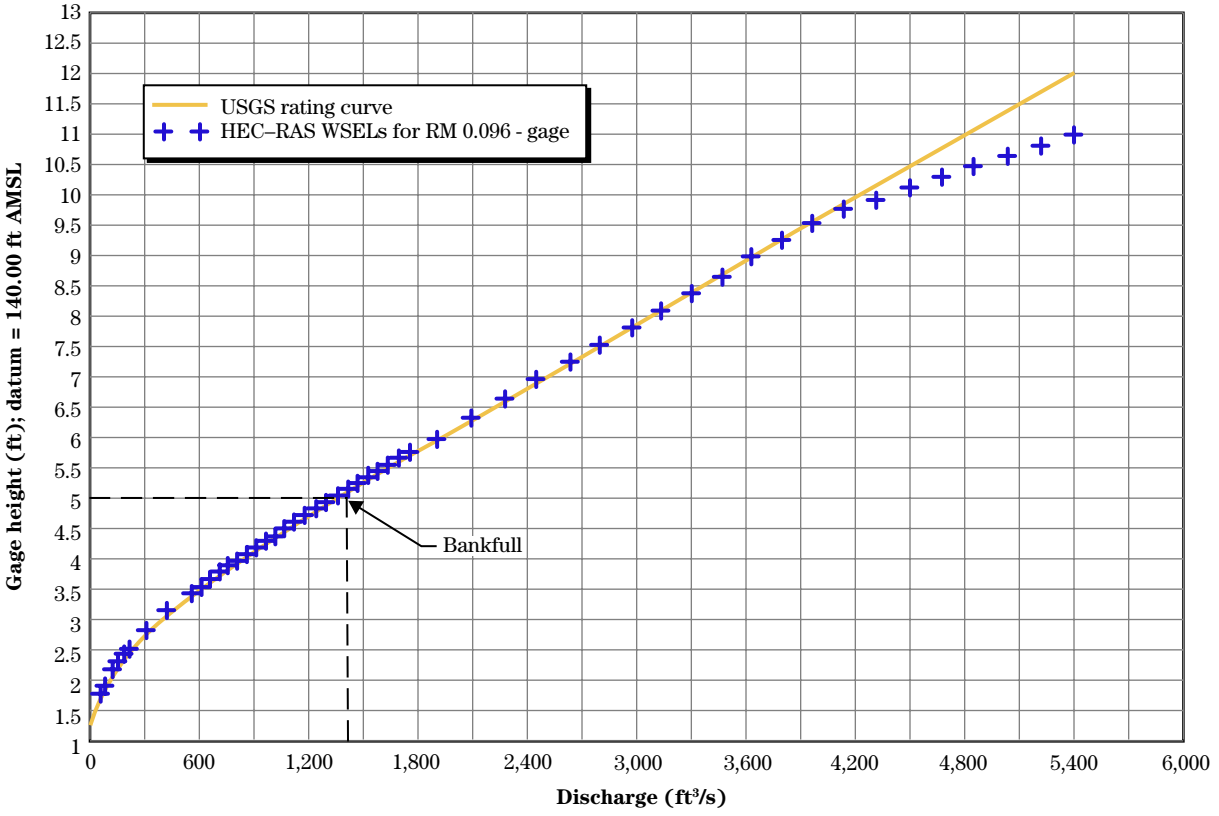
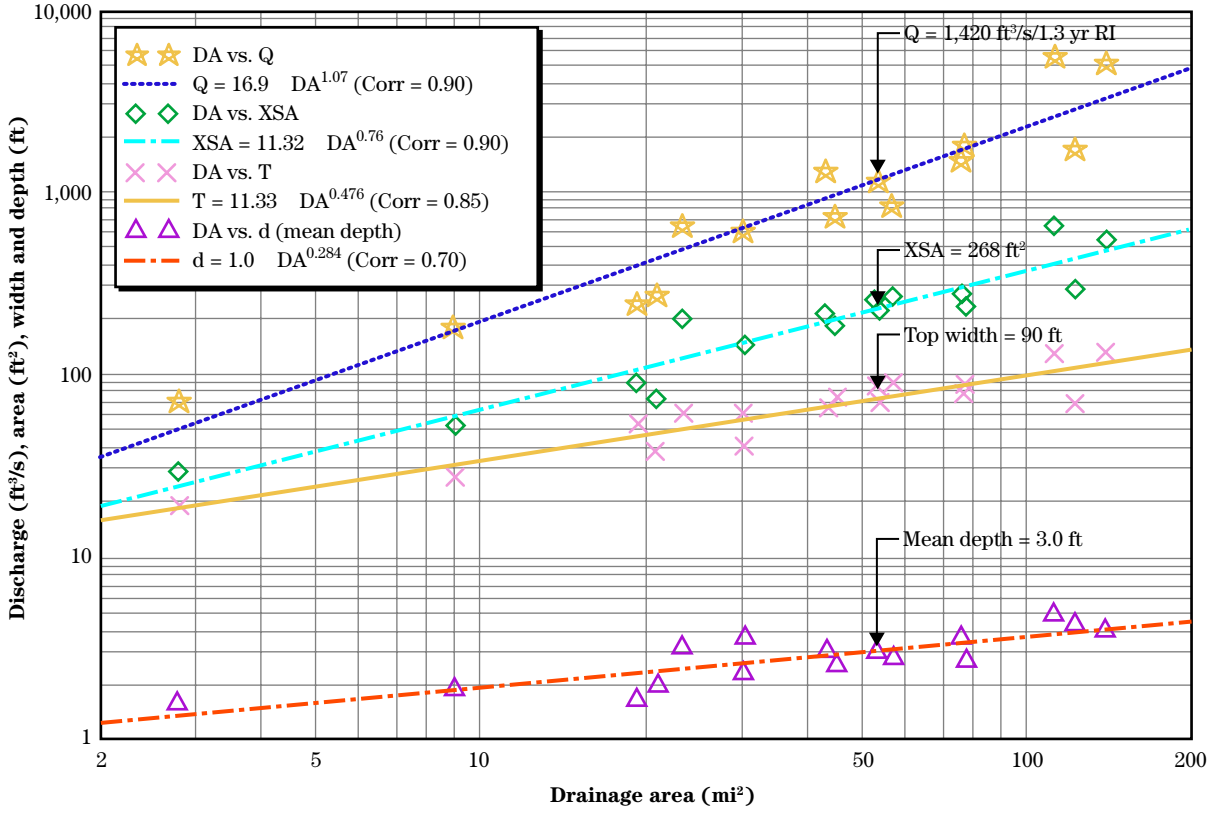


Figure TS5–5 Regional curves for hydraulic geometry



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.

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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.

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

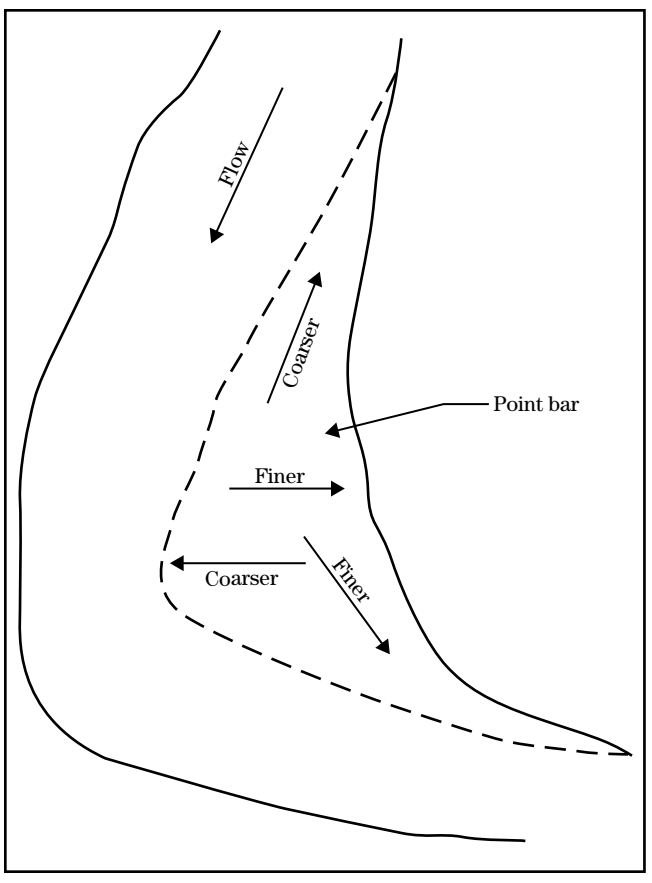
This approach requires the selection of a representative cross section for a reach. In streams with relatively uniform depths, between three and five samples should be taken across the section to account for lateral variations. In streams with variable depths, more samples are required. Twenty verticals 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



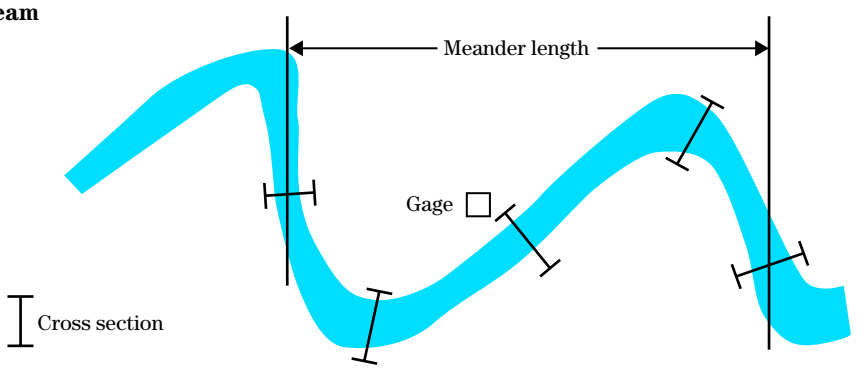
An alternative to the cross-sectional approach is the reach approach. A reach is defined as a portion of the stream with similar morphology (identified by its homogeneity). Generally, five cross sections are laid out in the homogeneous reach. If there is a gage in the reach, locating the center cross 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.

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.

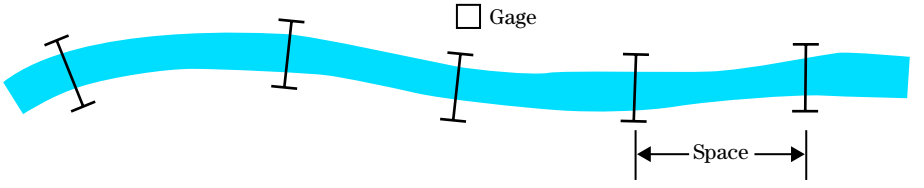
Since the spatial variability in most coarse bed streams is high, securing representative samples is difficult. River bars are frequently chosen as sampling sites because they are considered the most representative of the sediment moving in the stream, and they are usually dry during sampling. Specific 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



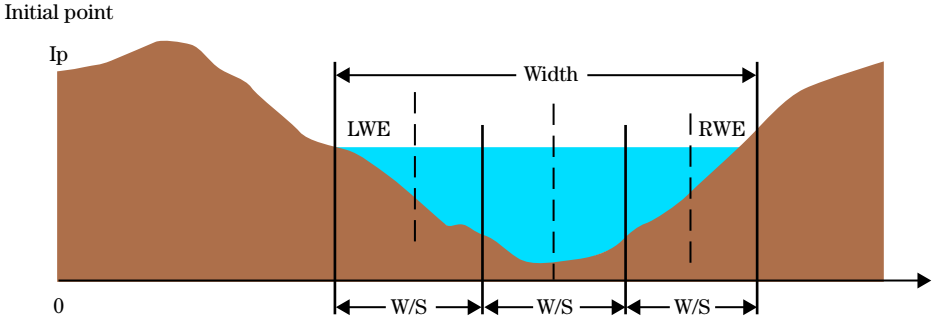
Straight channel



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

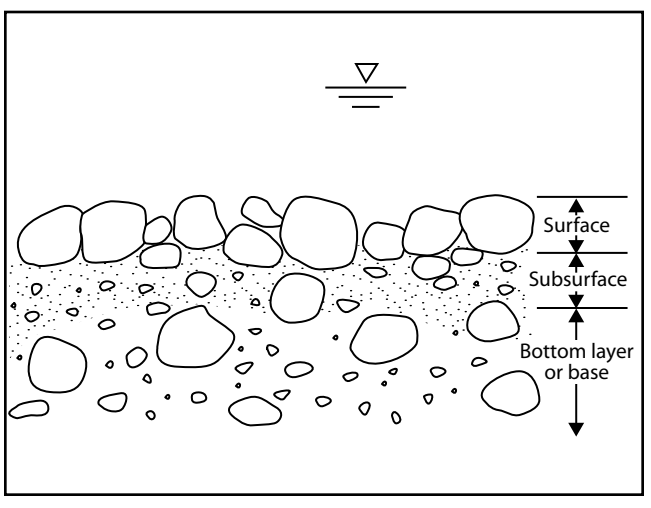
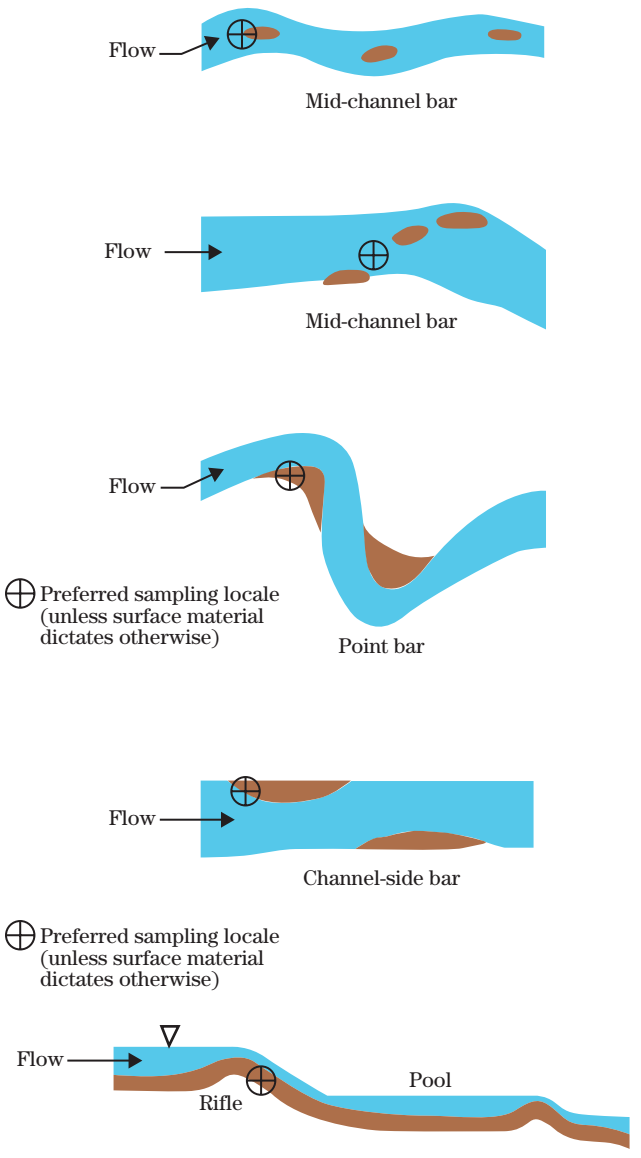


Figure TS13A–4 Coarse bed stream sampling hierarchy



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.

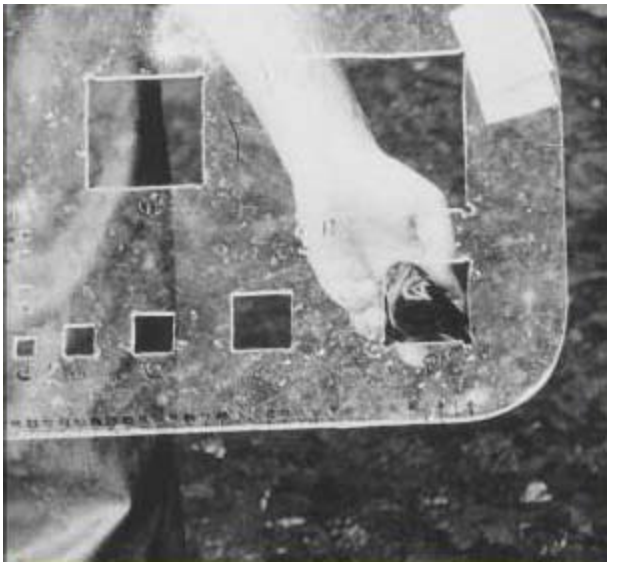
As previously noted, in coarse or gravel-bed streams, the top layers may be stratified by size due to armoring effects. Typically, bulk sampling is employed to characterize the subsurface or base layers. However, to quantify the particle size of the surface, a surface sampling technique is typically 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

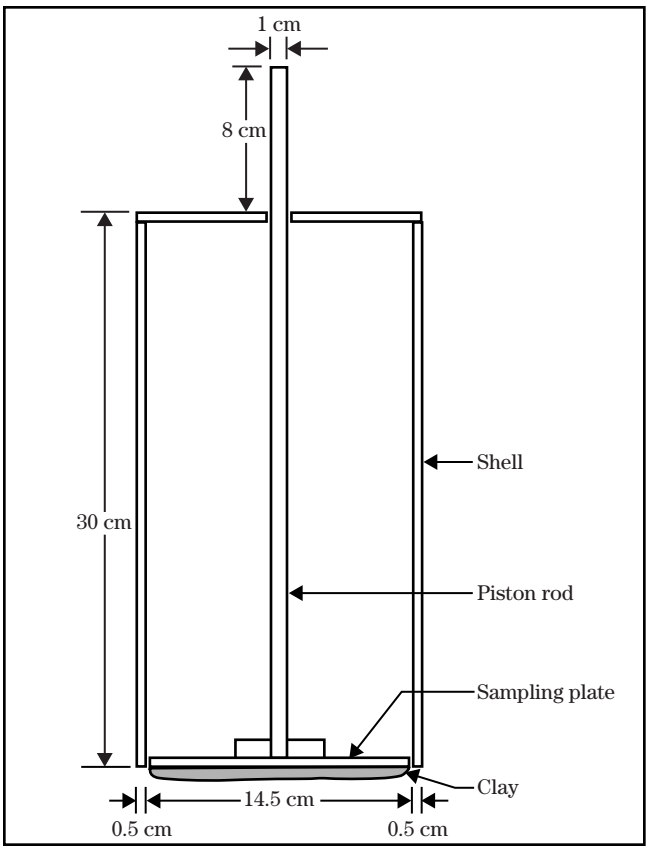
$$C = \frac{1}{\sum p(S)_i D_i^{-1}} \qquad \text{(eq. TS13A–2)}$$

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.

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.

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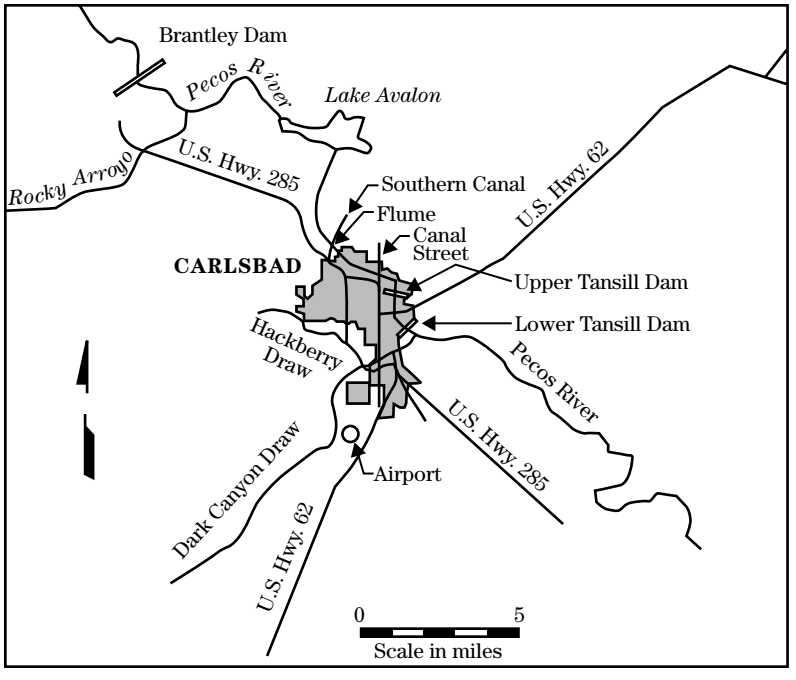
A sediment budget analysis was conducted by the U.S. Army Corps of Engineers (USACE) as part of the reconnaissance level planning study for a flood-damage reduction project for the City of Carlsbad, New Mexico (Copeland 1995). This example describes the sediment budget analysis used to identify the magnitude 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



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.

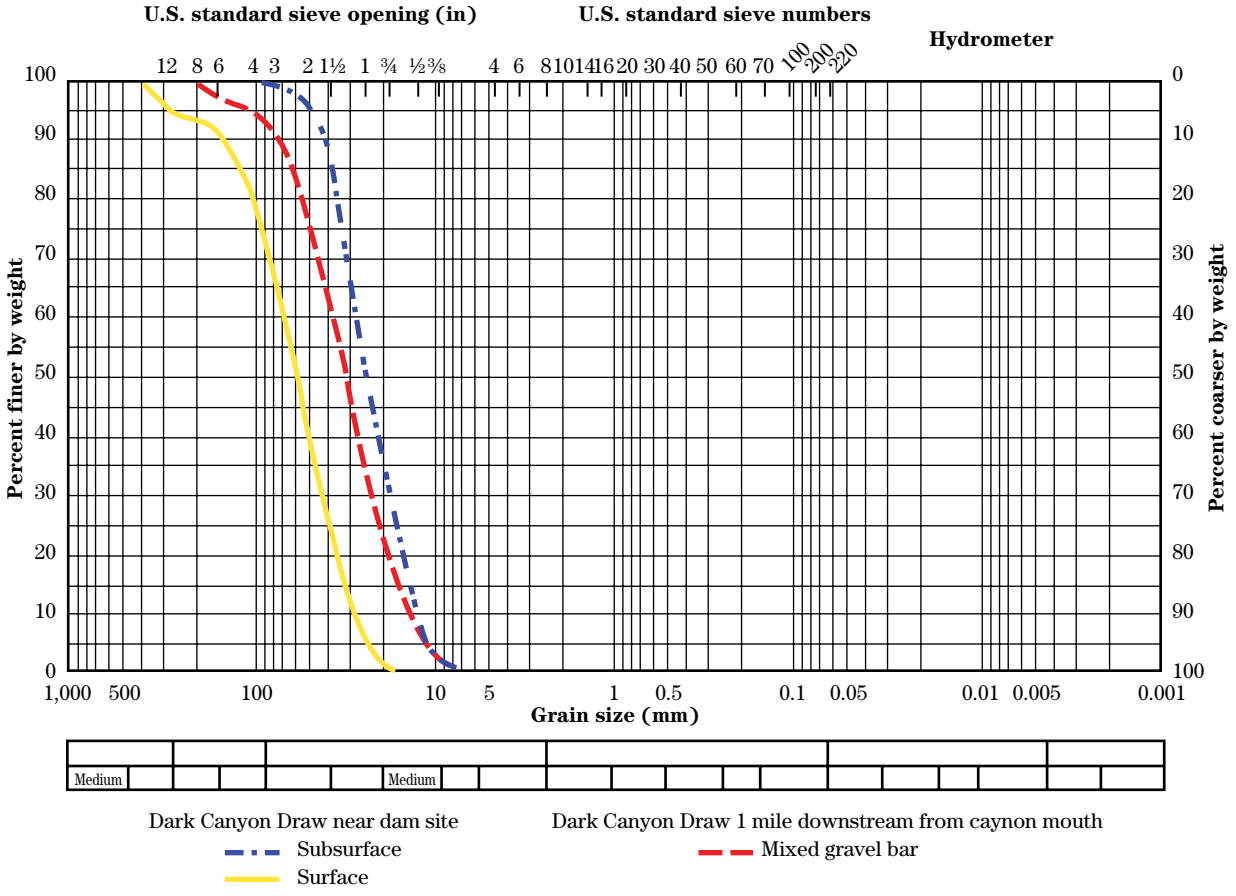
Bed-material samples were collected during the field reconnaissance. Sample size class distributions were determined using the Wolman (1954) pebble count method and the volumetric bulk method. Due to the limited scope of the sediment impact assessment, samples were collected at only two sites. Both surface 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.

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

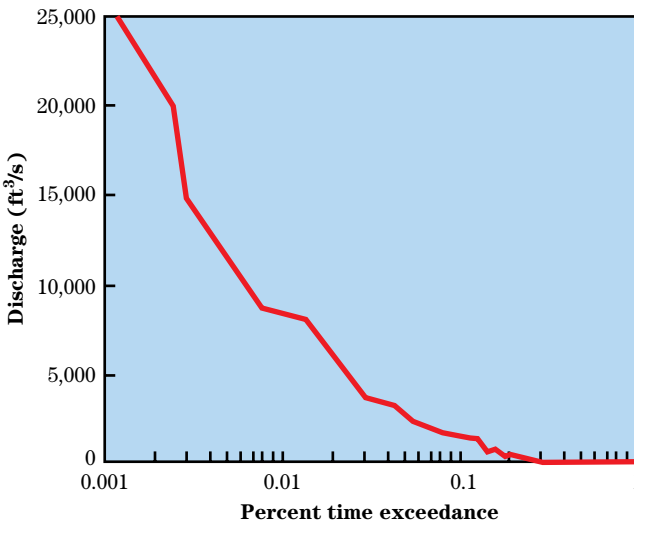


shapes similar to the 1 percent chance exceedance hydrograph. The flow-duration curve is shown in figure TS13B–4.

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).

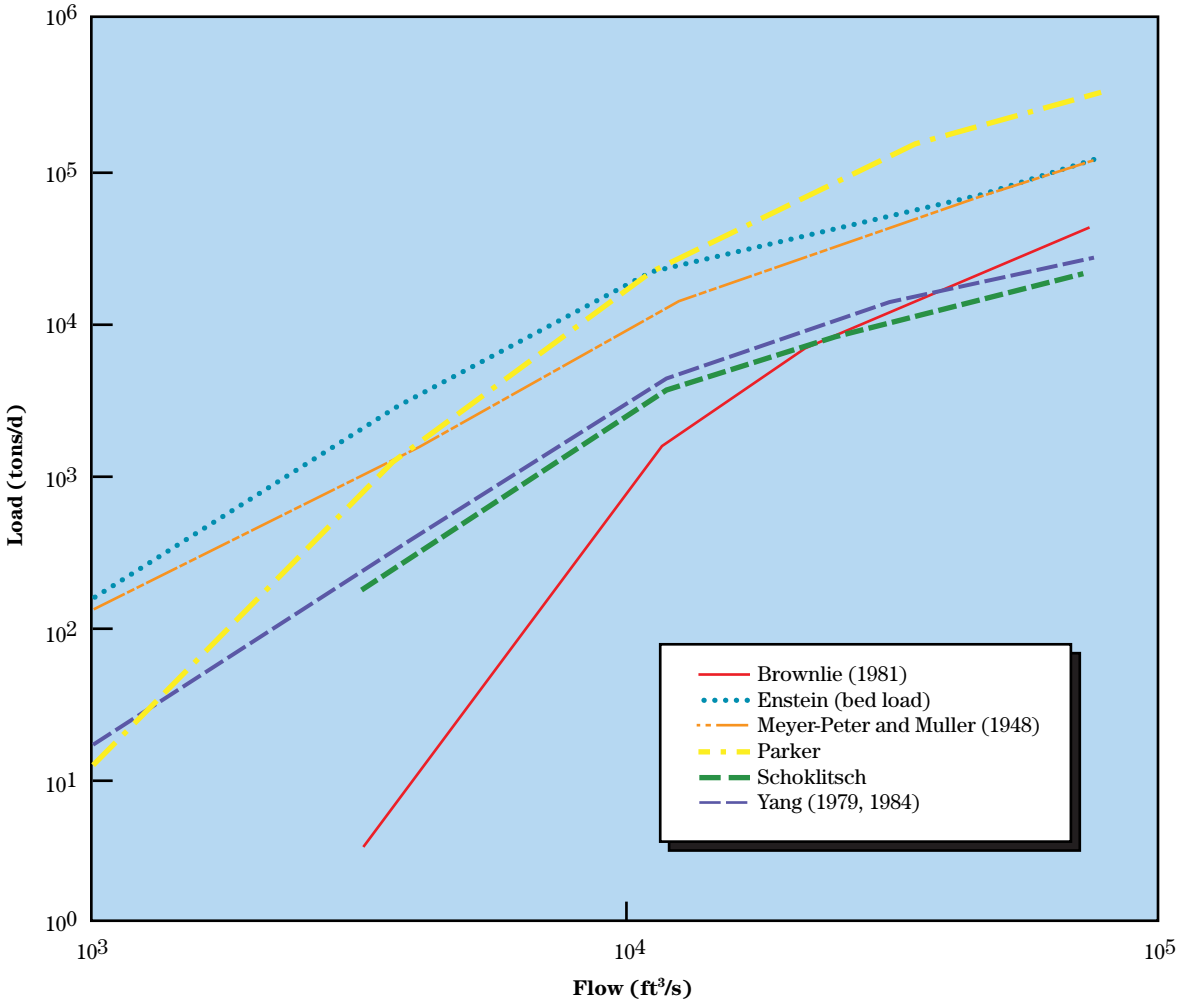
Figure TS13B–4 Dark Canyon Draw, Carlsbad, NM, flow-duration curve (1973–1992)



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

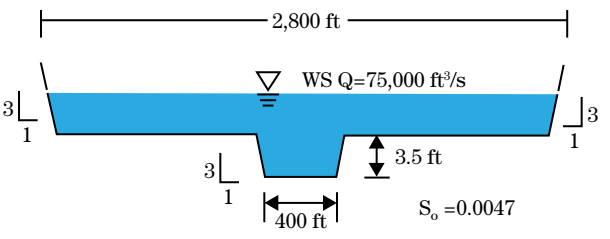


The following criteria were chosen for the diversion channel design:

- 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



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.

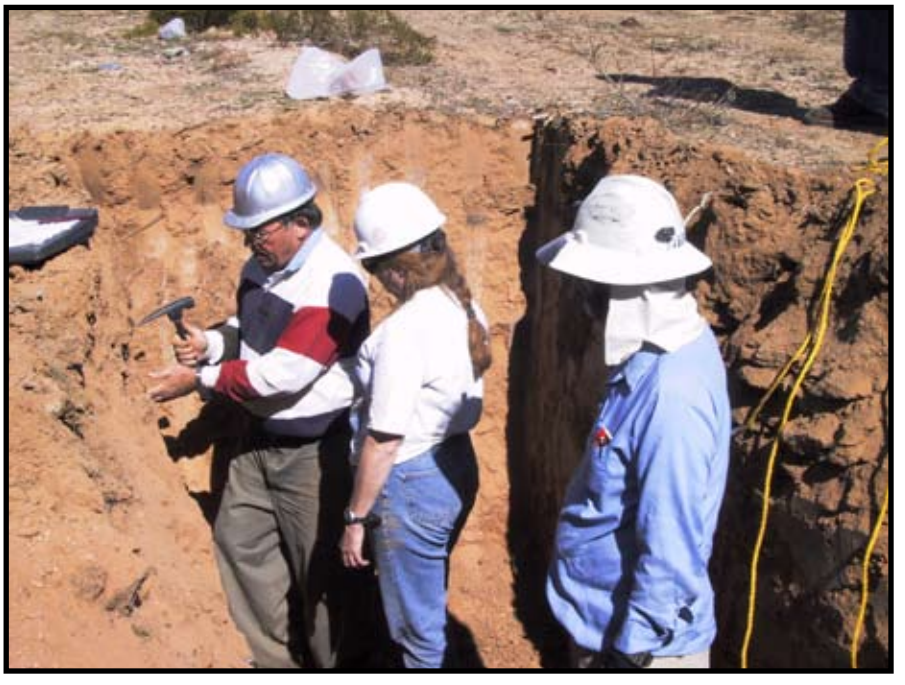
Further analysis

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)



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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.

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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

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.

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.

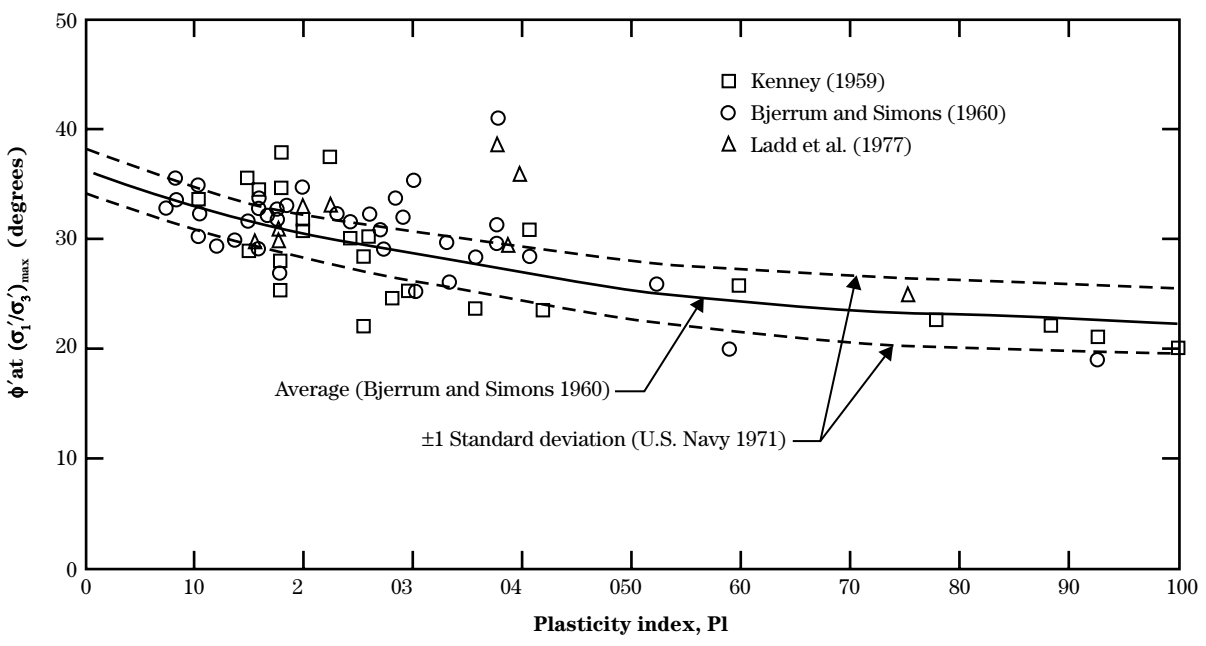
Stream channel banks can fail when conditions change that affect the stability of the slope. Examples of changes in conditions include changes in the potentiometric surface (water table) in the slope; changes in the slope configuration including increased height of the slope due to stream bed degradation, 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

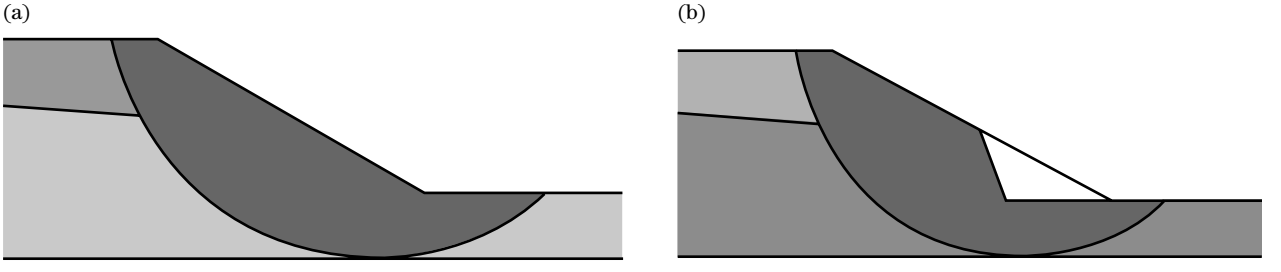


Figure TS14A–4 Failed streambank in Tarboro, NC, following high channel flows following Hurricane Floyd. Erosion of the toe caused bank instability in the slope. Measures to protect the toe of the slope are essential, in addition to assuring stability of the system.

(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.

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)

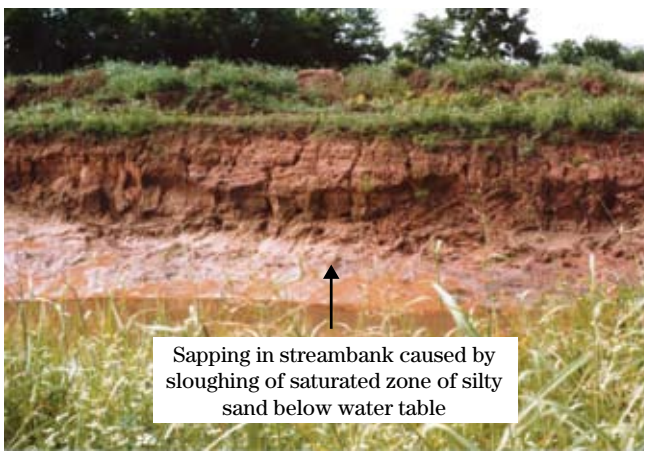
(d)

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.

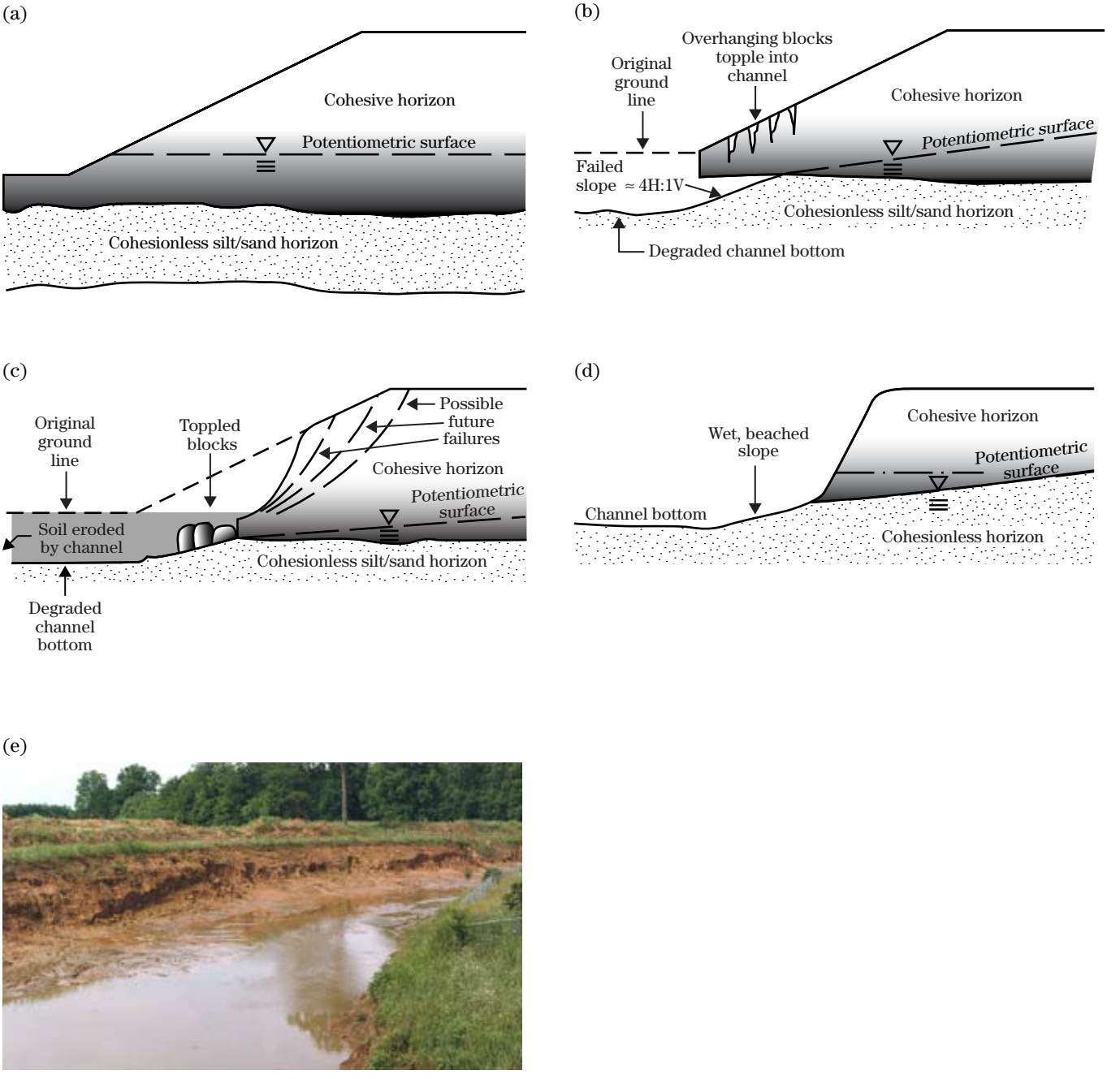
The stability of slopes of low plasticity soils is analyzed using a set of equations that are termed infinite slope equations. Commonly, stable saturated slopes for low plasticity soils range from about 2.5H:1V to 3.5H:1V. Some stream slopes may be stable in these types of soils in a moist condition 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 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.

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.

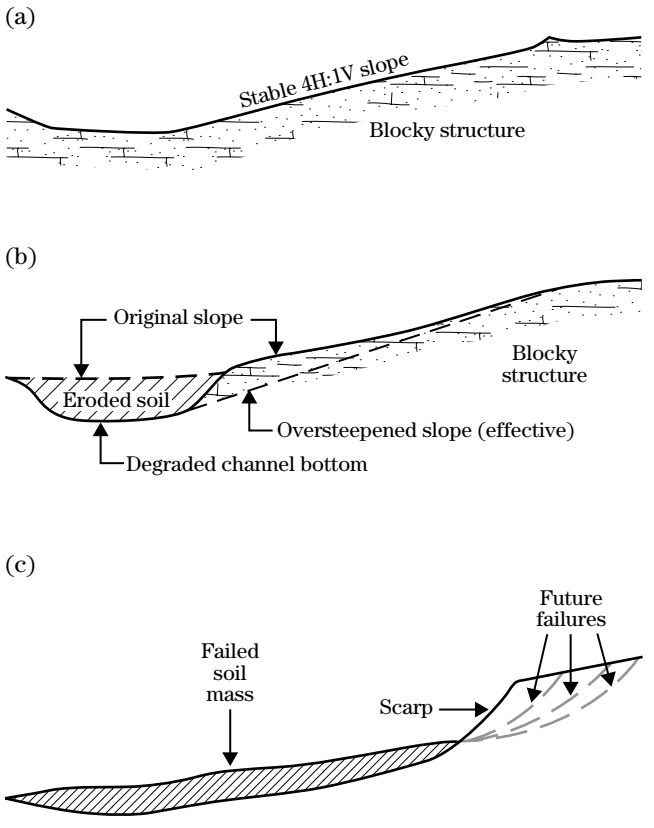
These types of failures have a shallow circular shape; are about 3 to 4 feet deep, measured normal to the slope surface; and frequently occur progressively. Larger failures follow small initial slides, if no corrective measures are taken. The scarp face (near-vertical surface at the top of the slide) 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.

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):

$$FS = \frac{\bar{c} \times \gamma_b z \times \cos^2\theta \times \tan\bar{\varphi}}{\gamma_{sat} \times H \times \sin\theta\cos\theta} \qquad \text{(eq. TS14A–1)}$$

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

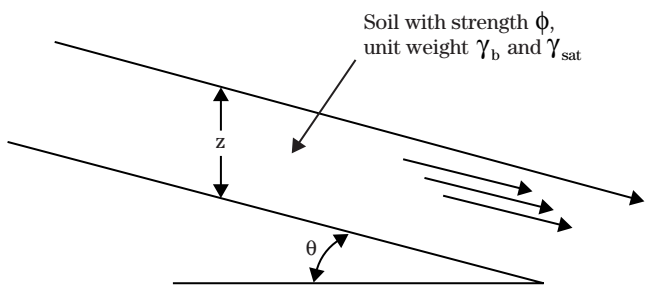


Figure TS14A–23 Blocky structured clay in streambank exposure

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 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.

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 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.

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.

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.

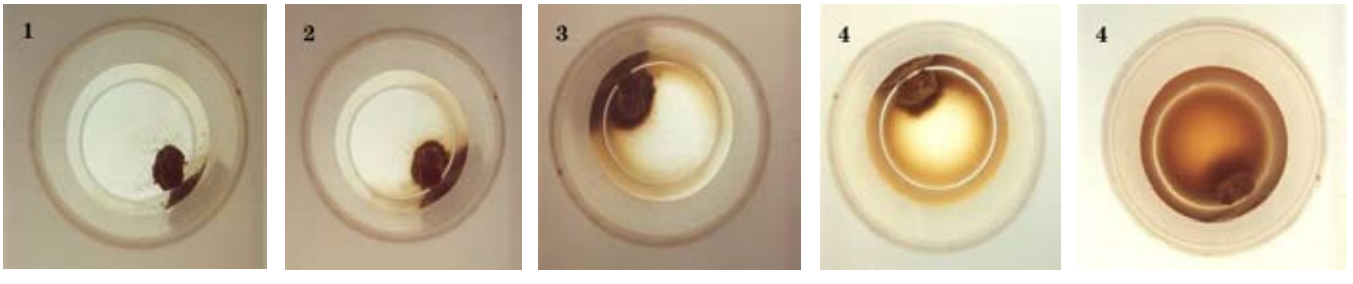
4—Cloud spreads around bottom and may cover bottom. The colloidal cloud may be so extensive that the whole bottom of the glass is covered.

Reactions 3 and 4 are a very positive indicator of dispersive soil.

Soil bioengineering 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.

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.

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.

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.

Figure TS14A–26 A streambank where highly dispersive clays were treated by applying and mixing in hydrated lime to the slope soils

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.

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.

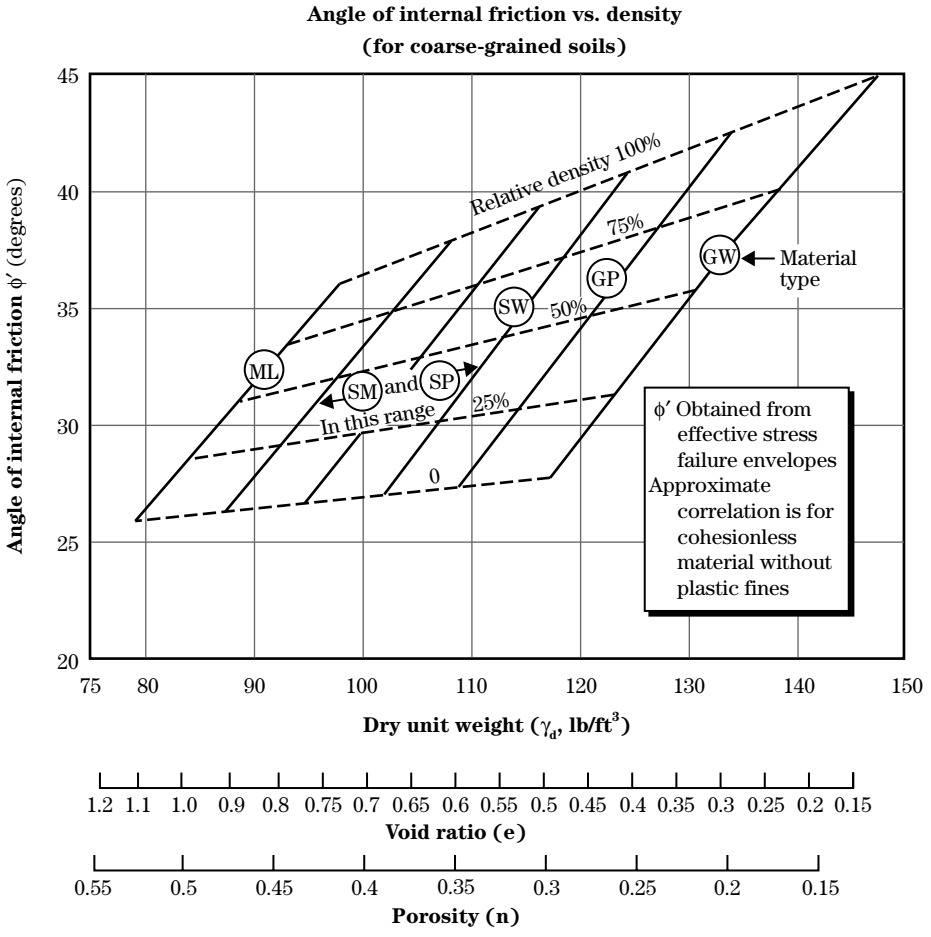
Direction of seepage forces

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



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:

$$FS = 3 \times \tan(26)$$
$$FS = 1.46$$

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

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

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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.

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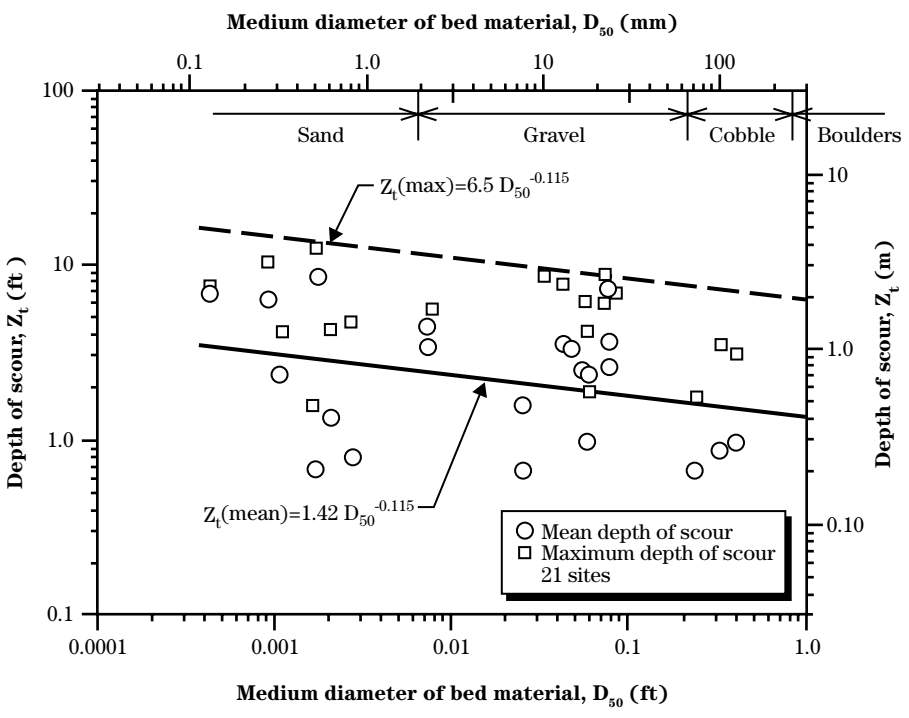
Scour is one of the major causes of failure for stream and river projects. It is important to adequately assess and predict scour for any stream or river design. Designers of treatments such as barbs, revetments, or weirs (that are placed on or adjacent to streambeds) must estimate the probable maximum 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.

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

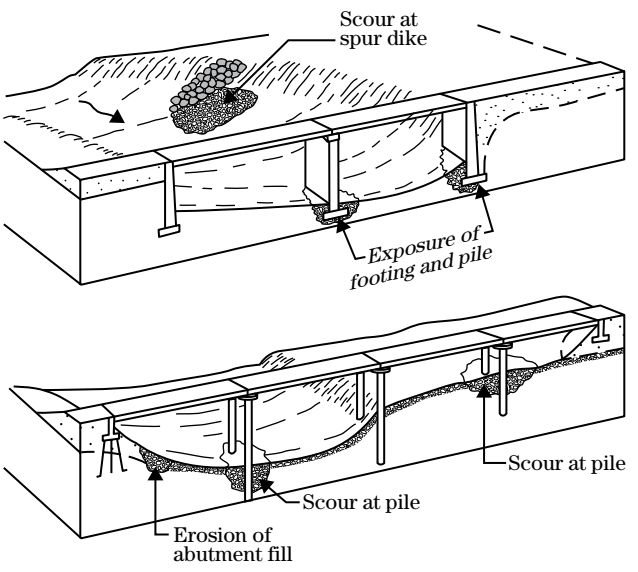
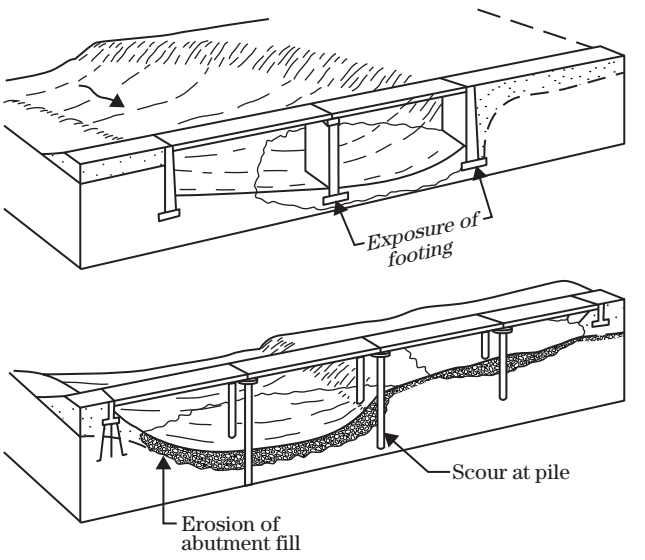


Figure TS14B–3 Examples of general scour



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.

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

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.

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

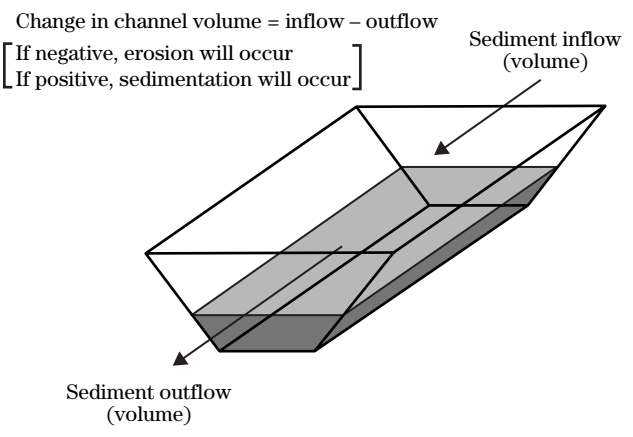
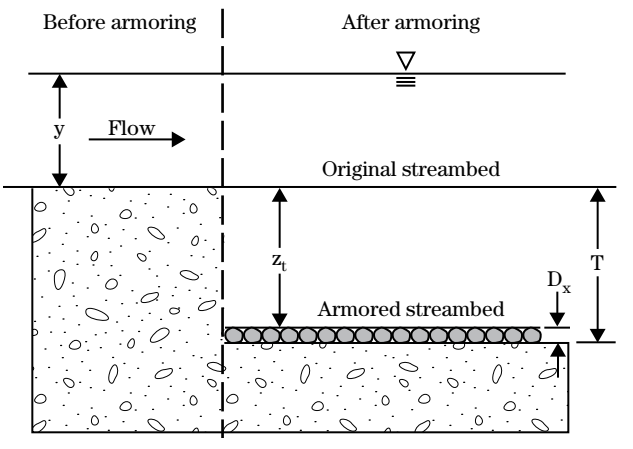


Figure TS14B–5 Definition of terms for armor limited scour



$$z_t = T - D_x \qquad \text{(eq. TS14B–2)}$$

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):

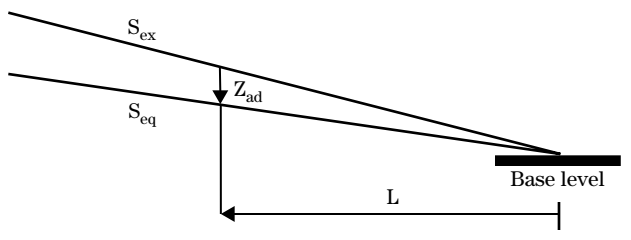
$$e = 0.245 + \frac{0.0864}{(0.1D_{50})^{0.21}} \qquad \text{(eq. TS14B–5)}$$

where:
D_{50} = median grain size in mm

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

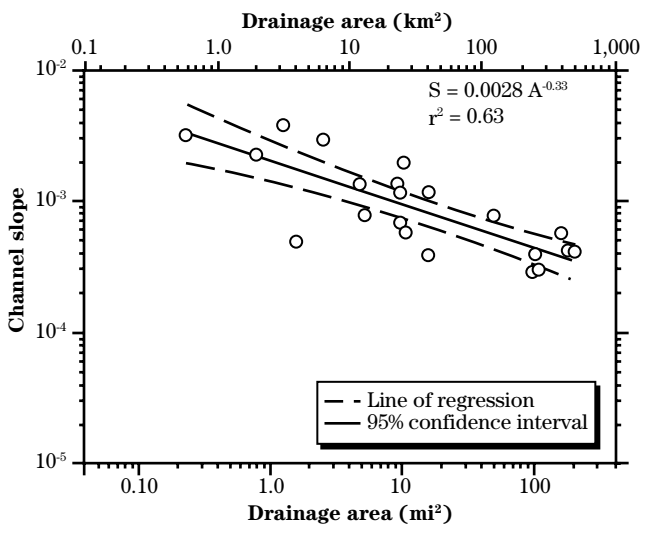
Nickpoint (or knickpoint) migration is a dramatic form of vertical instability that occurs in cohesive soils (fig. TS14B–7). A nickpoint (or headcut) is an abrupt change or inflection in the longitudinal profile of a cohesive streambed. In noncohesive materials, analogous features are manifest as short, 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



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).

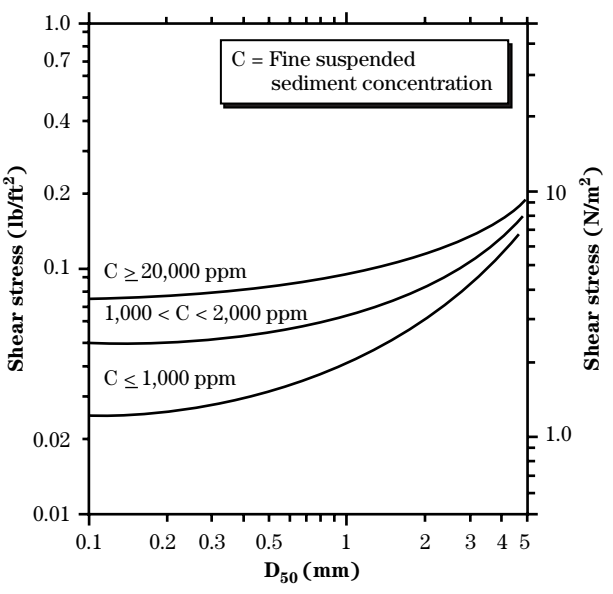
$$S_{eq} = \left(\frac{\tau_c}{\gamma_w y}\right) \qquad \text{(eq. TS14B–7)}$$

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.



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.

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

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.

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 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 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 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:

$$\tau_o = \gamma_\omega R S_e$$

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:

$$K_1 = 0.055 \tanh\left[12A^{-0.59} \exp\left(-0.0004A\right)\right] \qquad \text{(eq. TS14B–28)}$$

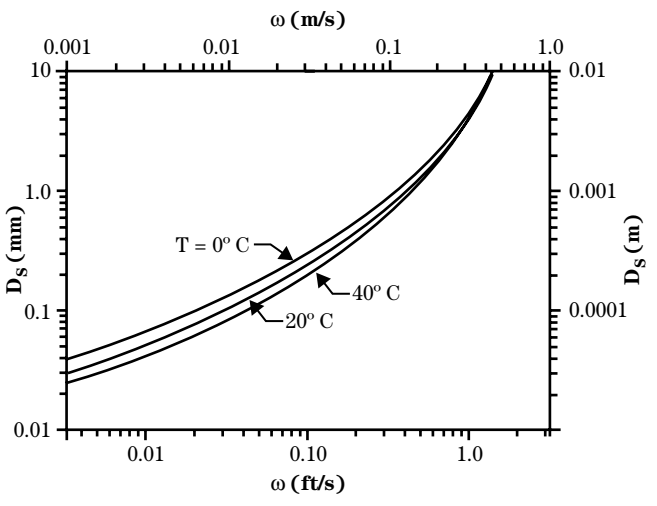
$$K_2 = 1.06 \tanh\left[0.016A^{0.50} \exp\left(-\frac{120}{A}\right)\right] \qquad \text{(eq. TS14B–29)}$$

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 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.

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.

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

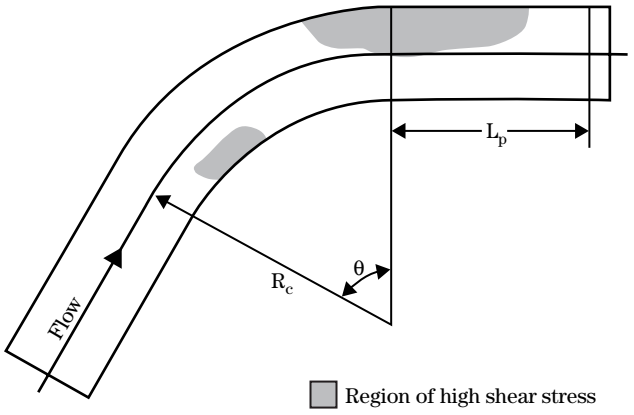
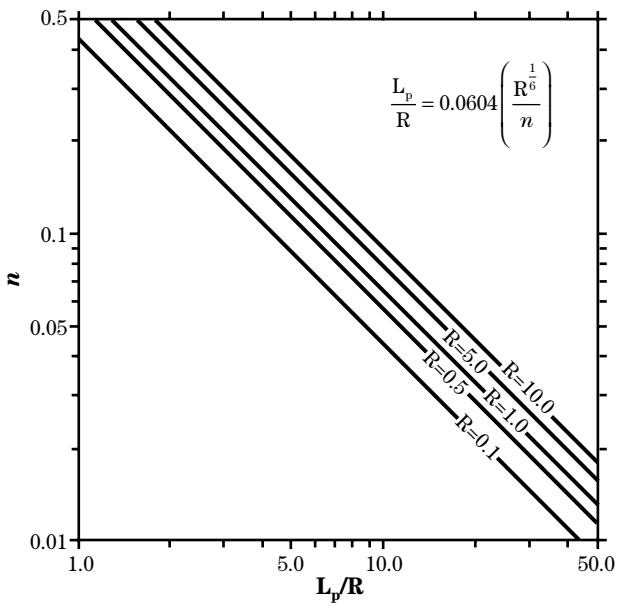


Figure TS14B–13 Recommended length of protection divided by hydraulic radius, L_p/R, as a function of Manning roughness coefficient for the bend, n, and hydraulic radius, R



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.

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.):

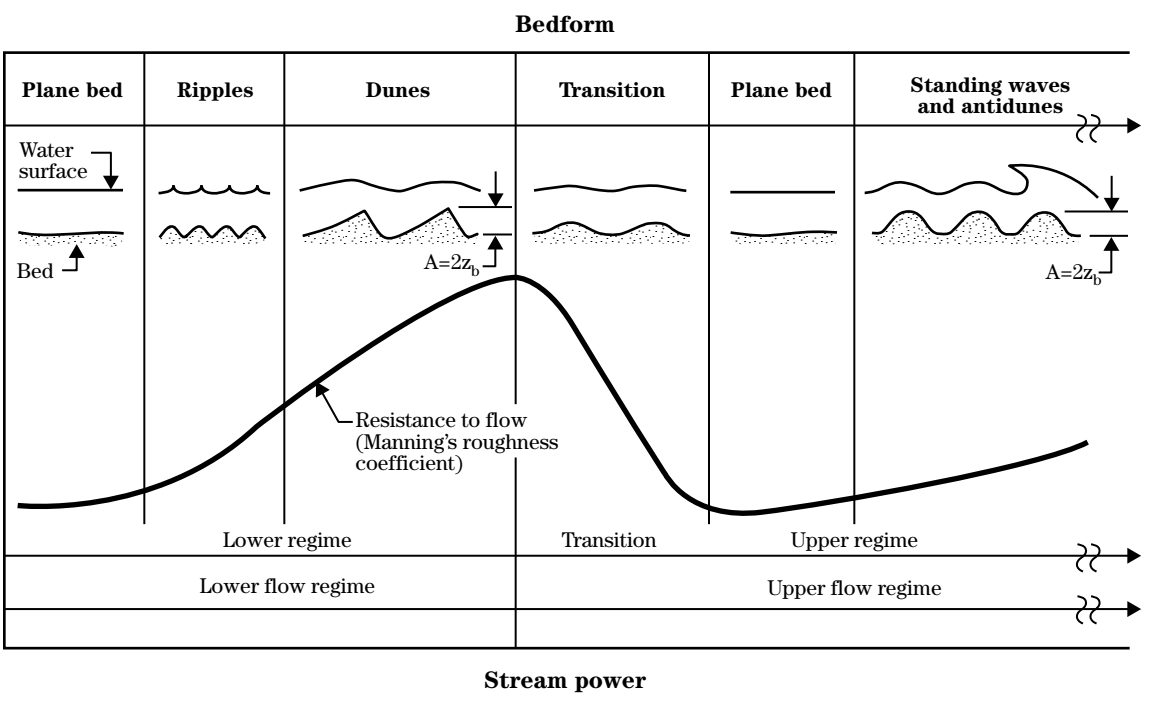
$$\theta_c = \frac{0.24}{D_*} + 0.055\left[1 - \exp\left(-0.02D_*\right)\right] \qquad \text{(eq. TS14B–41)}$$

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.

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).

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:

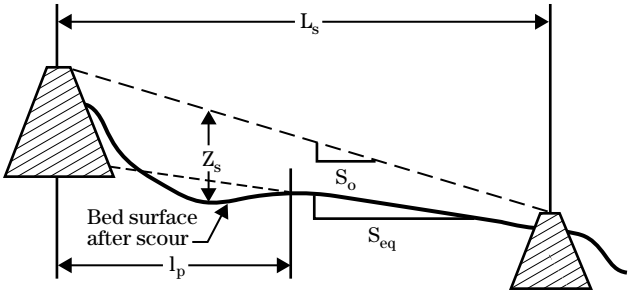
S_o = initial longitudinal bed slope

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.

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)

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)

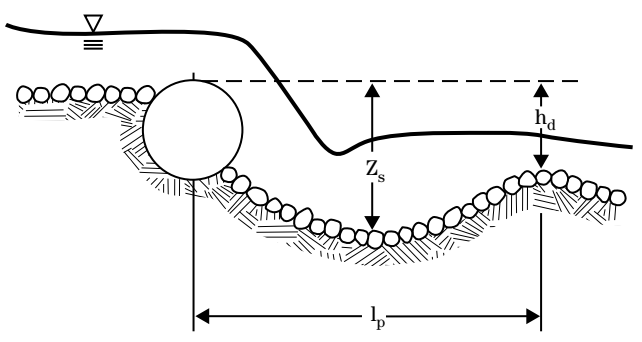
z_s = depth of scour, ft (m)

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:

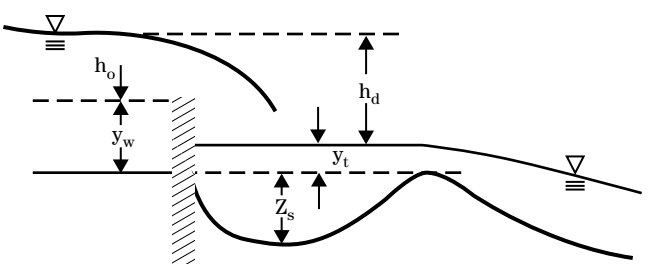
$$A_{50} = \frac{Q_d}{W_w y_w \sqrt{gD_{50}\Delta S_g}}$$

(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 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)

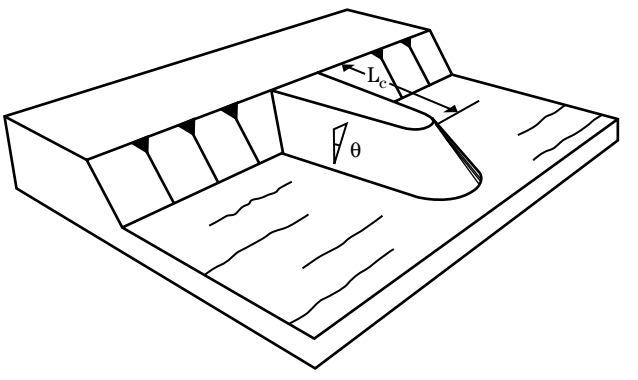
y = mean flow depth in approaching flow, ft (m)

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).

$$z_t(\max) = K D_{50}^{-0.11}$$

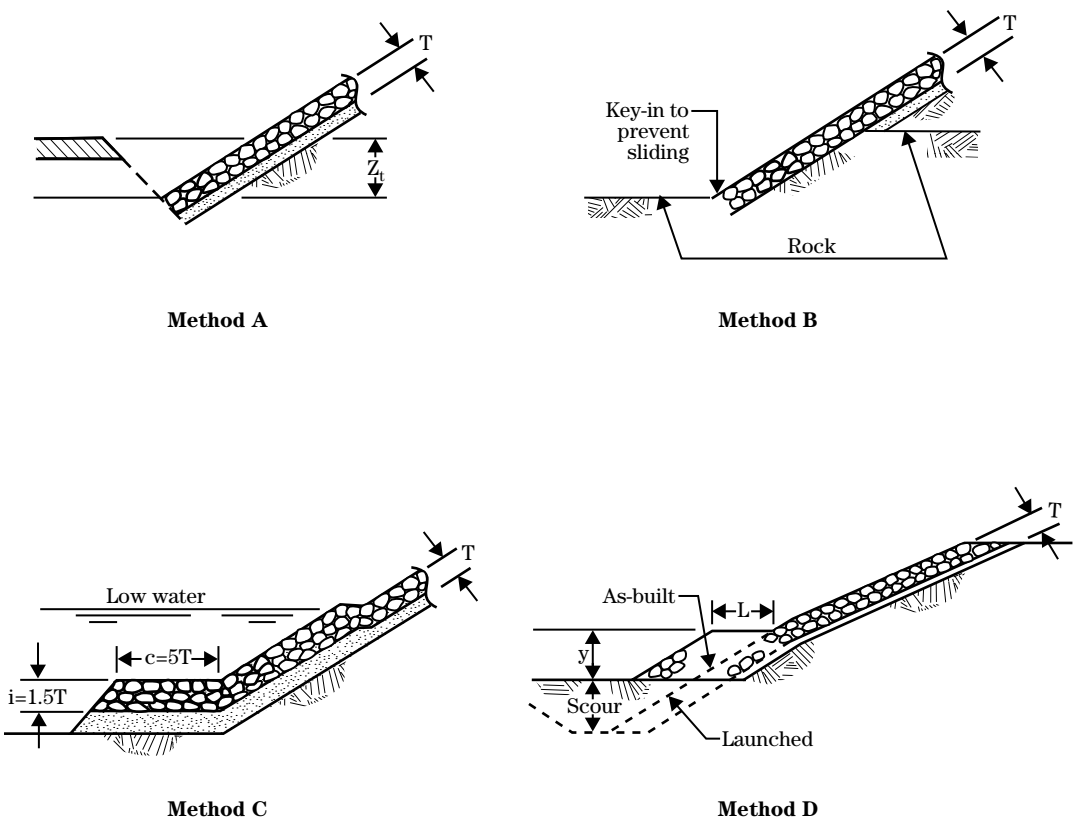
$$z_t \max = 6.5(0.043)^{-0.115} = 9.3 \text{ ft}$$

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.

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)

CAT

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.

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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.

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.

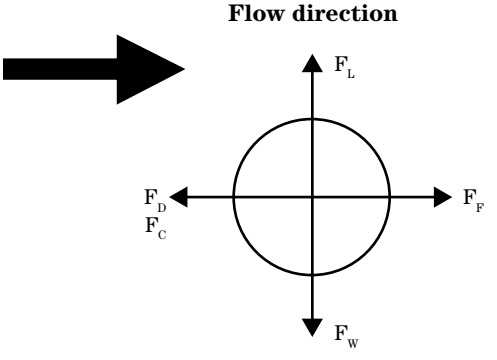
Size is one of many considerations when designing riprap for use in protecting channel bed and banks. The designer must also address issues such as material strength, density, angularity, durability, length-to-width ratio, gradation, bedding, piping potential, and channel curvature. These important design and construction considerations are addressed in NEH654 TS14K.

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



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

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

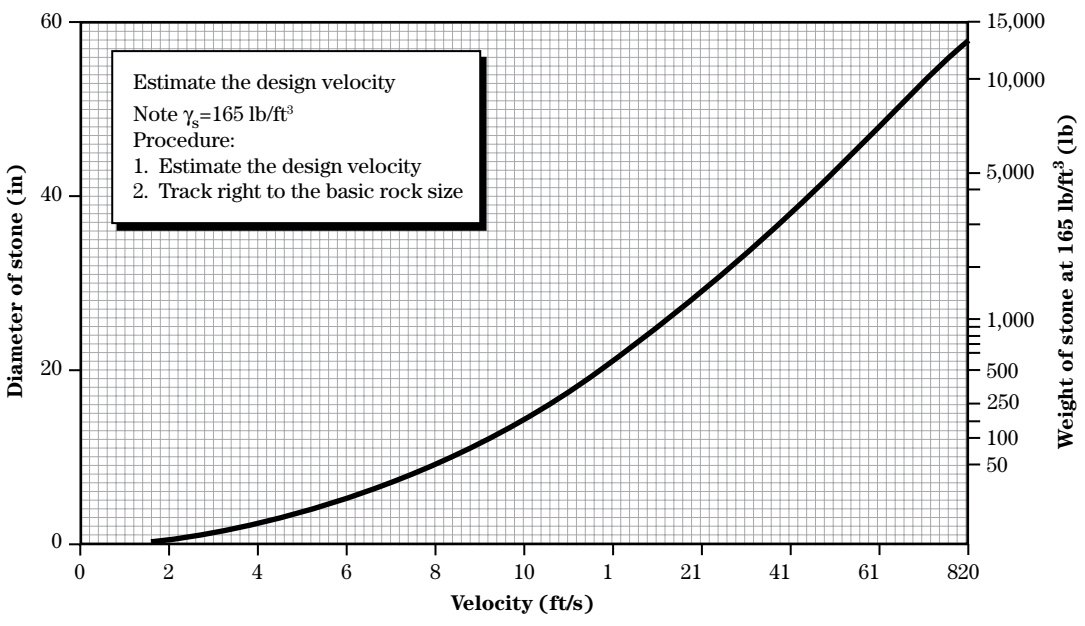
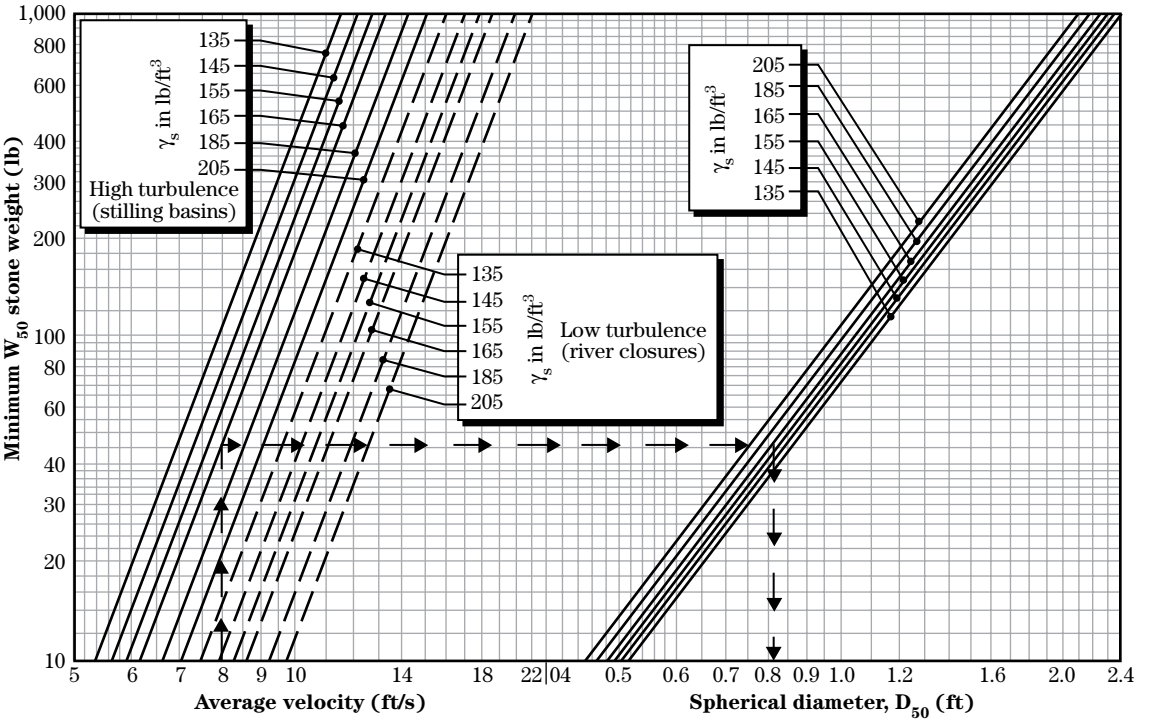


Figure TS14C–6 Graphical solution for Isbash technique

(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:

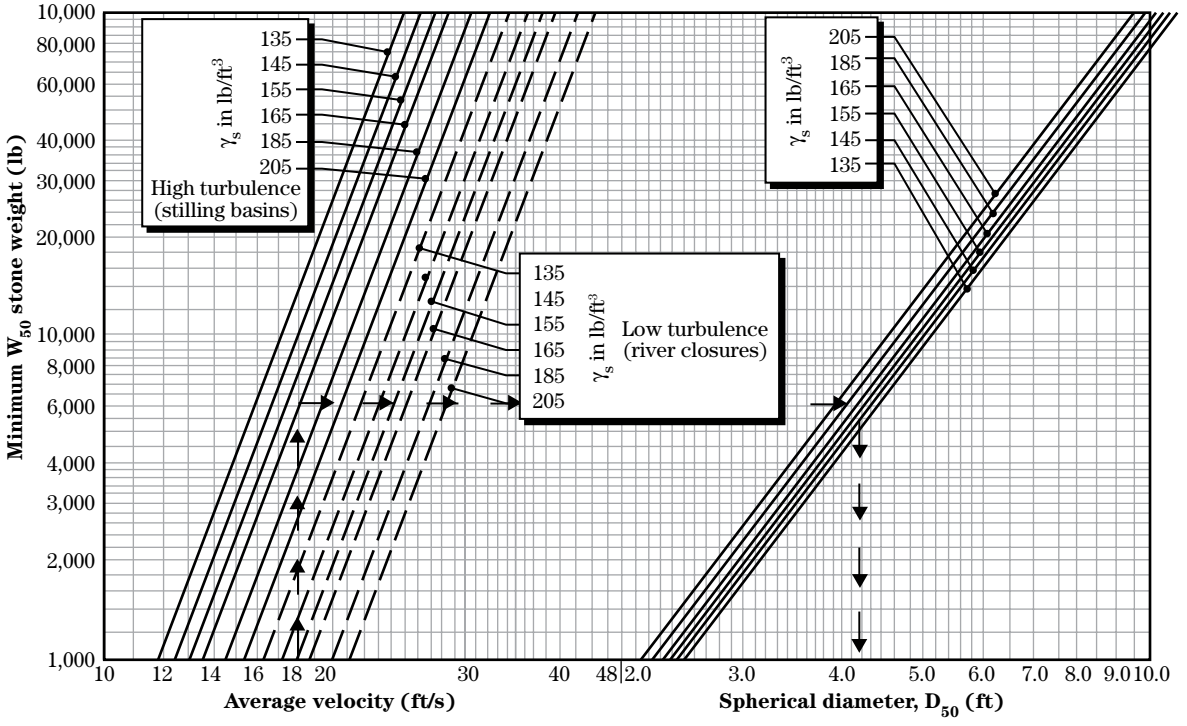
V	=	Velocity, ft/s
γ_s	=	Specific stone weight, lb/ft^3
γ_w	=	Specific weight of water, 62.5 lb/ft^3
W_{50}	=	Weight of stone, subscript denotes Percent of total weight of material containing stone of less weight
D_{50}	=	Spherical diameter of stone having the same weight as W_{50}
C	=	Isbash constant (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

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) →	n-value = *0.045*
Bed slope = *0.0060* ft./ft.	Bed slope (5:1) = *0.200* ft./ft. →	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.*)

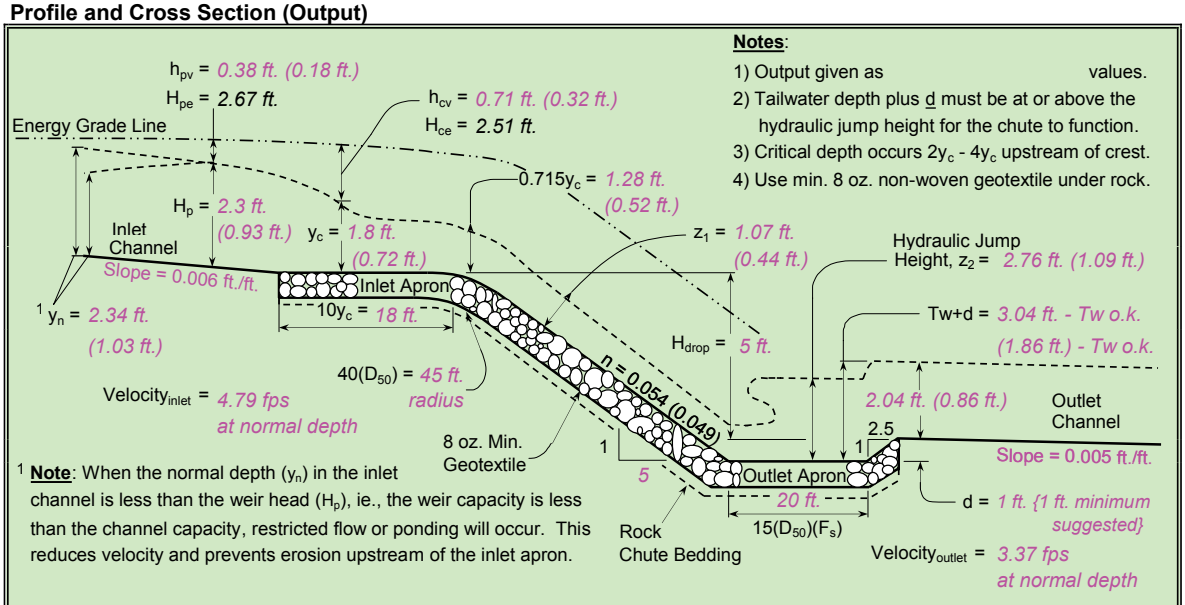
Chute capacity = Q5-year
Total capacity = Q10-year
Minimum capacity (based on a 5-year, 24-hour storm with a 3 - 5 inch rainfall)

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*

____: *The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.*
_____:

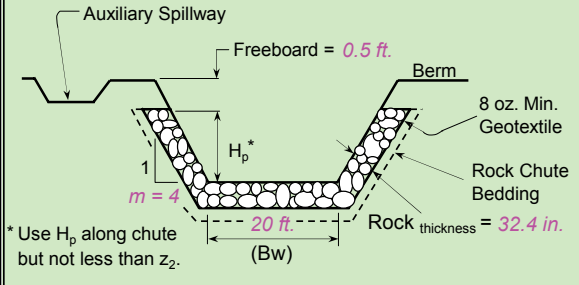
Profile and Cross Section (Output)



Notes:
1) Output given as 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

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

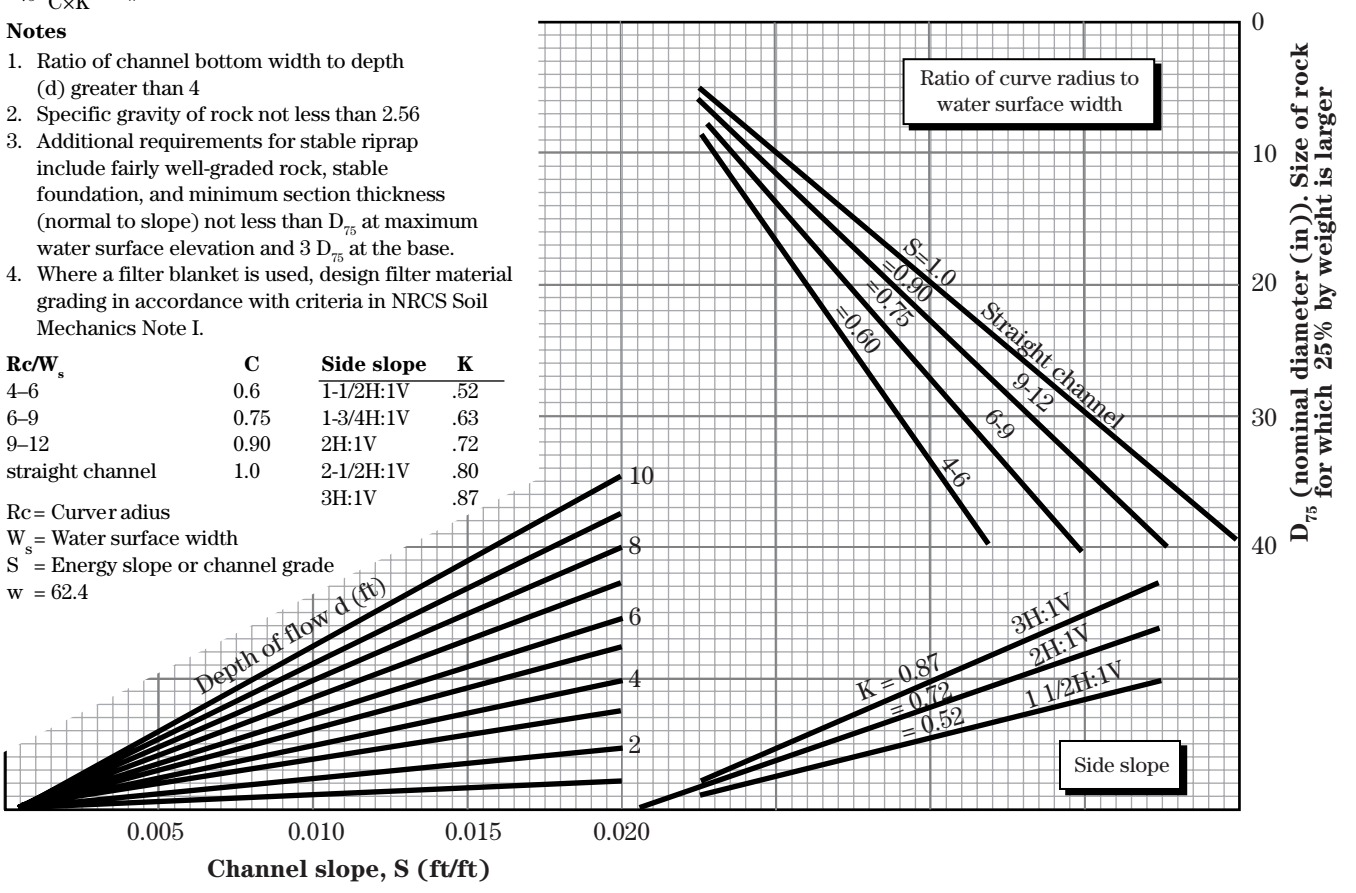
$$D_{75} = \frac{3.5}{C \times K} \times \gamma_w \times d \times S$$

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 = Curver adius
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.

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.

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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.

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.

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.

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.

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

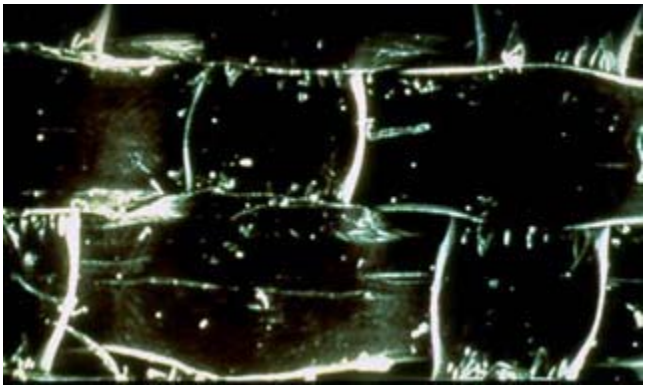


Figure TS14D–2 Needle-punched nonwoven geotextile

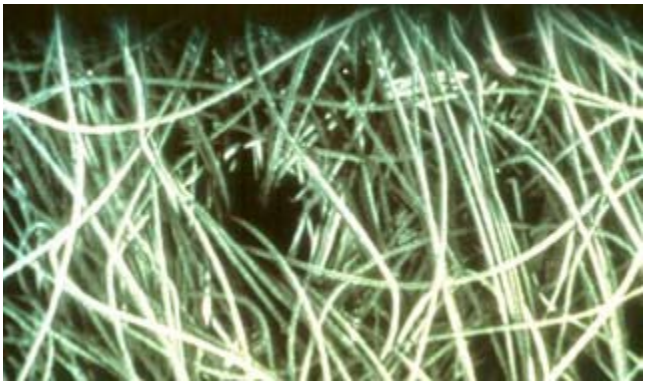
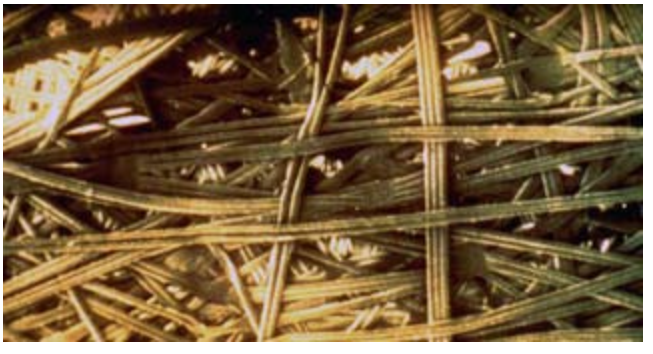


Figure TS14D–3 Heat-bonded nonwoven geotextile



rounding materials to function primarily as reinforcement (ASTM D4439). A geogrid may be biaxial (fig. TS14D–4), or uniaxial (fig. TS14D–5).

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.

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

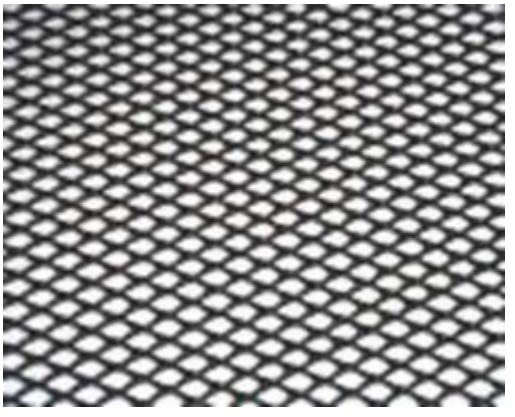


Figure TS14D–7 Geocell

Rolled erosion control products 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.

Geosynthetics may provide one or more of the following functions on a stream restoration or stabilization project.

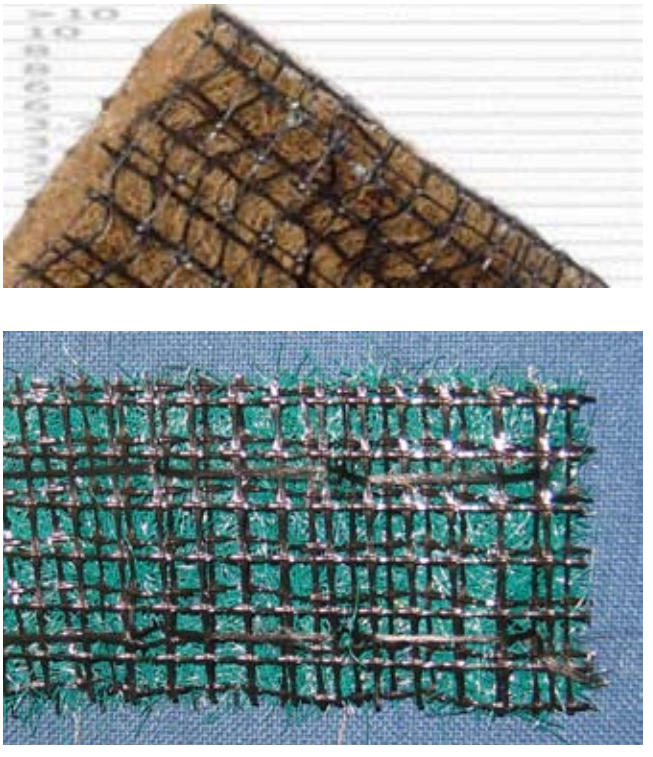
Geosynthetics used for drainage are intended to act as a conduit for fluid (typically water) within the plane of the fabric. Nonwoven geotextiles, geonets, or a composite of geotextiles and geonets are often used for this function.

Figure TS14D–8 Erosion control blanket

Filtration 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.

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

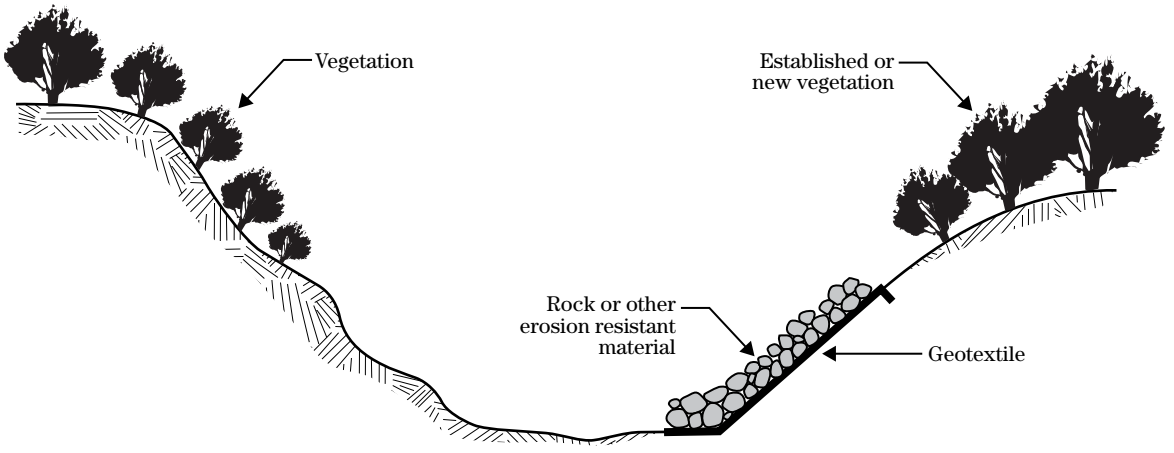


Figure TS14D–11 Reinforcement of steep streambank slope

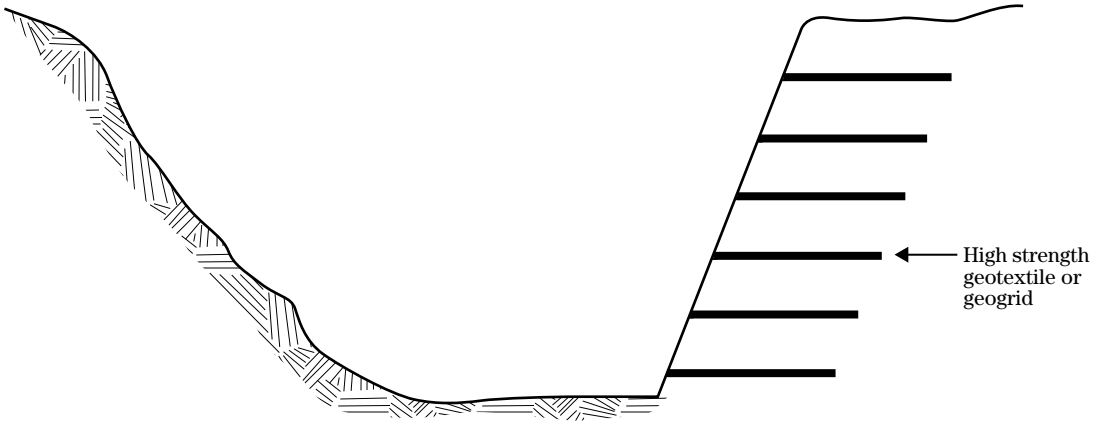


Figure TS14D–12 Mechanically stabilized earth wall

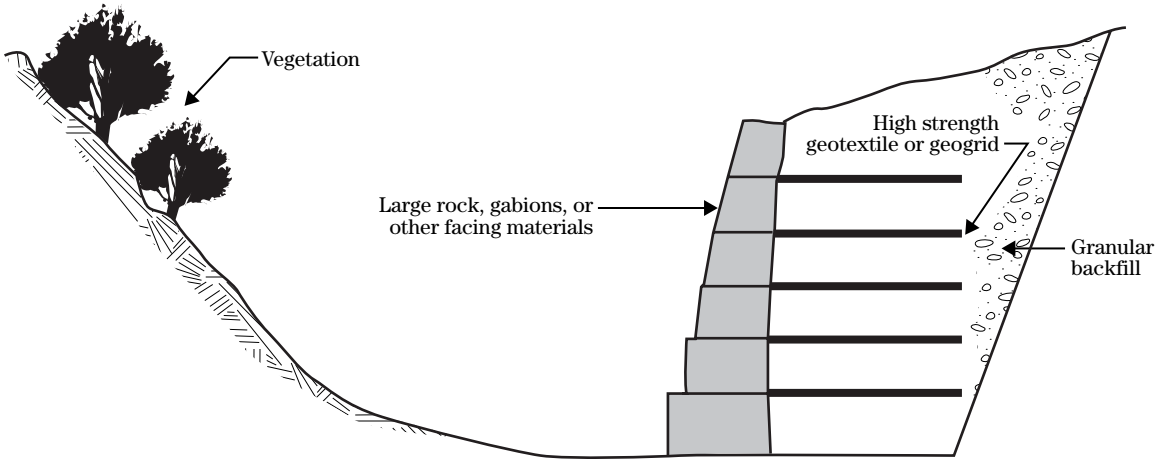


Figure TS14D–13 Earth retaining structure

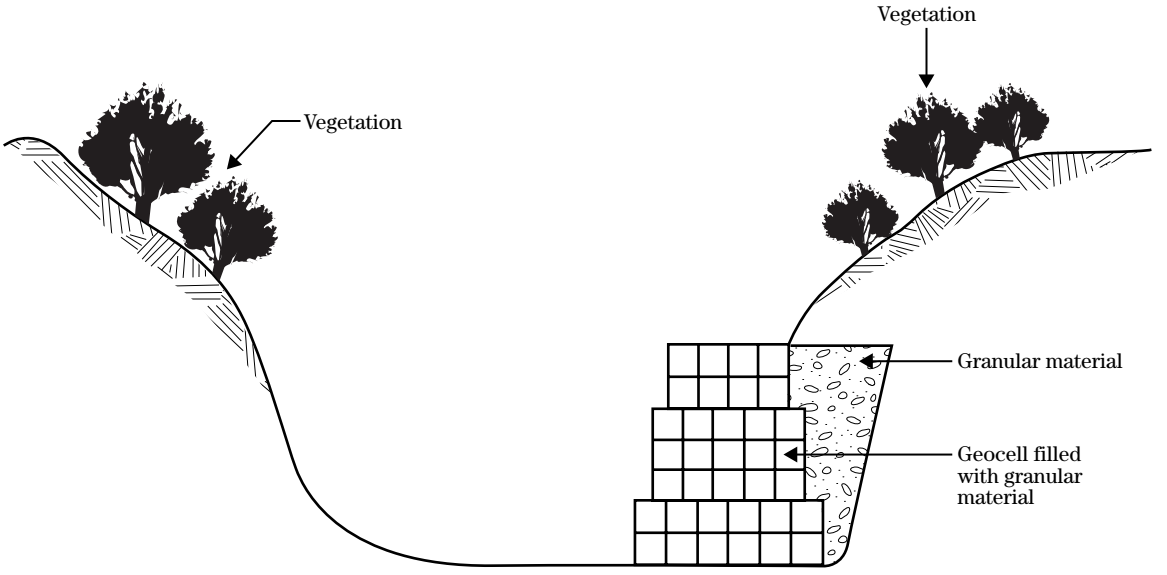
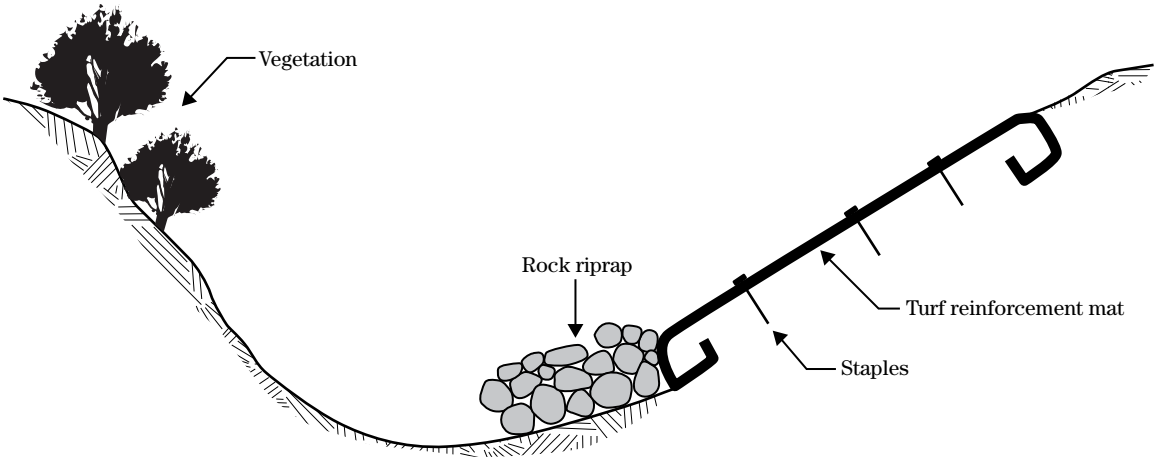


Figure TS14D–14 Erosion protection



(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.

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.

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.

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*)

***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}

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.

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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.

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.

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 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-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 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

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 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.

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

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.

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

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:

$$\frac{F_t}{\text{anchor}} = \frac{FS\left(F_d + F_b\right)}{A_n} \qquad \text{(eq. TS14E–3)}$$

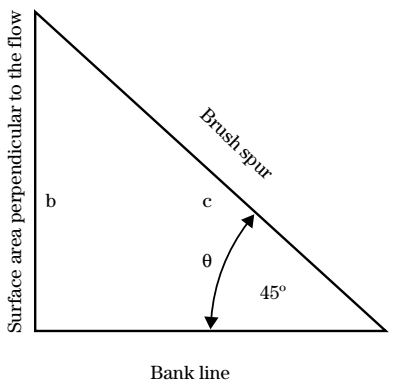
where:
FS = factor of safety
A_n = number of anchors

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).

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

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

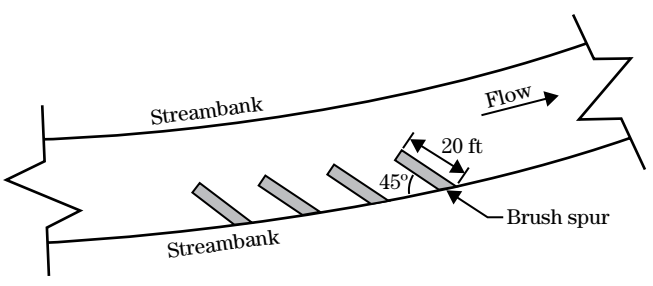


Table TS14E–1 Duckbill® specifications (rated for class 5 soils, see table TS14E–2)

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

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.

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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.

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.

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

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

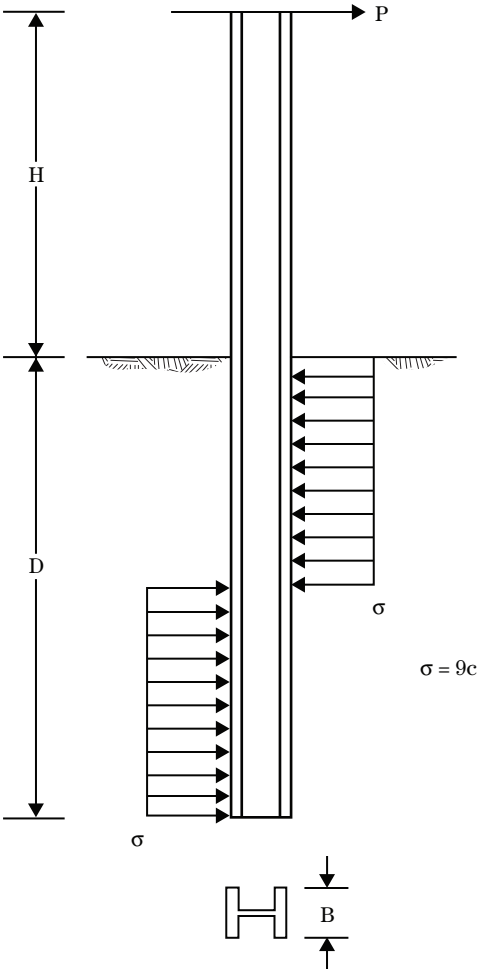


Figure TS14F–2 Schematic showing applied lateral force and assumed soil reactions for a single rigid pile driven into a cohesive soil. Uniformly decreasing load applied

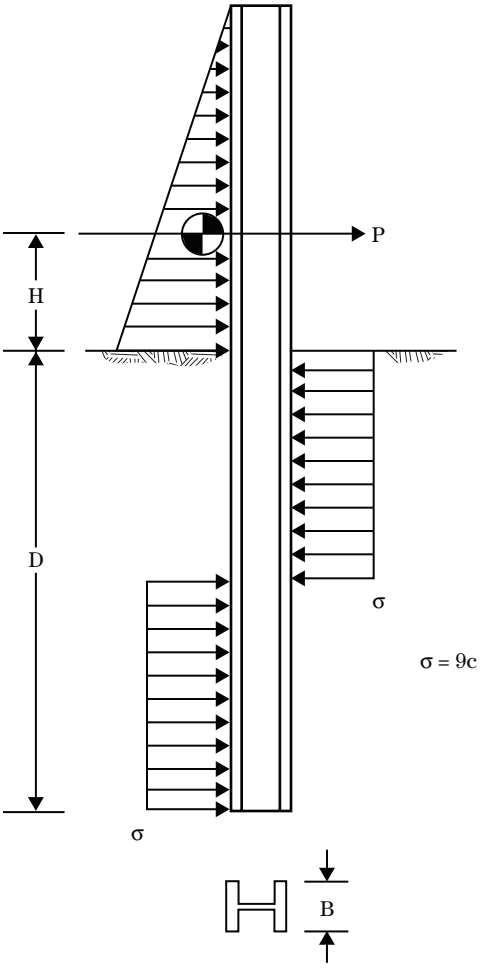


Figure TS14F–3 Schematic showing applied lateral force and assumed soil reactions for single rigid pile driven into a cohesive soil. Concentrated load applied at top of pile

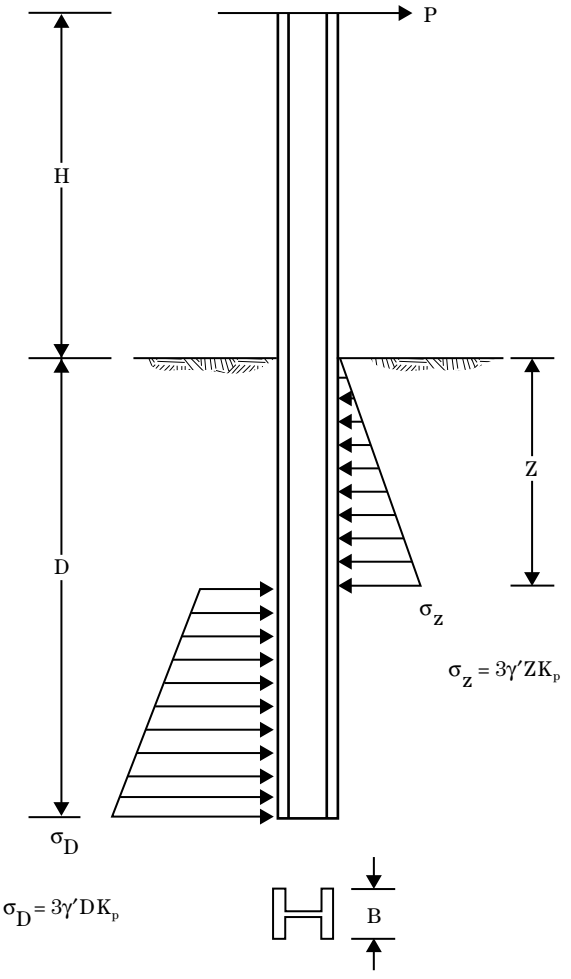
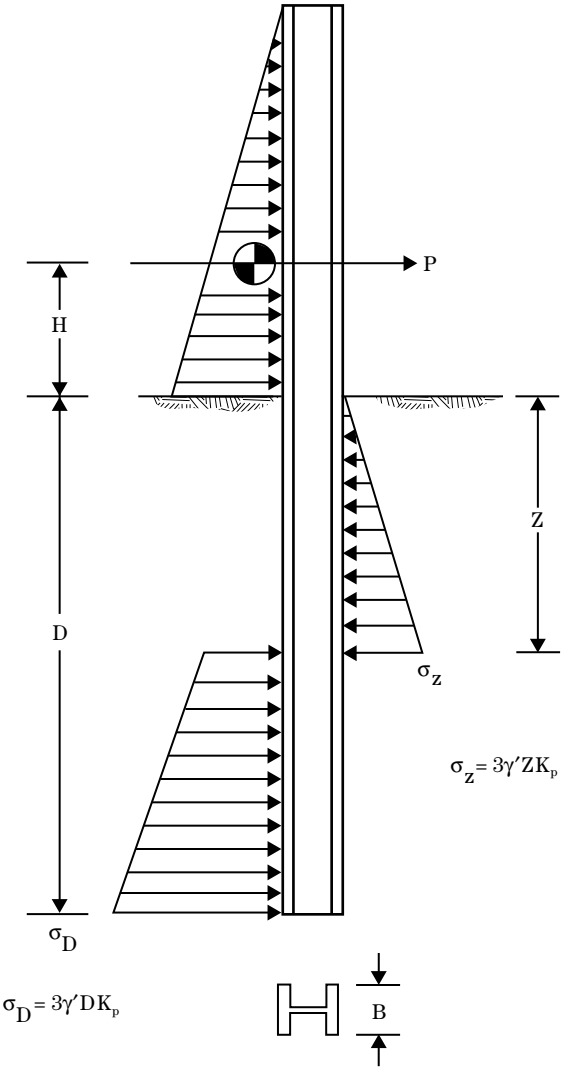
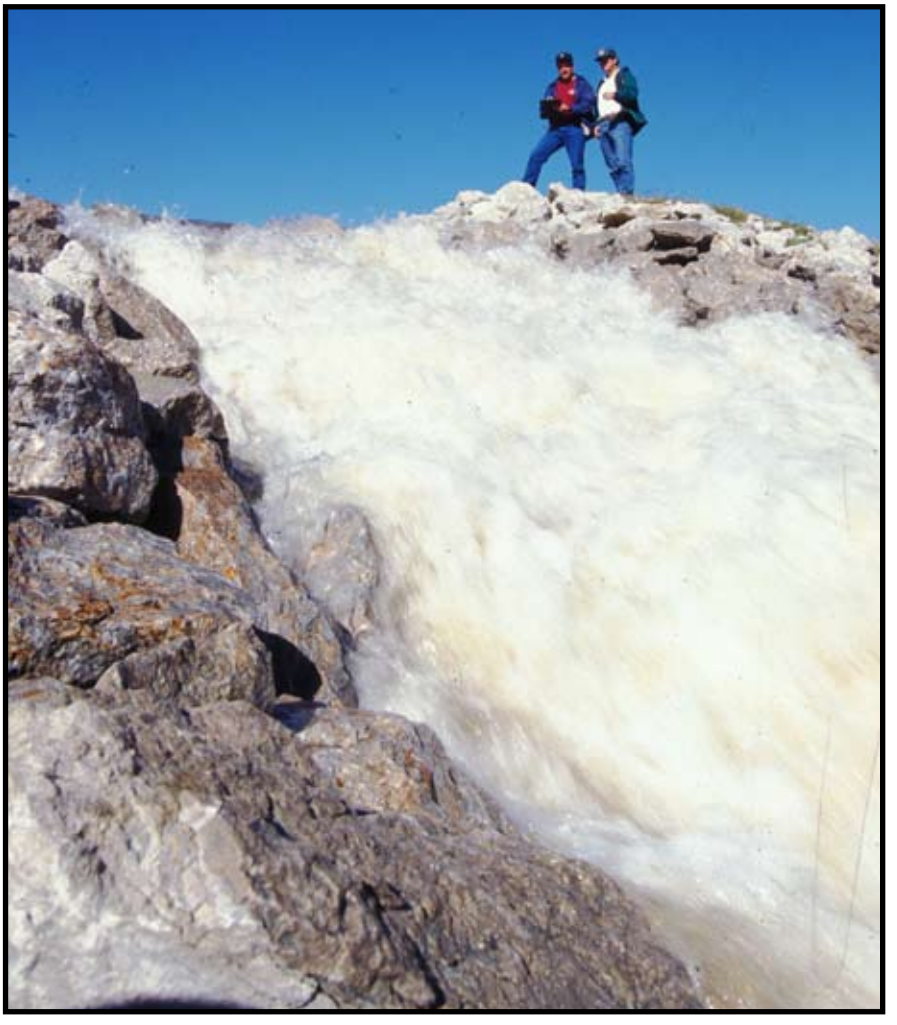


Figure TS14F–4 Schematic showing applied lateral force and assumed soil reactions for single rigid pile driven into a cohesive soil. Uniformly decreasing load applied as shown





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.

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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.

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.

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})

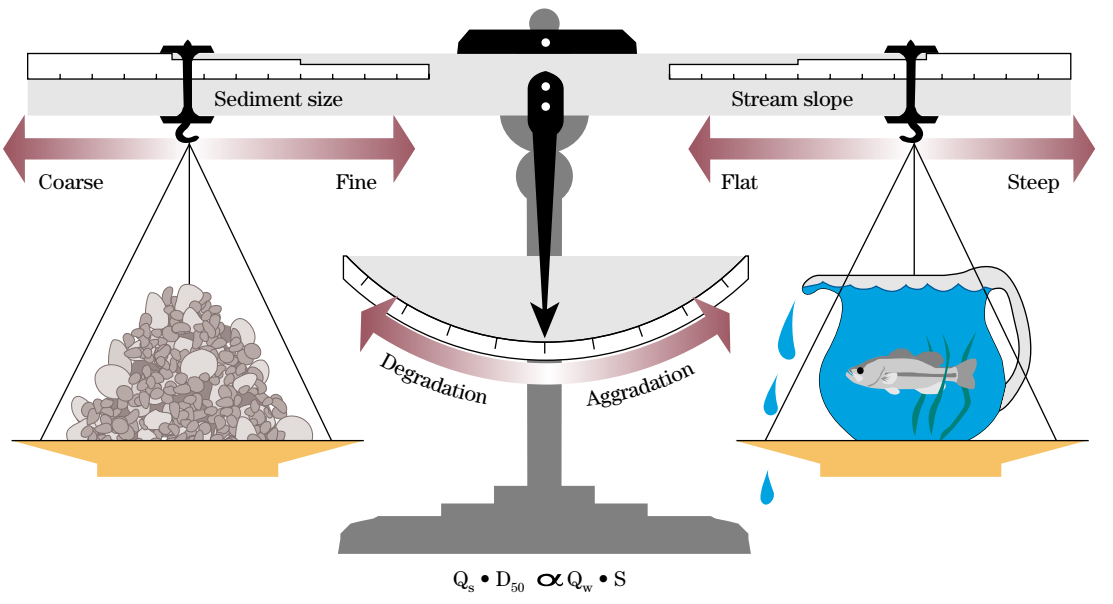


Figure TS14G–2 An energy diagram for the preconstruction condition

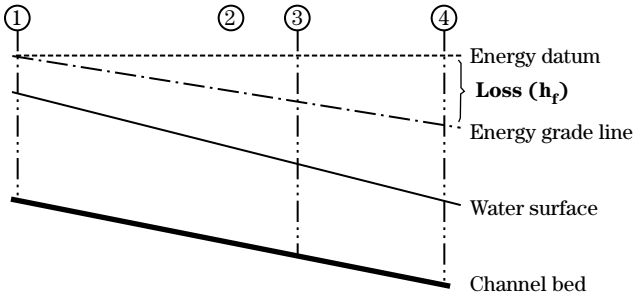


Figure TS14G–3 The modified energy diagram (shown in red) for a bed control structure

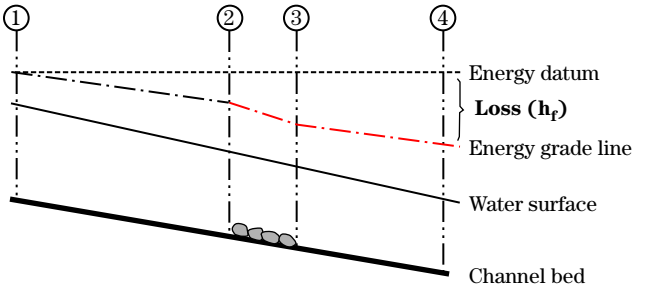
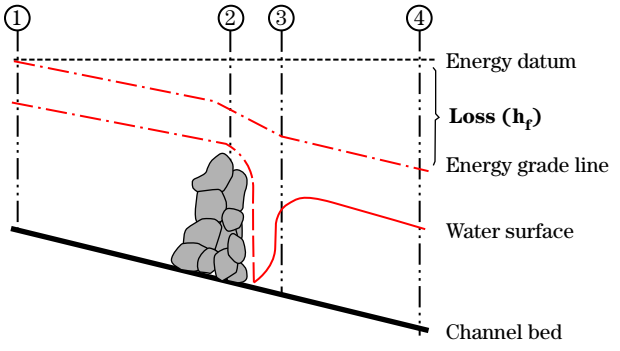


Figure TS14G–4 The modified energy diagram (shown in red) for a hydraulic control structure



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

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

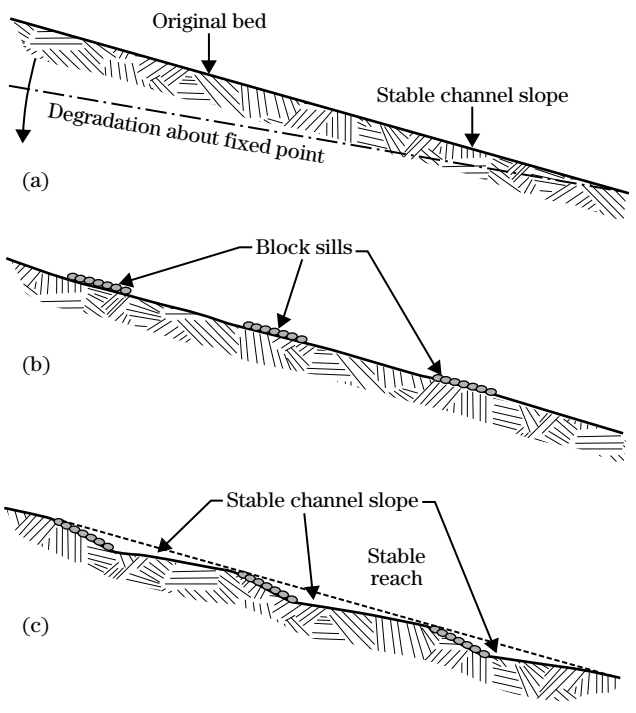
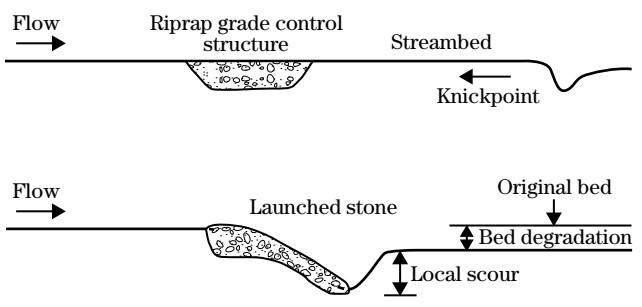


Figure TS14G–6 Top—riprap grade control structure; Bottom—subsequent launching of riprap at the grade control structure in response to advancing bed degradation and local scour

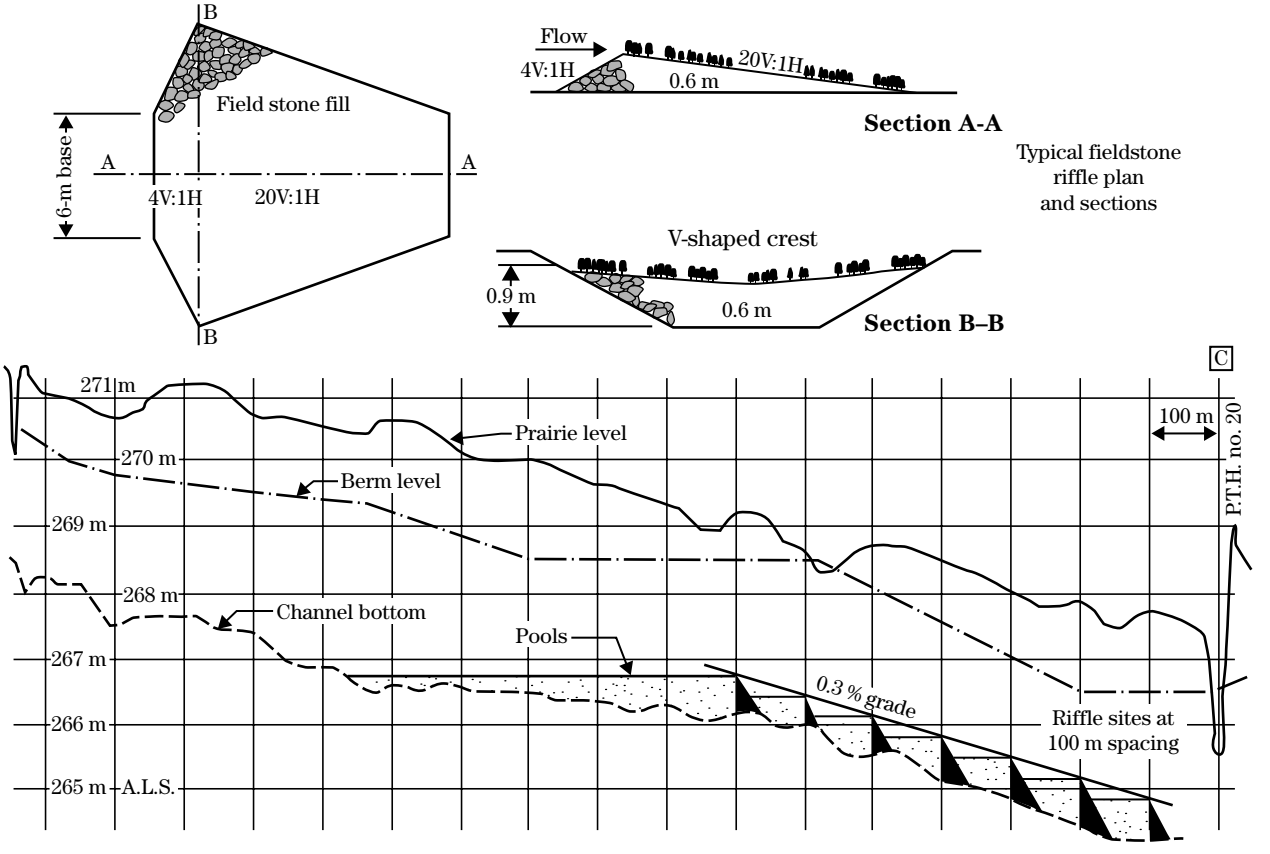


natural riffles. For the Mink Creek example shown in figure TS14G–7 (Newbury 2002), the structures were designed to a height of 0.6 meter that would impound shallow pools for passage of young walleye fry. No cutoff walls or filters were used in this installation, but the structure was sealed by infilling 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



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)

$$\text{tractive force (kg/m}^2\text{)} = \text{incipient diameter (cm)} \qquad \text{(eq. TS14G–5)}$$

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:

$$q = 0.52\, D_{50}^{1.89}\, S_o^{-1.50} \text{ for } S_o < 0.10 \qquad \text{(eq. TS14G–6)}$$

$$q = 4.30\, D_{50}^{1.89}\, S_o^{-1.50} \text{ for } 0.10 < S_o < 0.40 \qquad \text{(eq. TS14G–7)}$$

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:

$$\text{minimum rock size (m)} = 0.1724 \ln(\text{bankfull shear stress, kg/m}^2) + 0.6349 \qquad \text{(eq. TS14G–8)}$$

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.

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

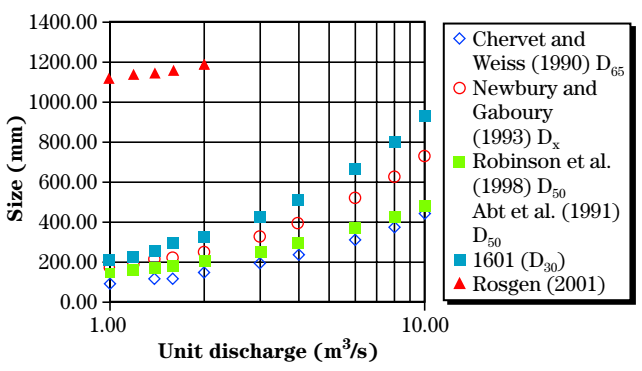
with the other methods. If the sloping loose rock structures are to be constructed in a location that will encounter completely submerged conditions, traditional riprap-sizing methods (USACE 1994f; U.S. Department of Transportation Federal Highway Administration (FHWA) 2001a) should be used to check 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.

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.

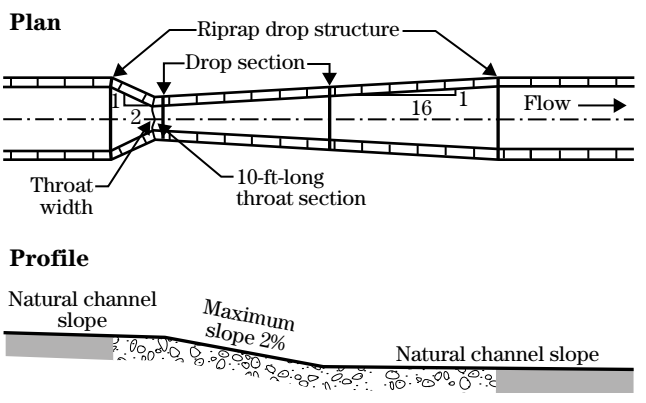
Grade control can also be accomplished by lining the streambed with an erosion resistant material. These structures are designed to ensure that the drop is accomplished over a specified stream reach that has been lined with riprap or some other erosion-resistant material. Rock riprap gradient control 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).

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.

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).

Figure TS14G–11 Rock riprap gradient control structure



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–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

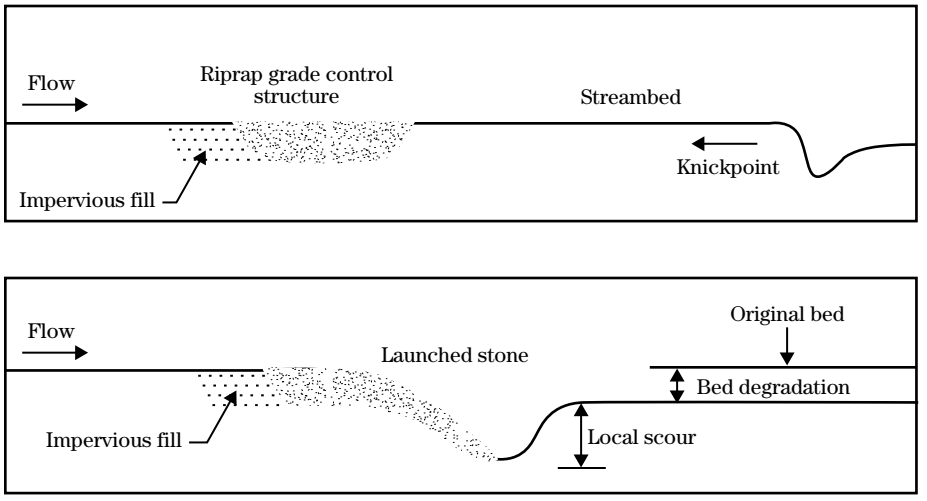


Figure TS14G–13 Top—built riprap grade control structure with a sheet pile cutoff wall (*top*); Bottom—launching of riprap at the grade control structure in response to bed degradation and local scour

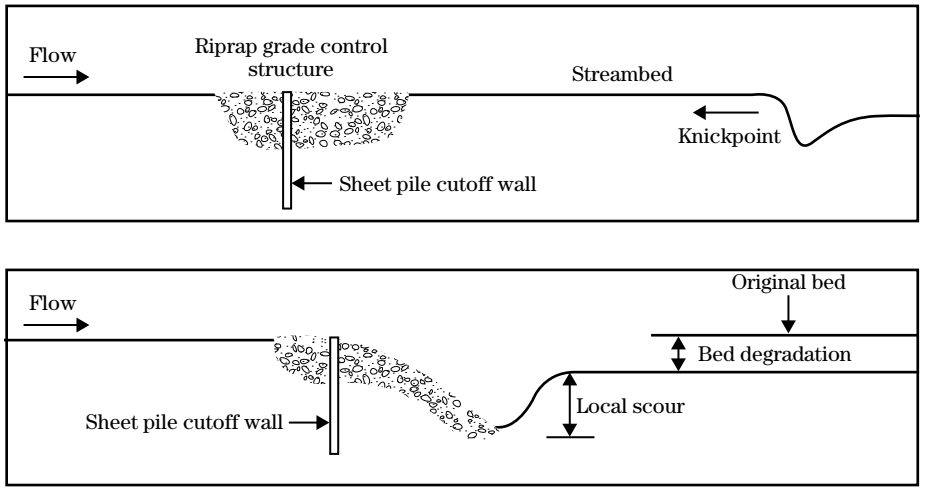
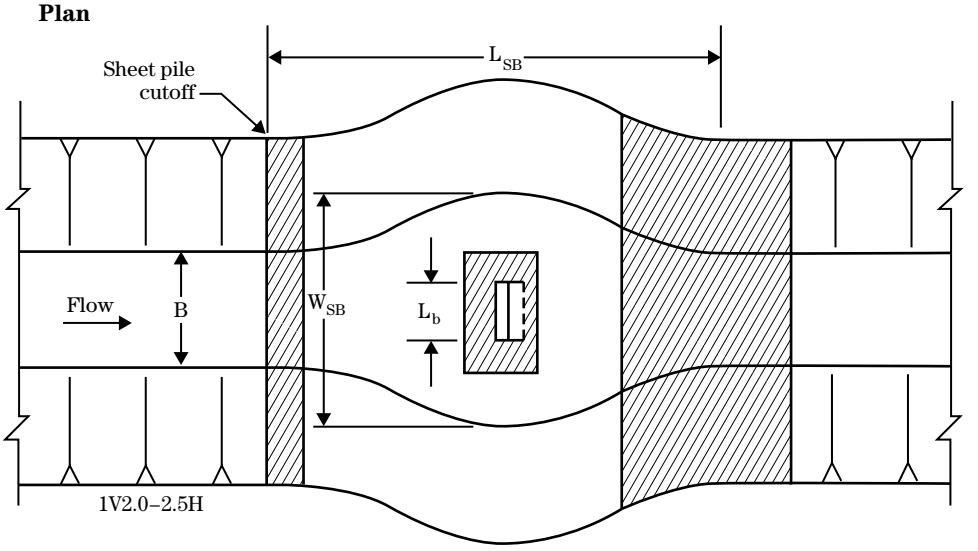
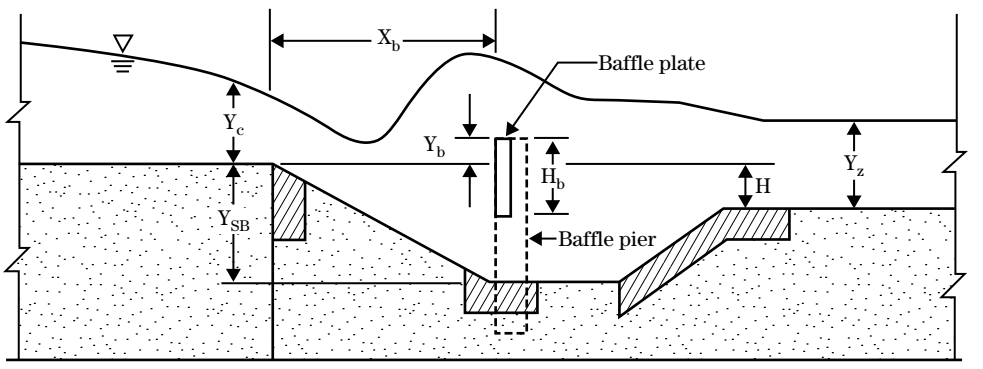


Figure TS14G–14 ARS-type grade control structure with preformed riprap-lined stilling basin and baffle plate

Plan



Profile



- 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.

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

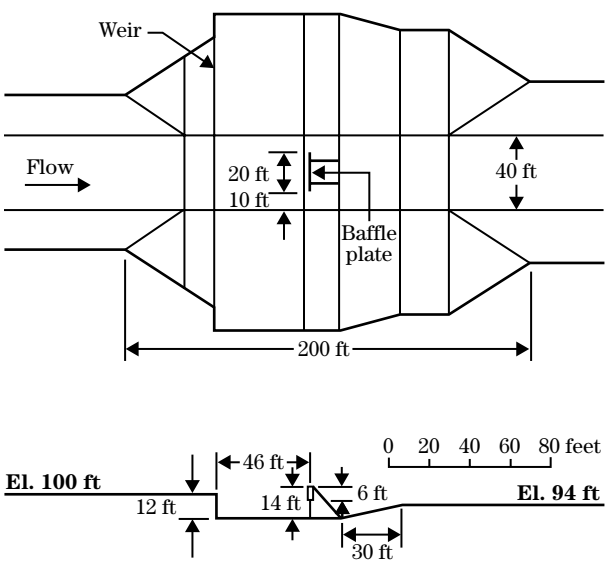


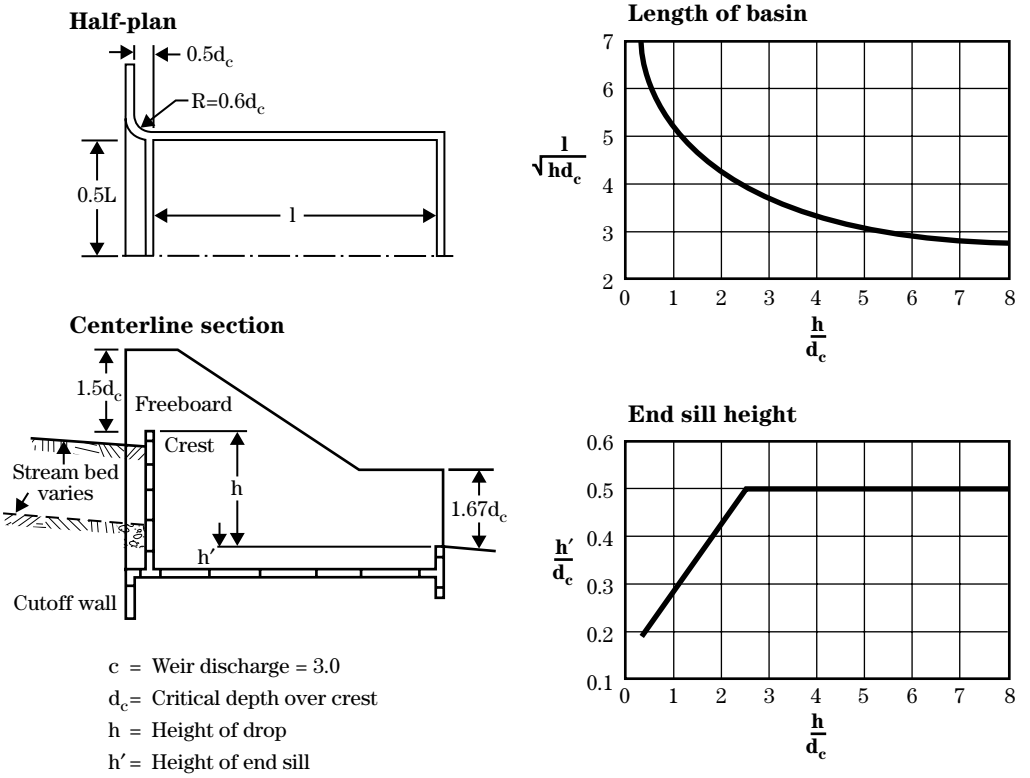
Figure TS14G–16 ARS-type grade control structure with grouted riprap face

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.

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.

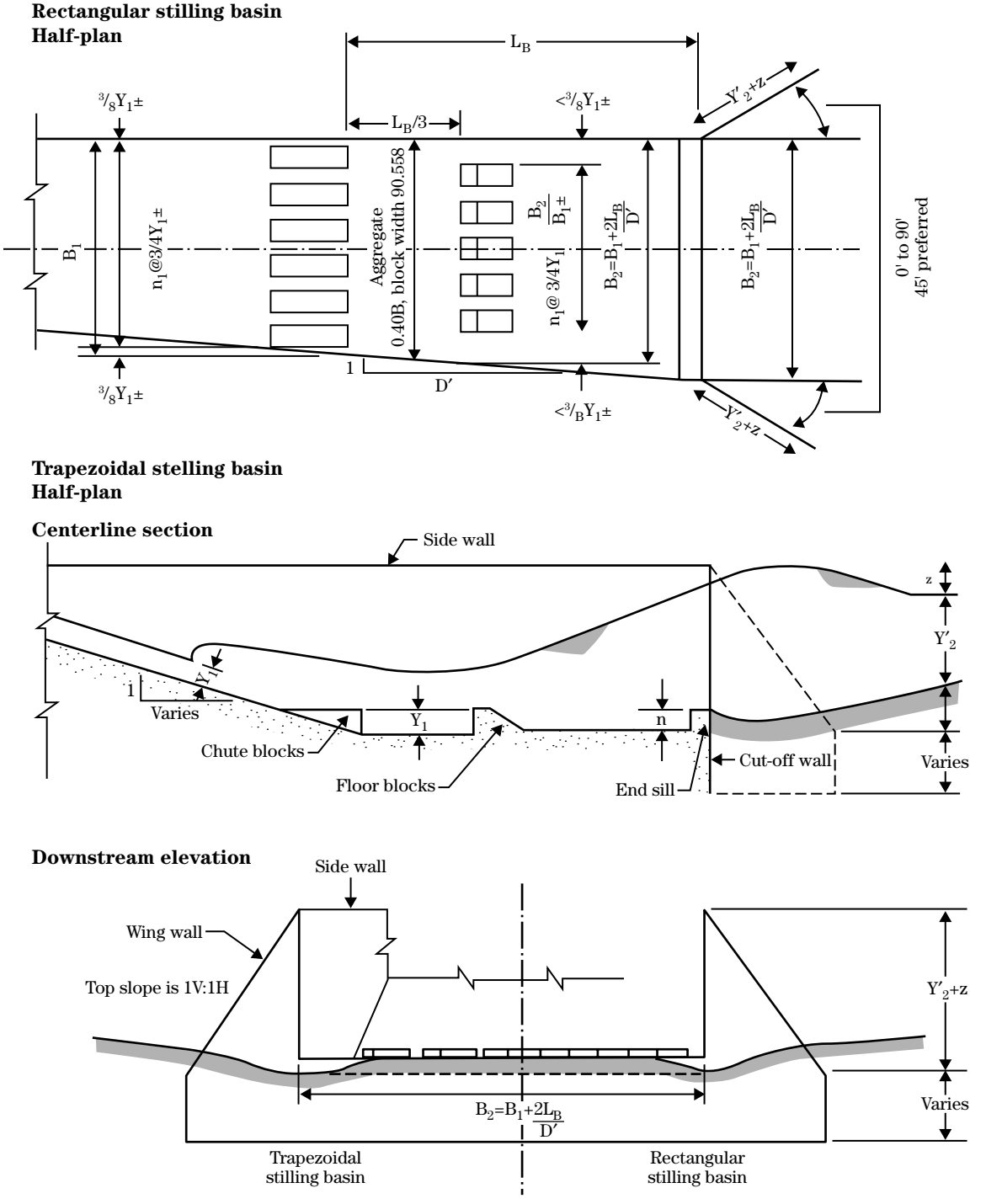
Figure TS14G–17 CIT-type drop structure



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.

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.

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:

$$N = \frac{H}{h} \qquad \text{(eq. TS14G–11)}$$

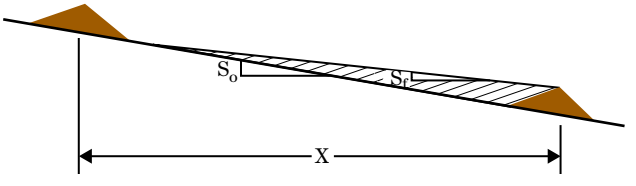
where:
h = selected drop height of the structure

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



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

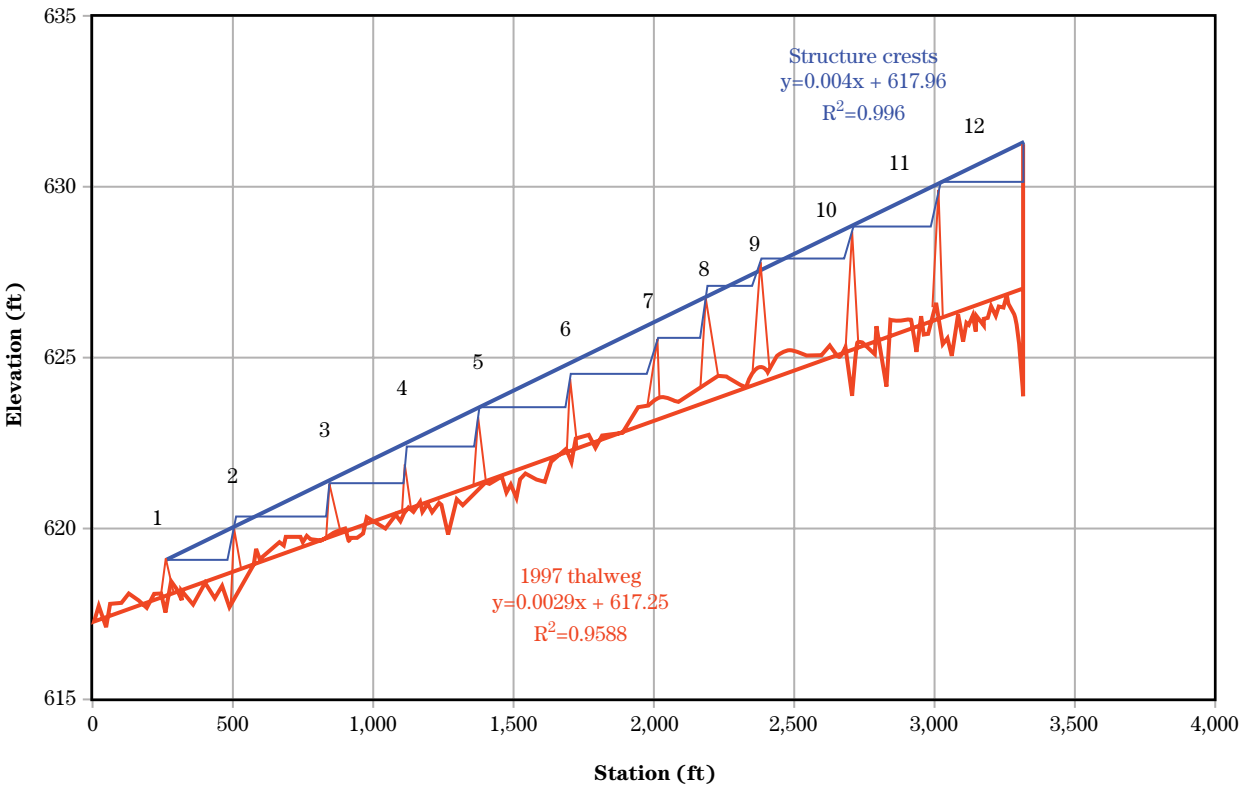


Figure TS14G–21 Blue Creek, IL, thalweg profile surveyed in 2002

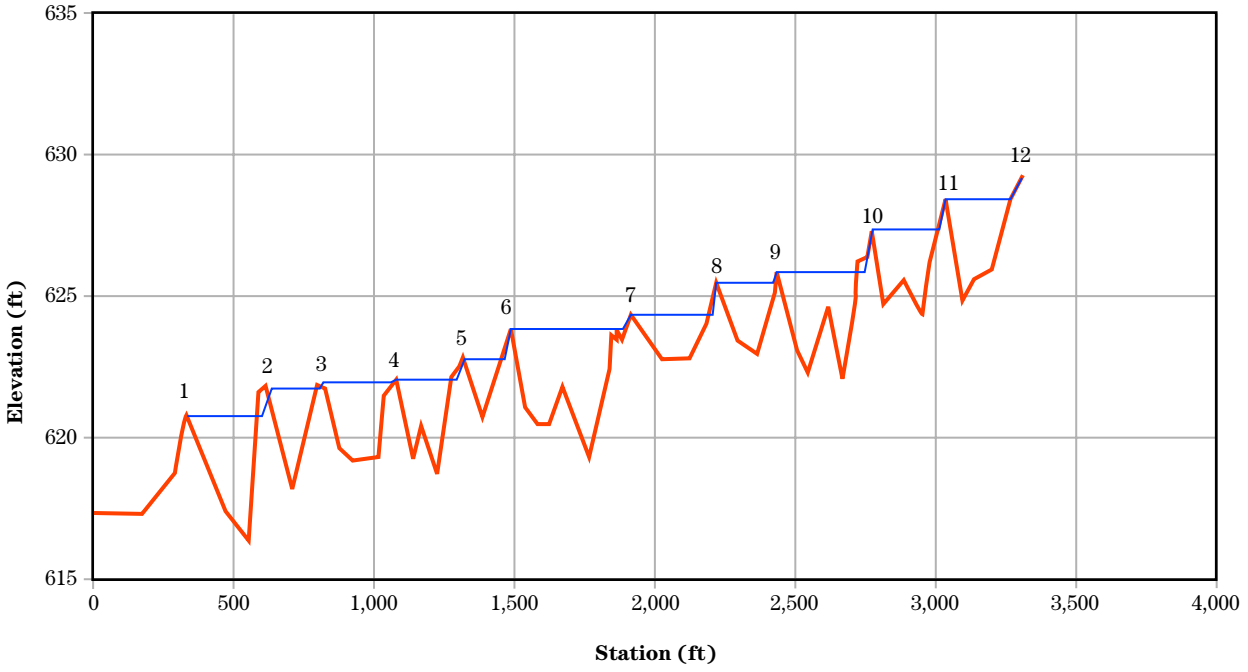


Figure TS14G–22 Grade control structure 1 month and 18 months after construction (Blue Creek, IL)

Figure TS14G–23 Grade control design (Blue Creek, IL)

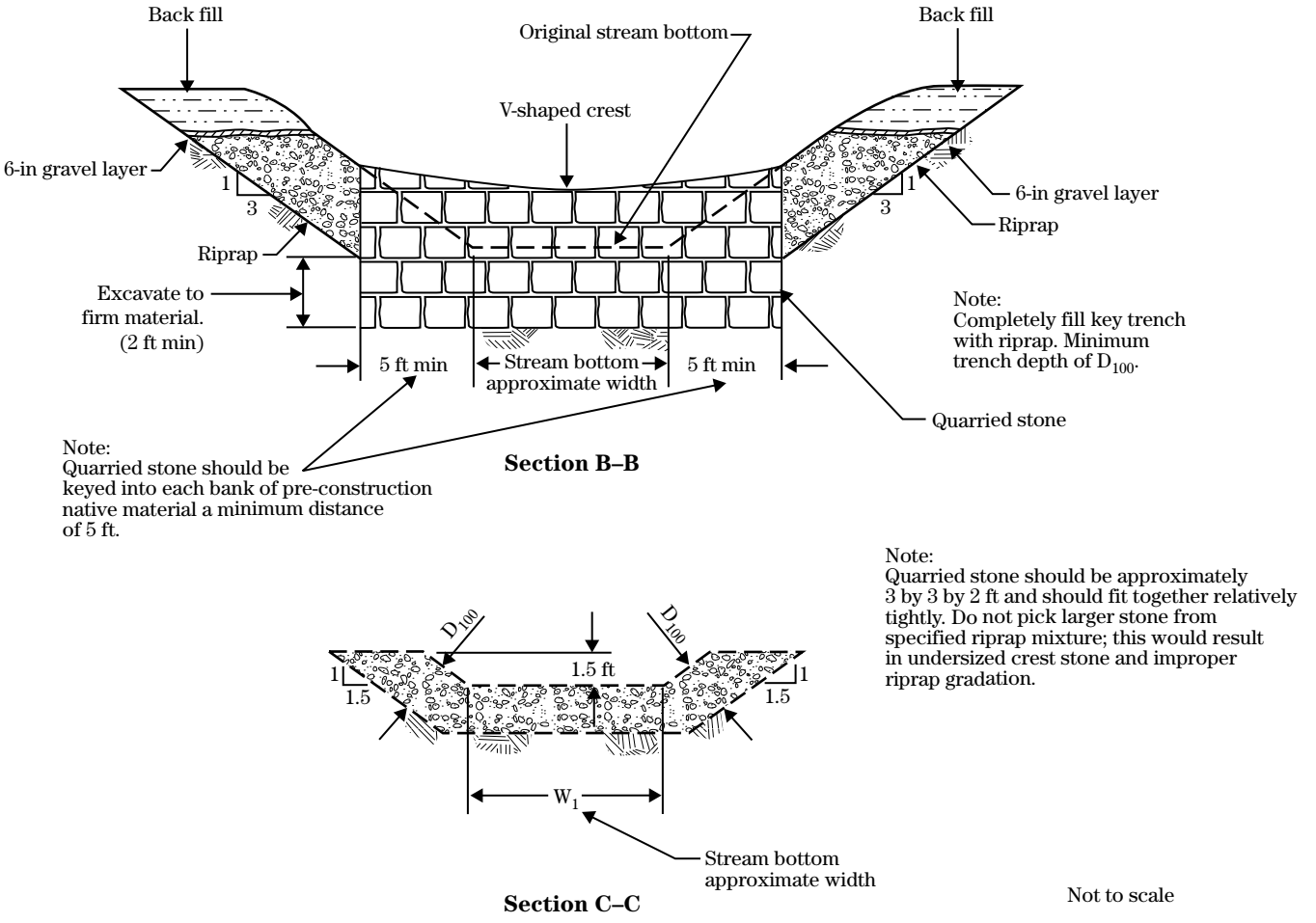


Figure TS14G–24 Grade control design (Blue Creek, IL)

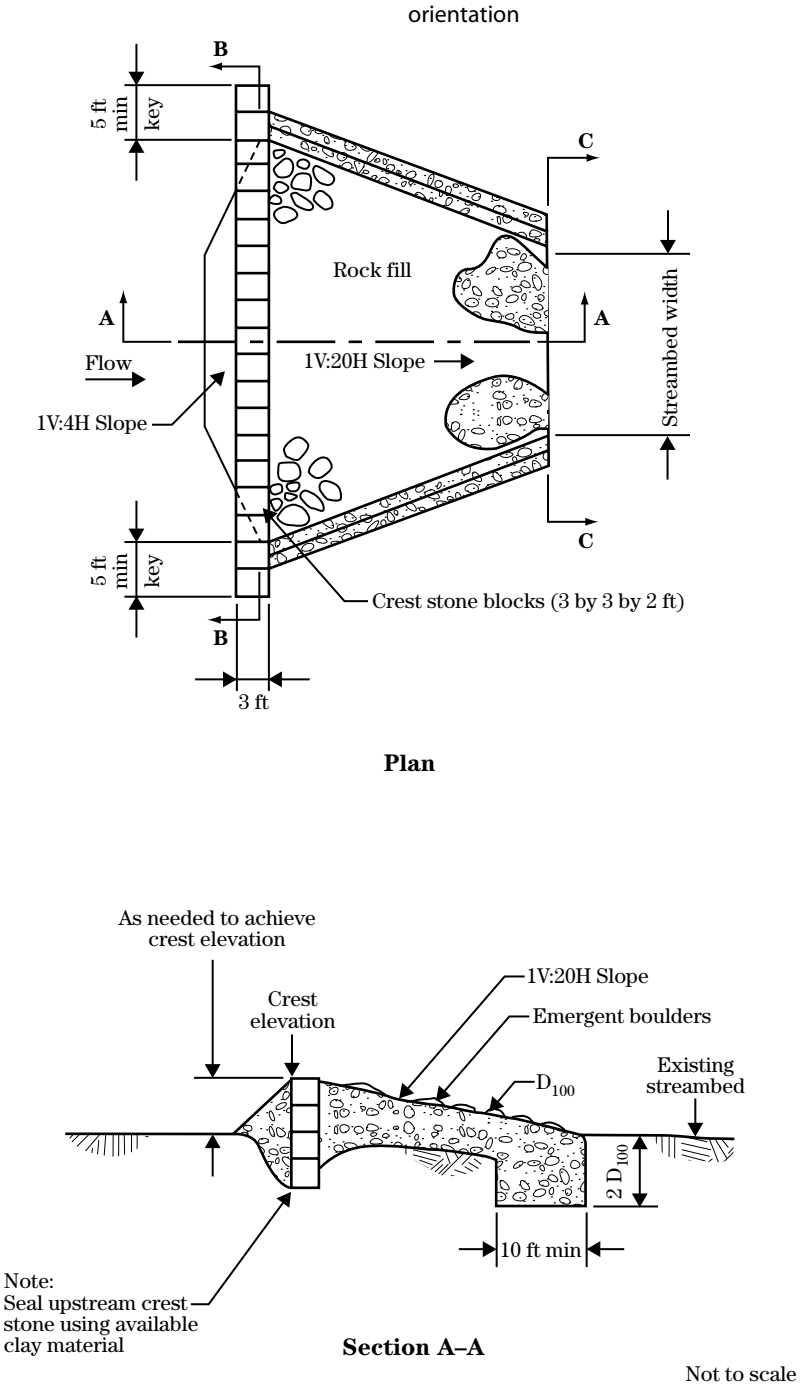
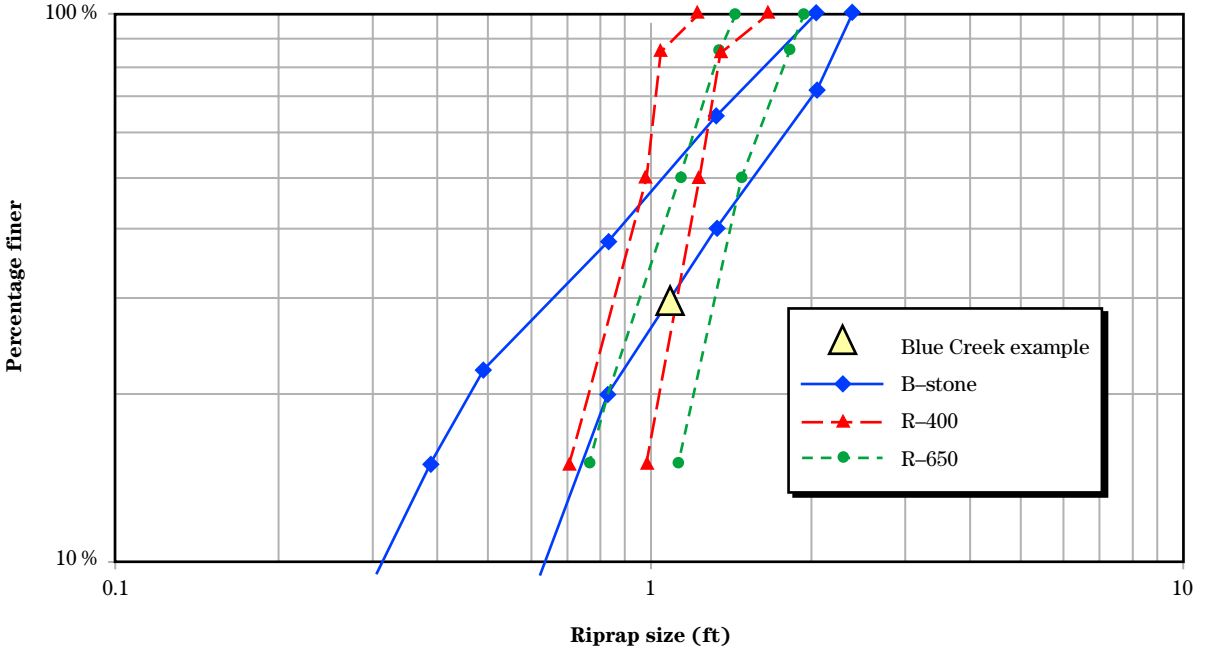


Figure TS14G–25 Riprap gradations for B-Stone, R–400, R–650, and D_{30} from the Blue Creek example



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.

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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.

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.

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 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.

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 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)

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 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 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)

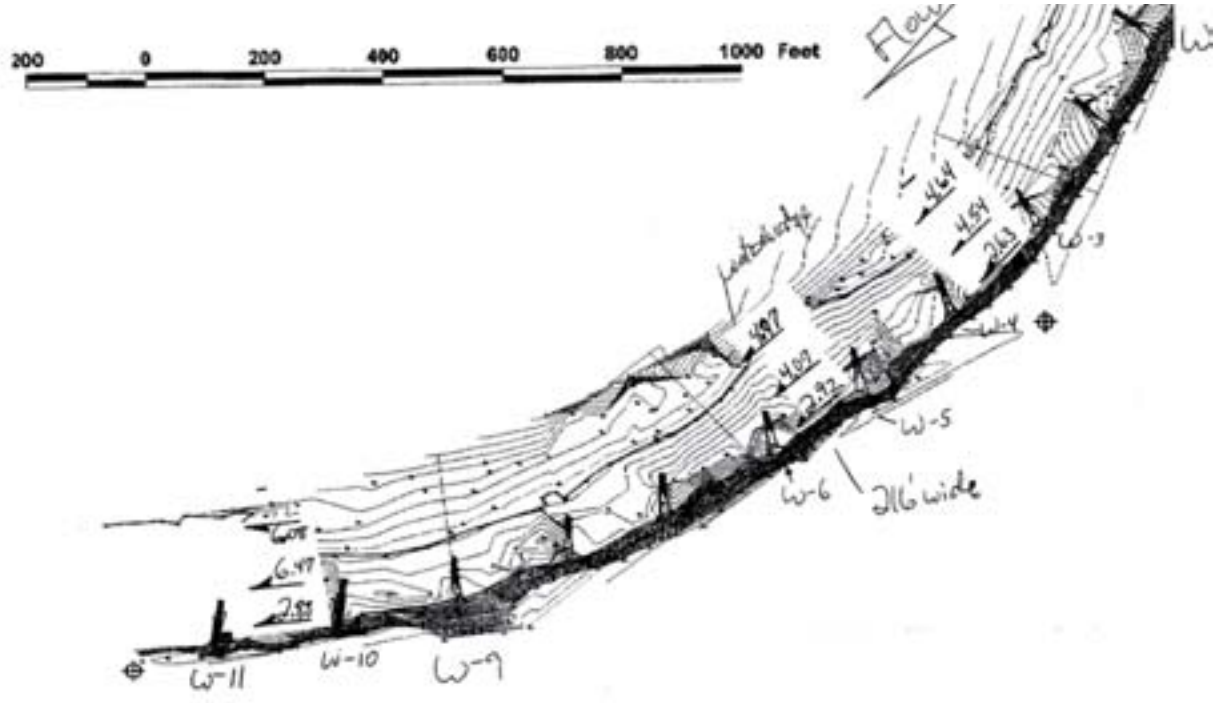
(b)

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 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.

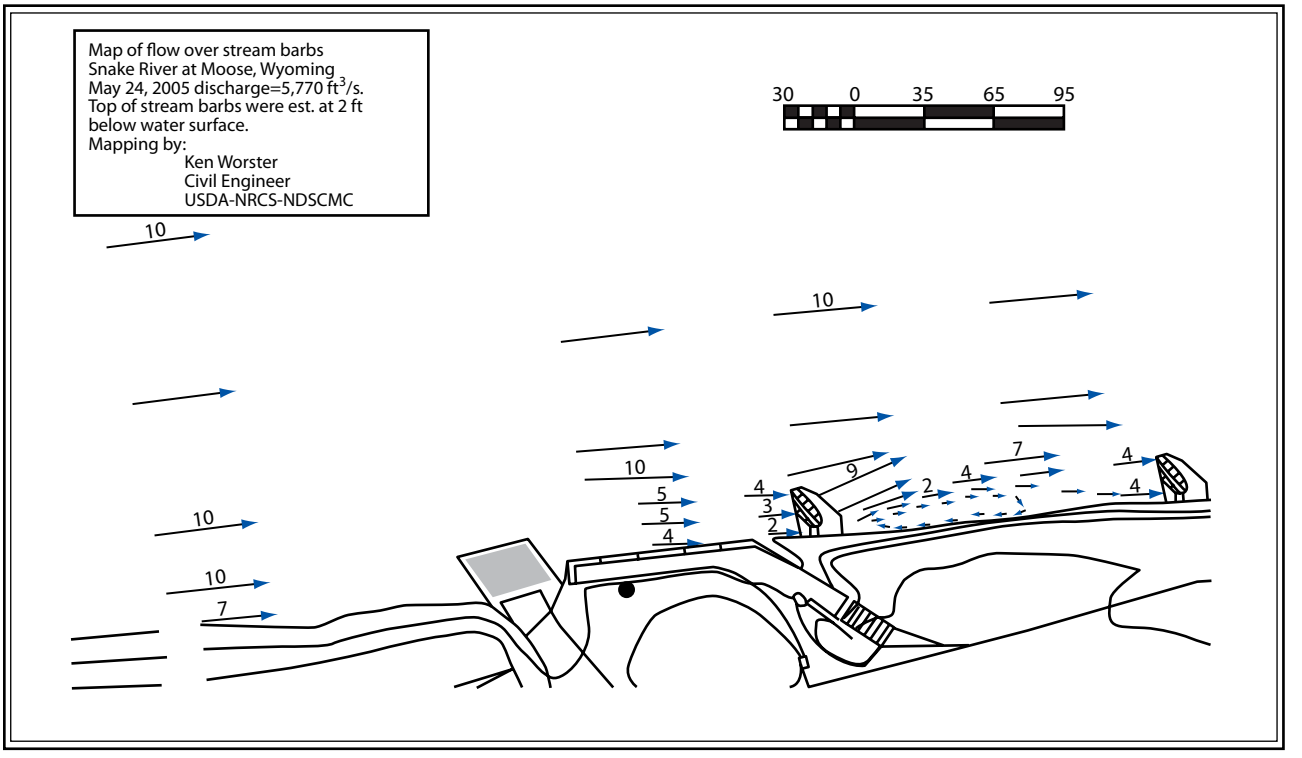
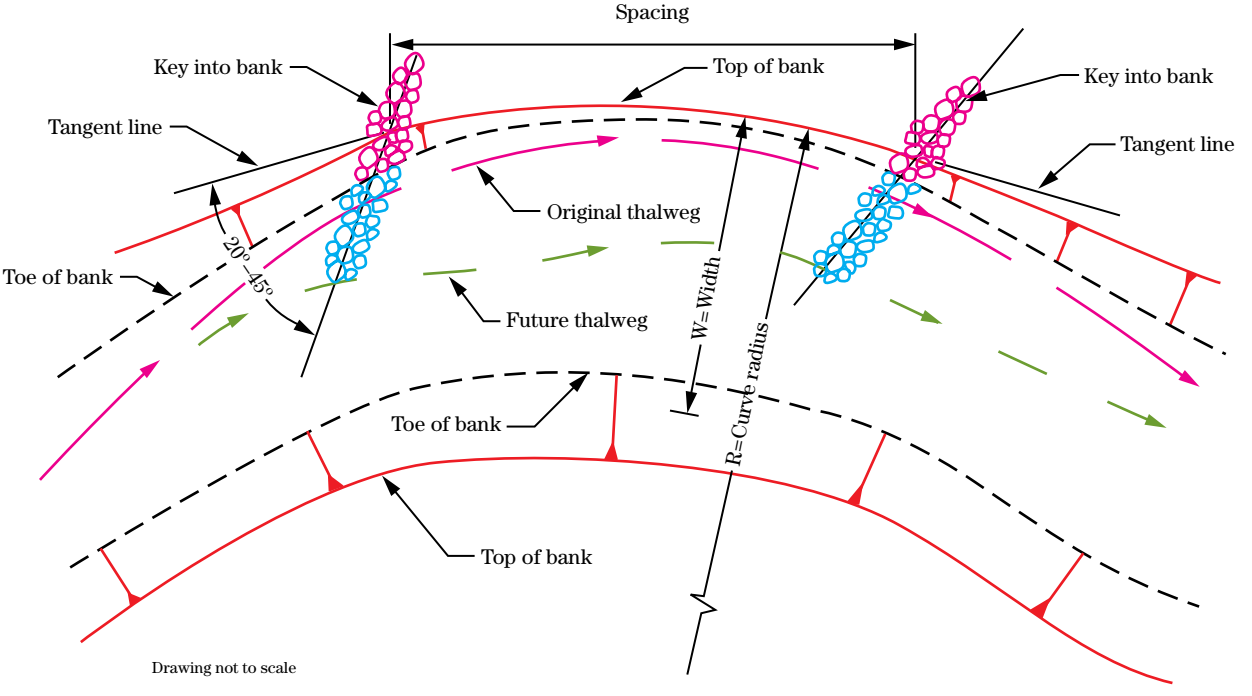


Figure TS14H–8 Typical stream barb design layout



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: $L_e = \frac{W}{4}$

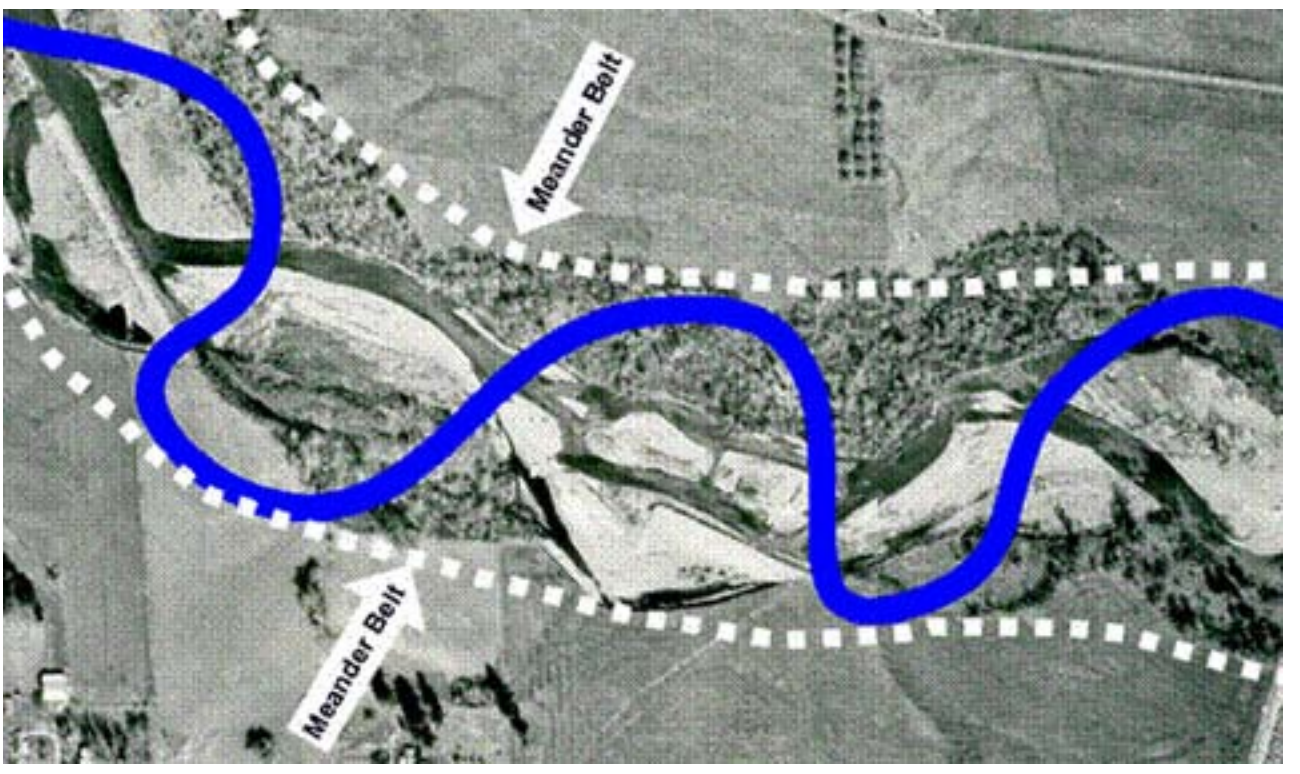
$$L_W = \frac{L_e}{\sin\theta}$$

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.

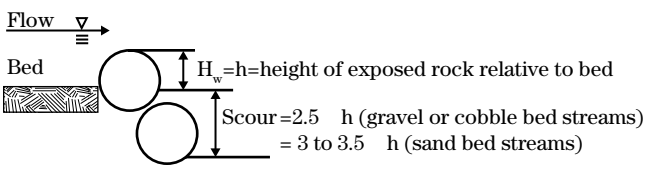
Width—The width of a stream barb generally ranges from one to three times the design D_{100} rock size. The width does not need to be more than two rock diameters and can even be the width of a single large rock at the tip of the barb. However, stream barbs with a top width of a single stone have been 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

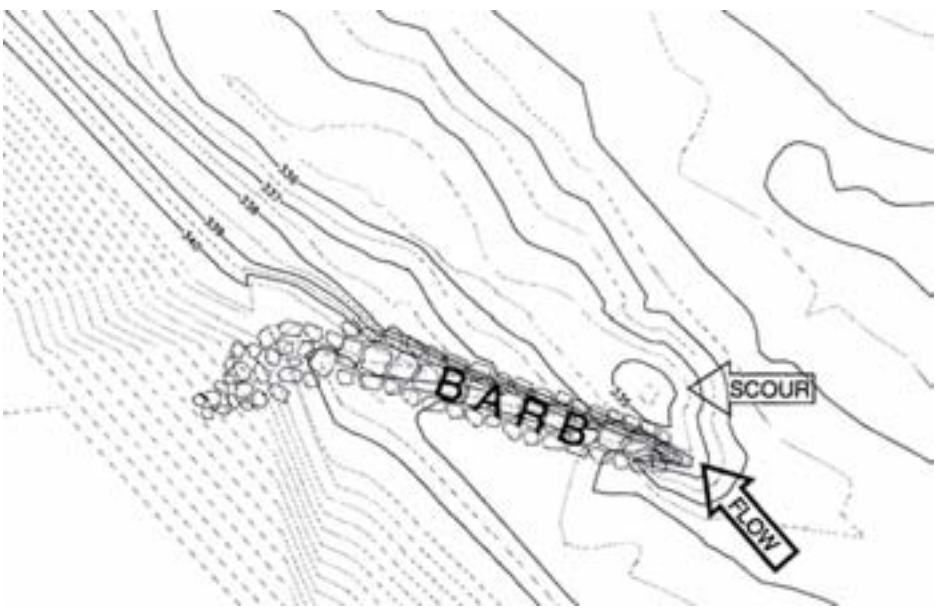
$D_{minimum}$ = 0.75 × D_{50}, as determined for streambank riprap

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.

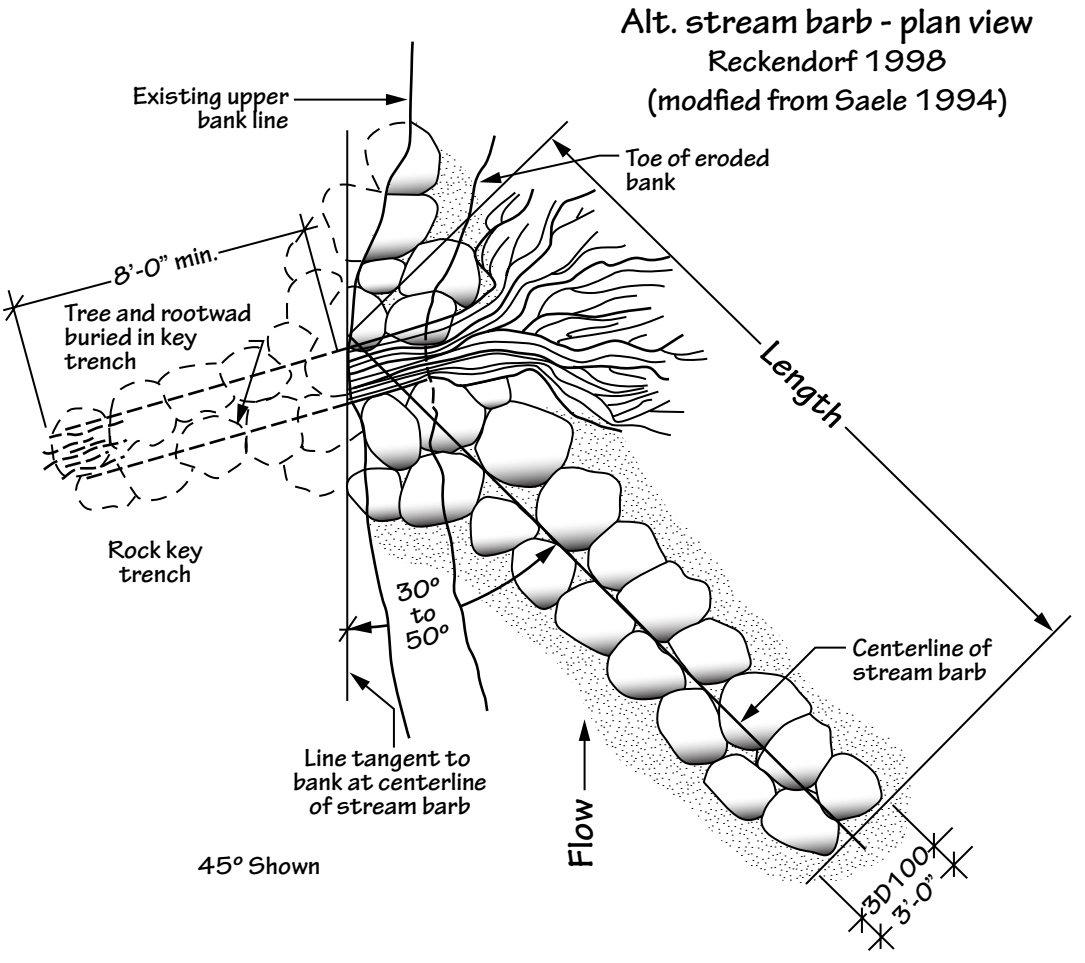
Figure TS14H–11 Scour effects at the barb tip



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.

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.

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.

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.

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

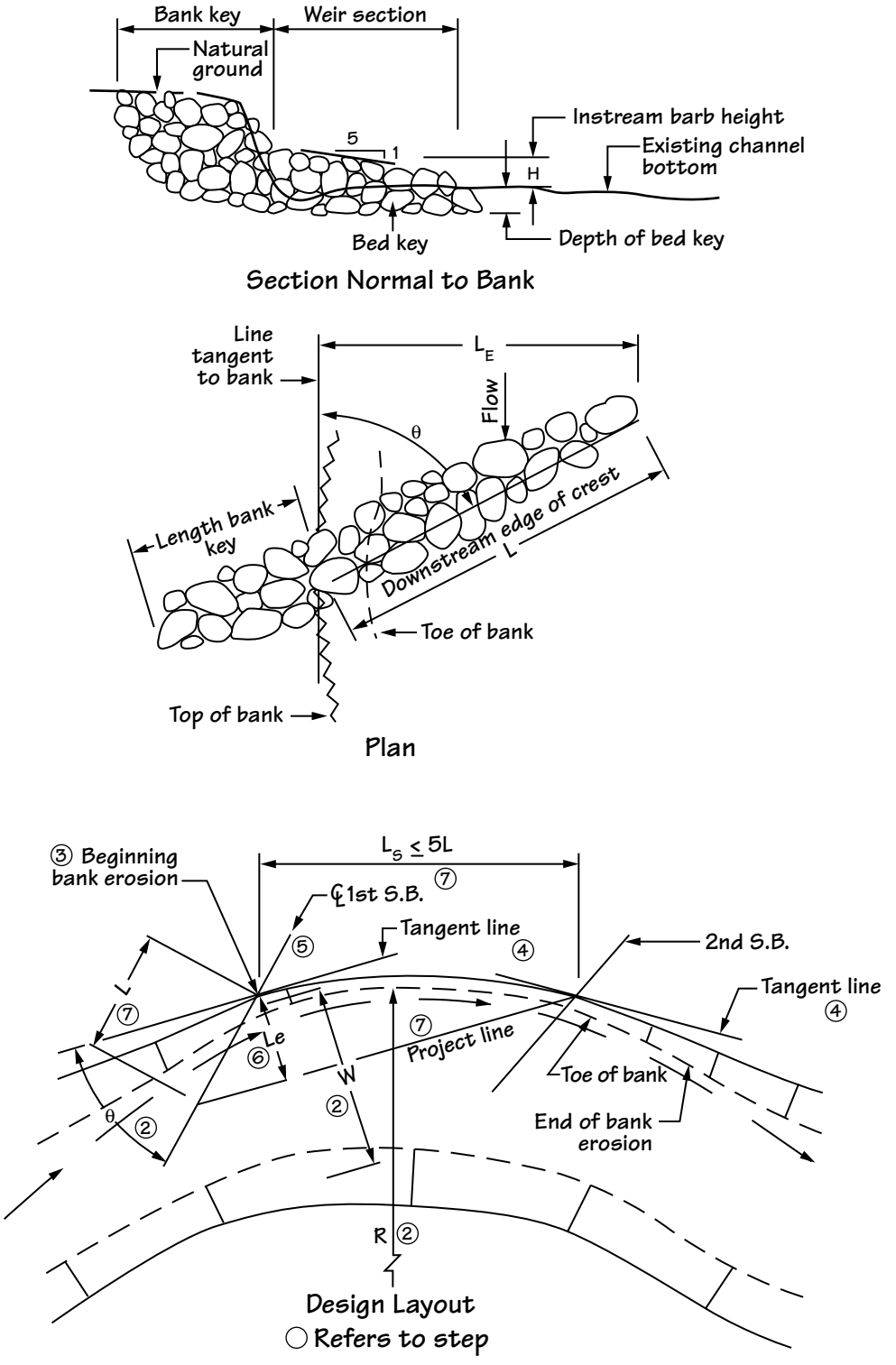


Figure TS14H–14 Typical stream barb construction drawing

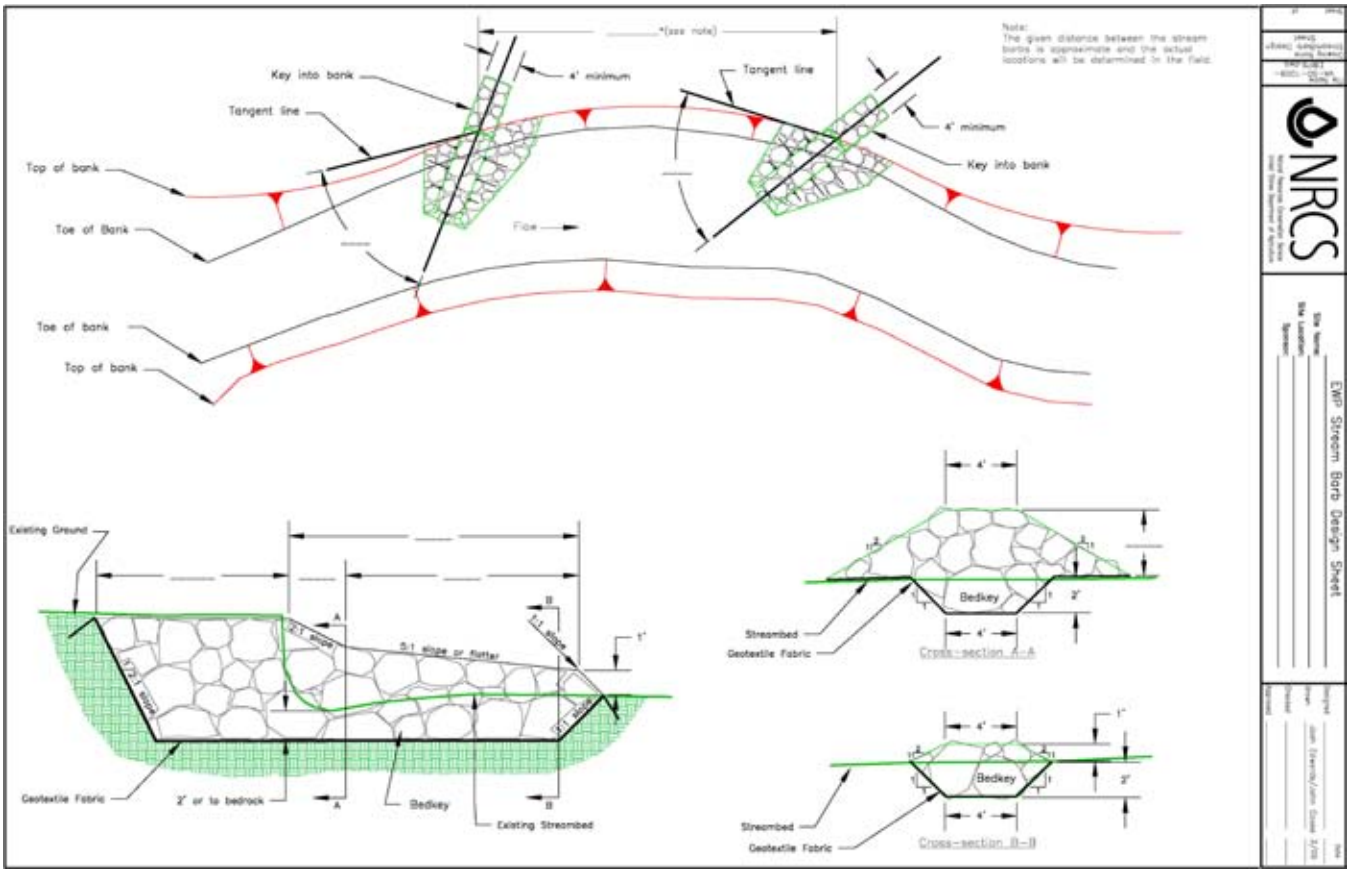
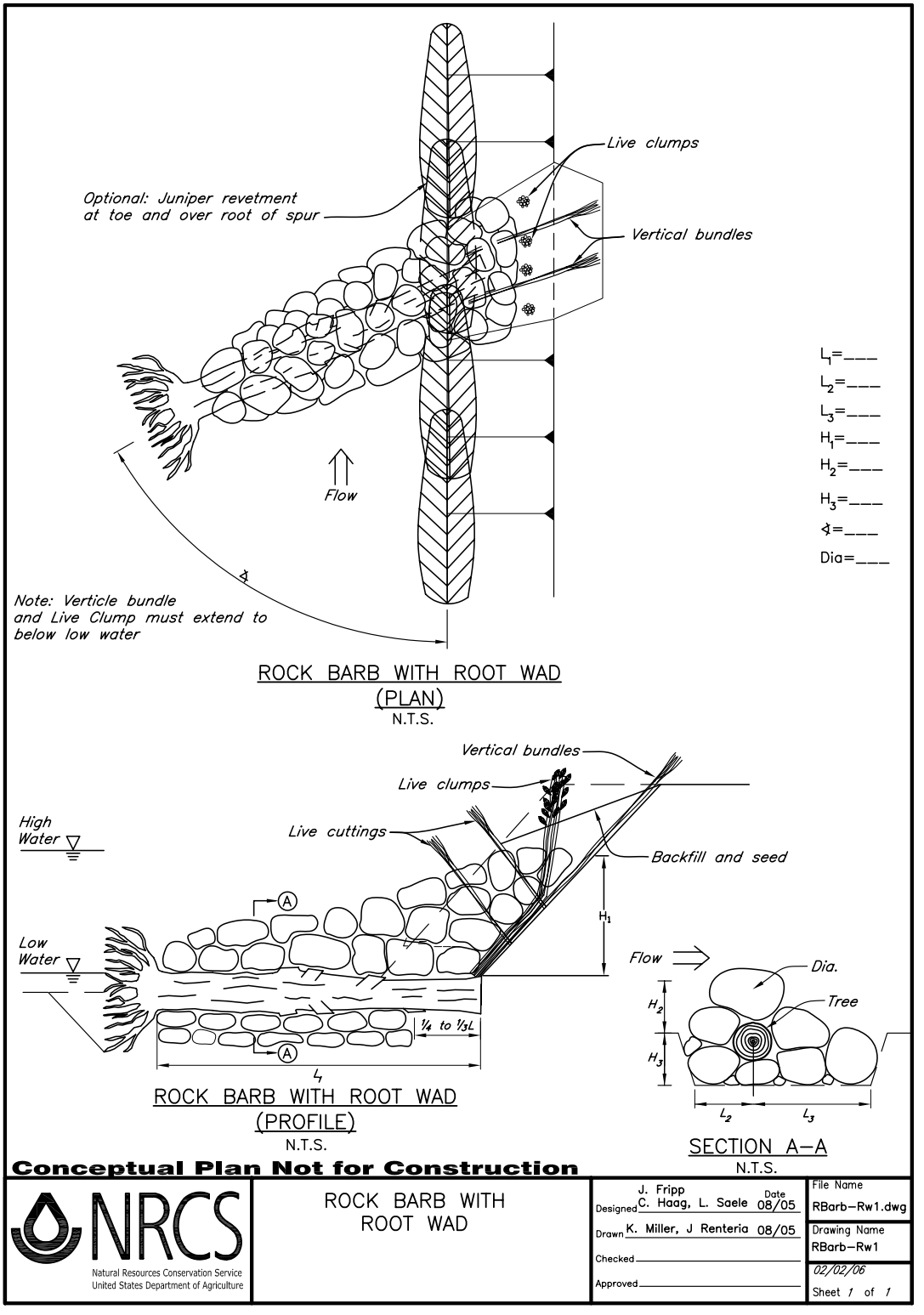


Figure TS14H–15 Detail showing the use of a rootwad incorporated into a stream barb



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.

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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.

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.

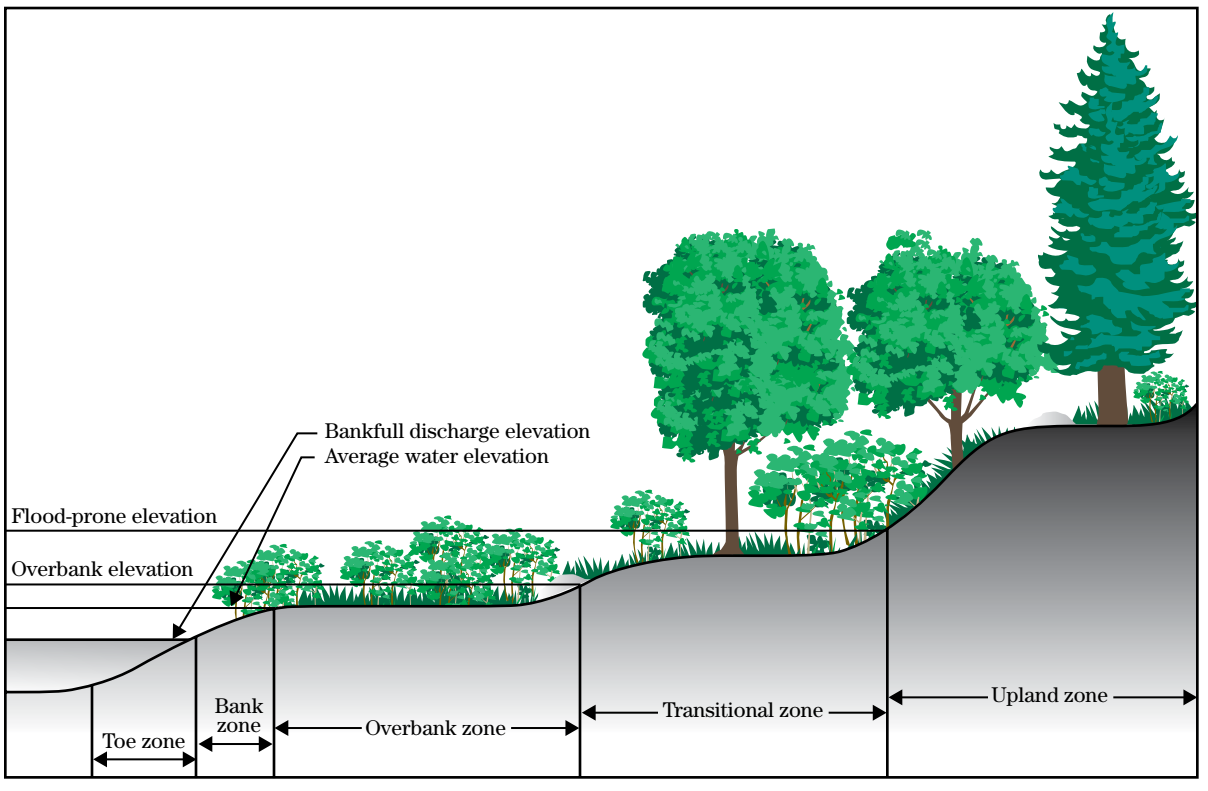
Bankfull flow is often considered to be synonymous with channel-forming discharge in stable channels and is used in some channel classification systems, as well as for an initial determination of main channel dimensions, plan, and profile. In many situations, the channel velocity begins to approach a 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

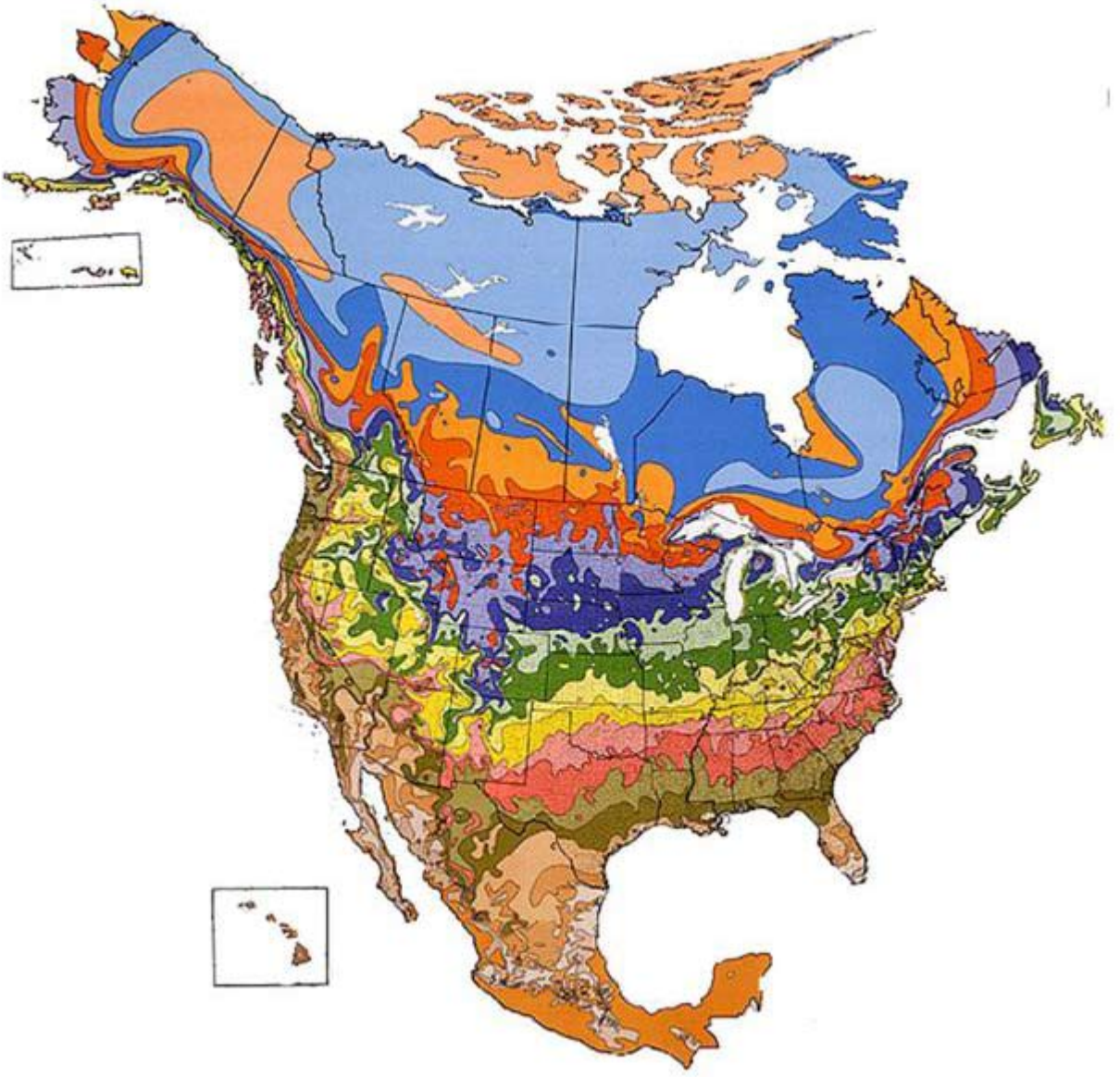


Figure TS14I–7 USDA plant hardiness zone map and key—Continued

USDA zone	Temperature range	Example cities

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.

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.

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

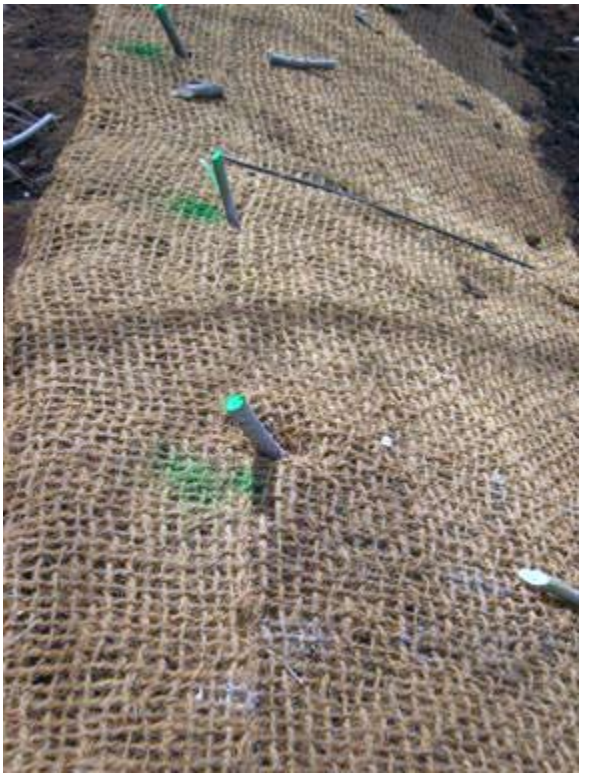


Figure TS14I–40 Combining fascines and fabric

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

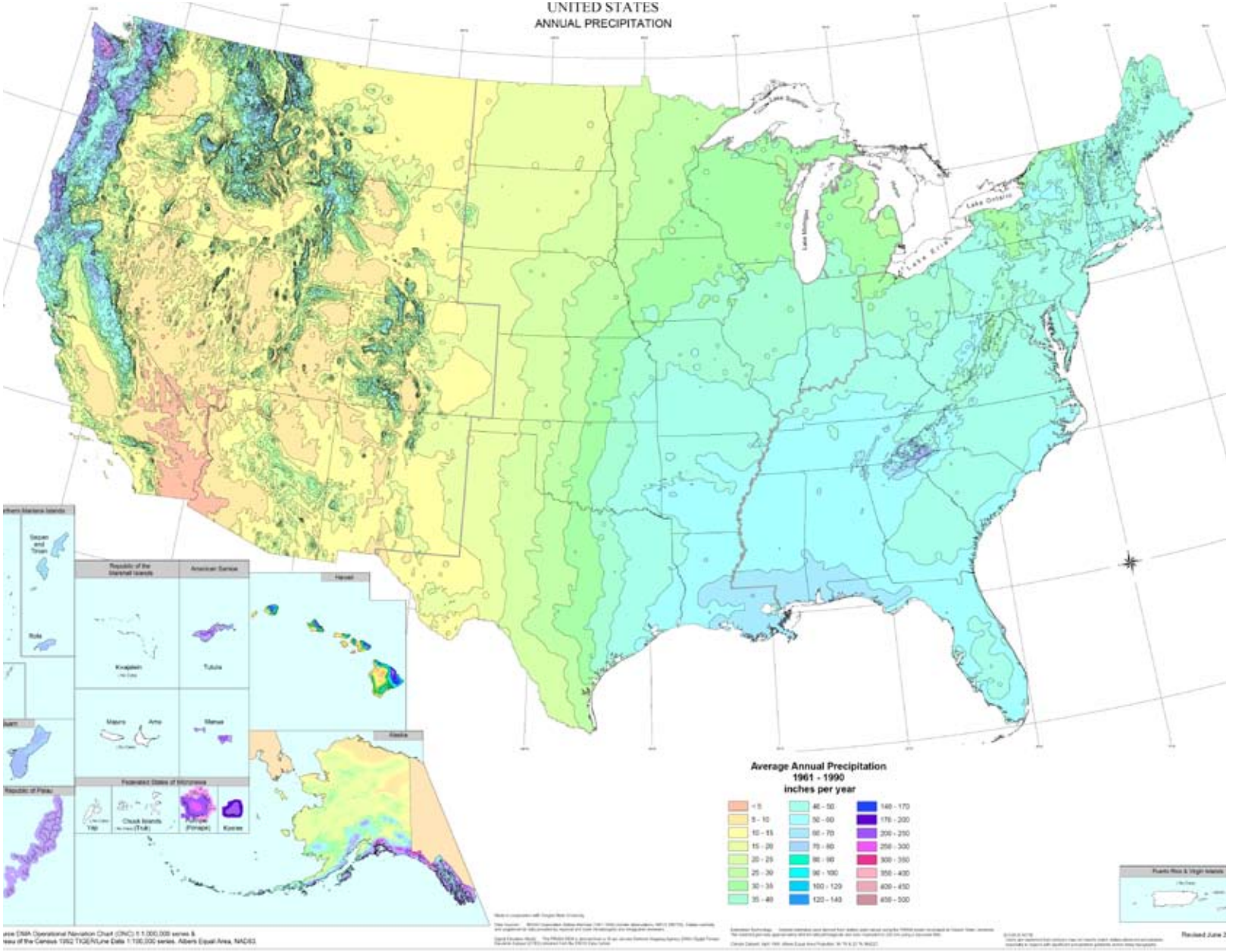
Cold climates place few constraints on the use of soil bioengineering treatments, which have been used widely in alpine areas and northern regions of Europe for hundreds of years. Many willows and other readily sprouting species are well adapted to cold conditions, and they perform excellently.

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.

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.

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.*)

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

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.

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

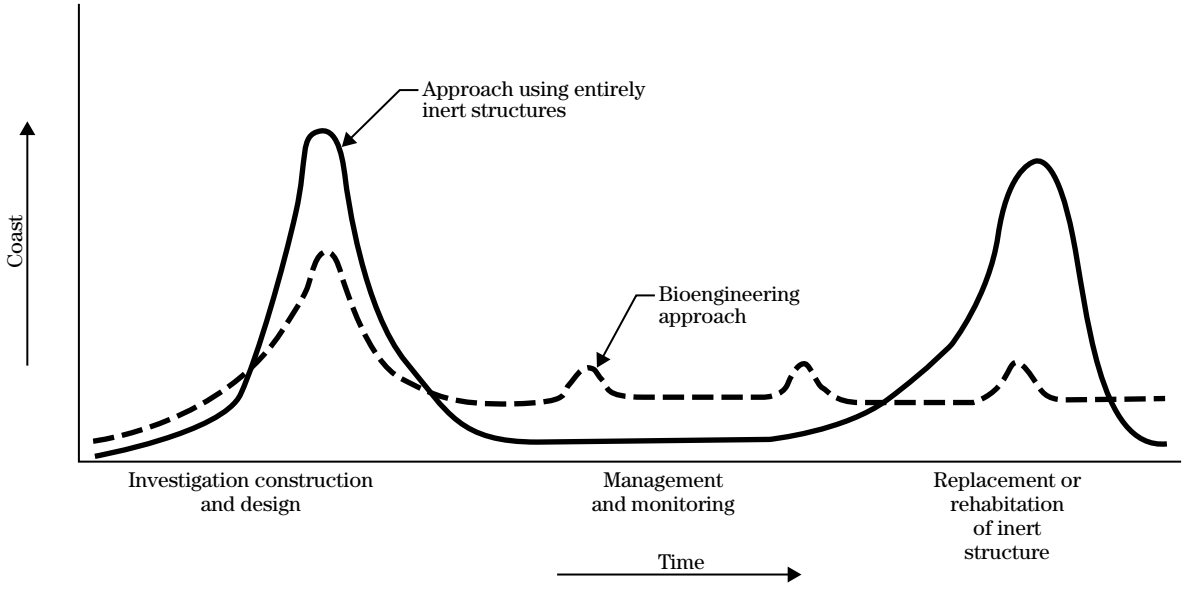
While soil bioengineering projects tend to be self-renewing and grow stronger with time, project areas require periodic monitoring and maintenance, particularly during the establishment stage. Maintenance is especially important on highly erosive sites. Maintenance could include removal of debris and 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.

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Issued August 2007

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.

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Figure TS14J–2 White River, IN, with large woody debris TS14J–4

Figure TS14J–3 Definition sketch for geotechnical forces on buried log TS14J–8

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.

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.

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 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.

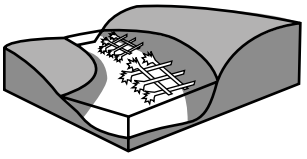
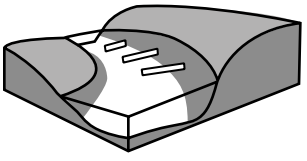
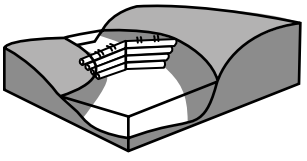
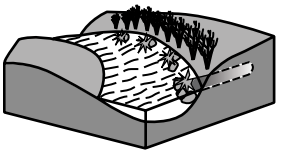
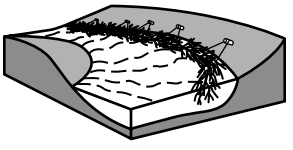
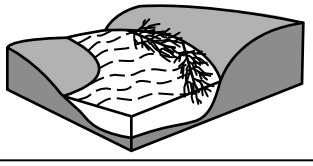
Existing designs for large wood structures may be grouped into a few basic configurations, as shown in table TS14J–3. Only general concepts are presented, as numerous variations are found. Combinations of woody materials with stone and living plant materials are common. The first three types shown in 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.

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

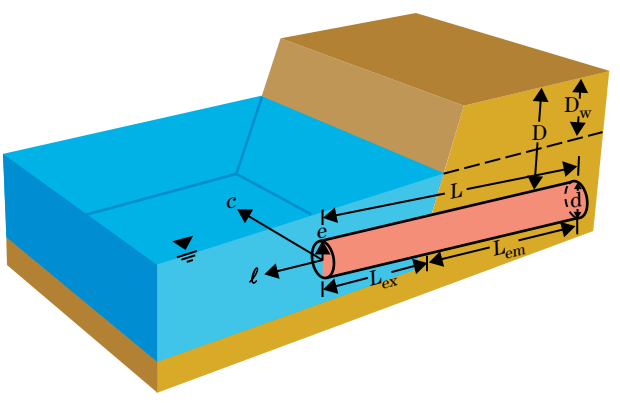
The lift coefficient on a single cylinder placed perpendicular to the flow is greatest (~0.45) when the cylinder is in contact with the bed and declines to near zero when the gap between the bottom of the cylinder and the bed exceeds one half times the cylinder diameter (Alonso 2004). As with drag, lift 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.

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.

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.

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.

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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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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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.

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.

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.

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

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.

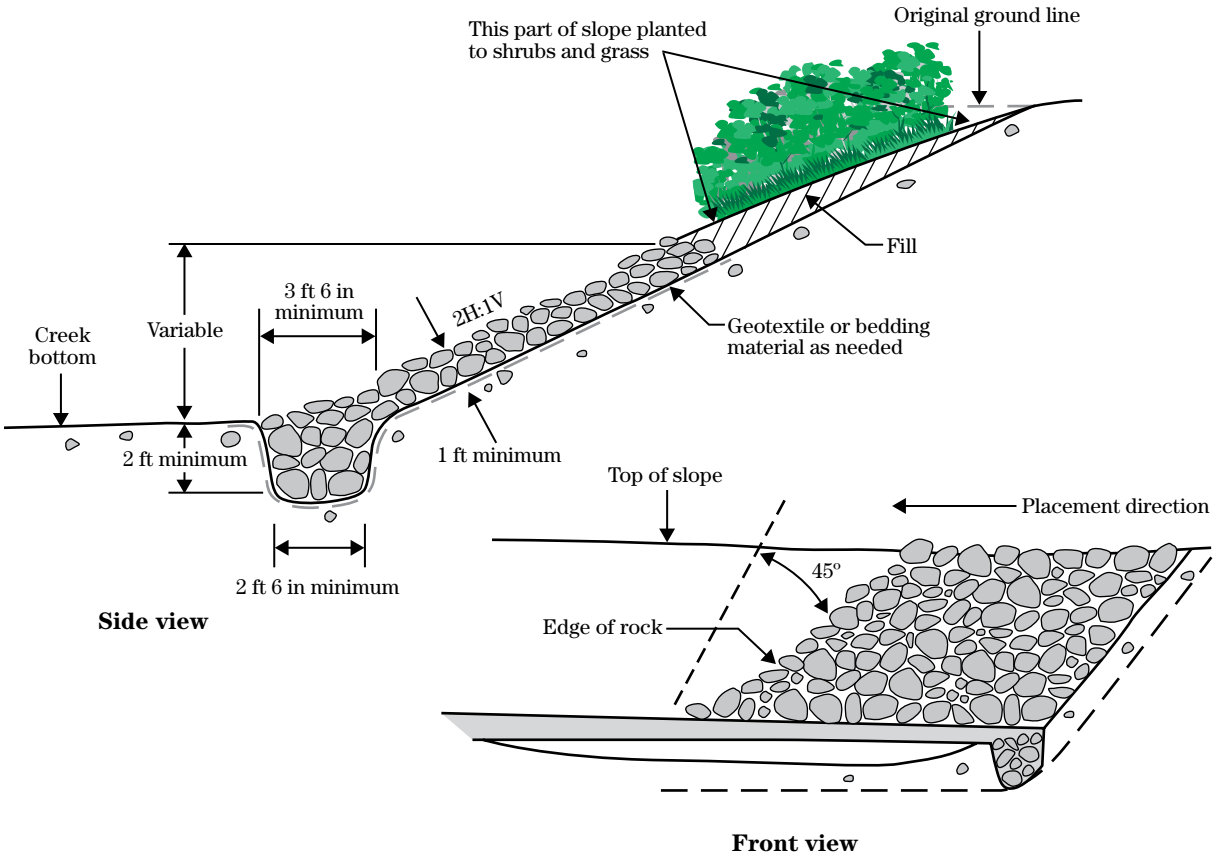
Riprap should be placed to provide a reasonably well-graded and dense mass of rock with a minimum of voids and with the final surface meeting the specified lines and grades. The larger stones should be placed in the toe trench or well distributed in the revetment. The finished stone protection should 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

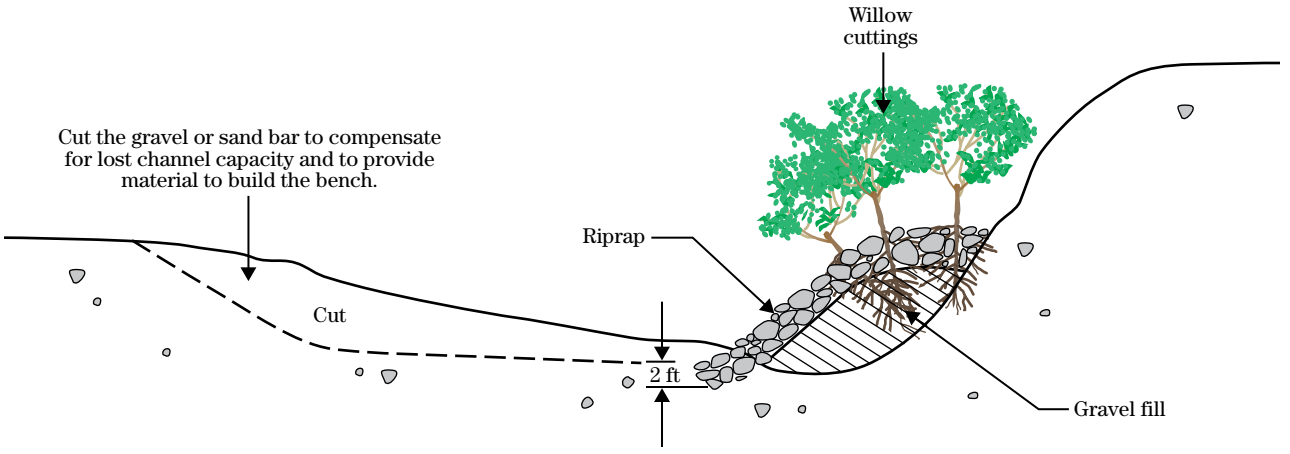
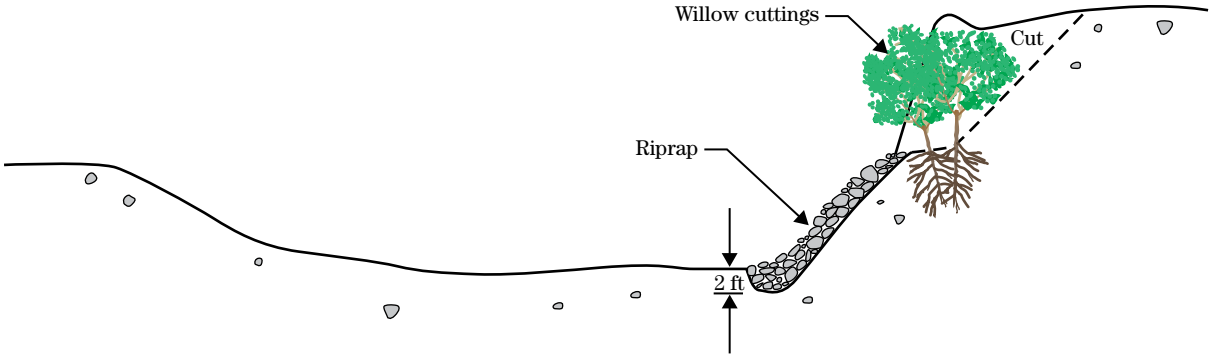


Figure TS14K–4 Excavated bench method



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.

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

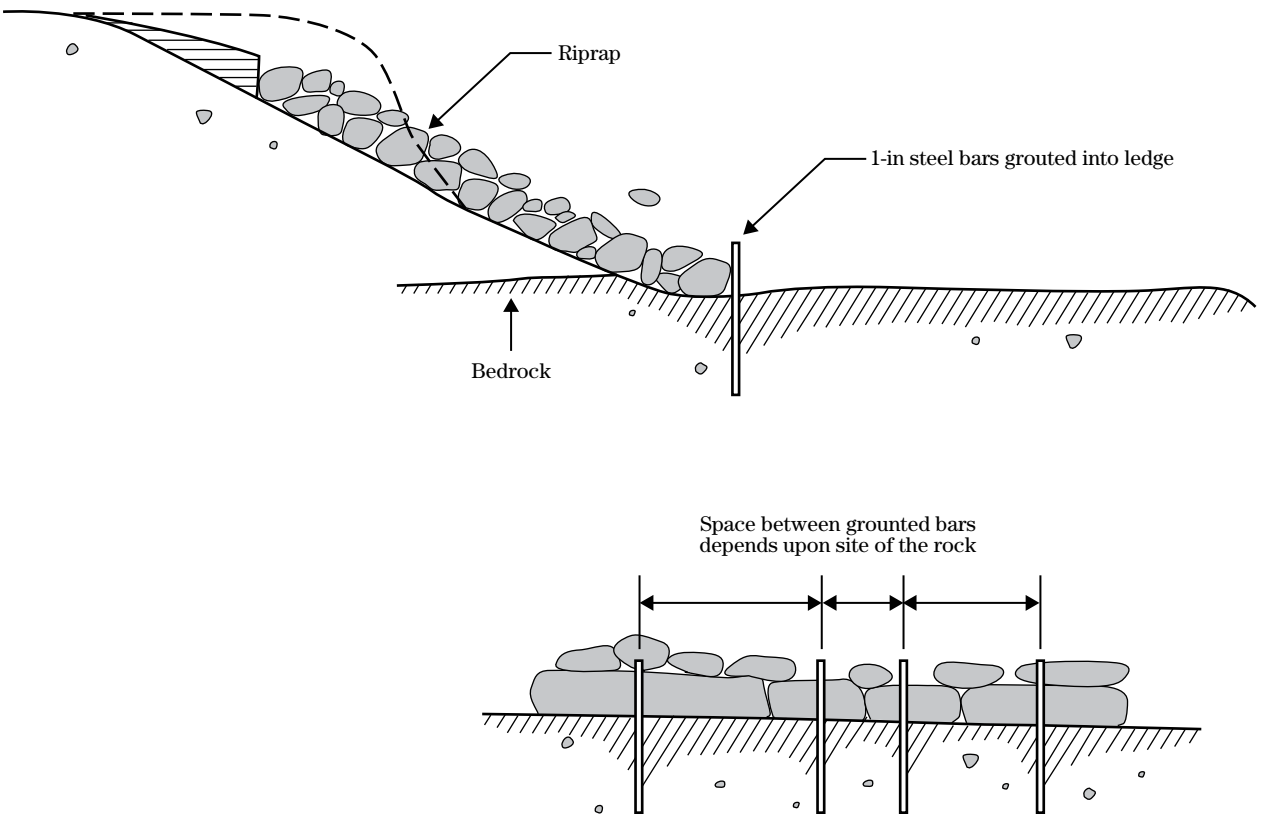
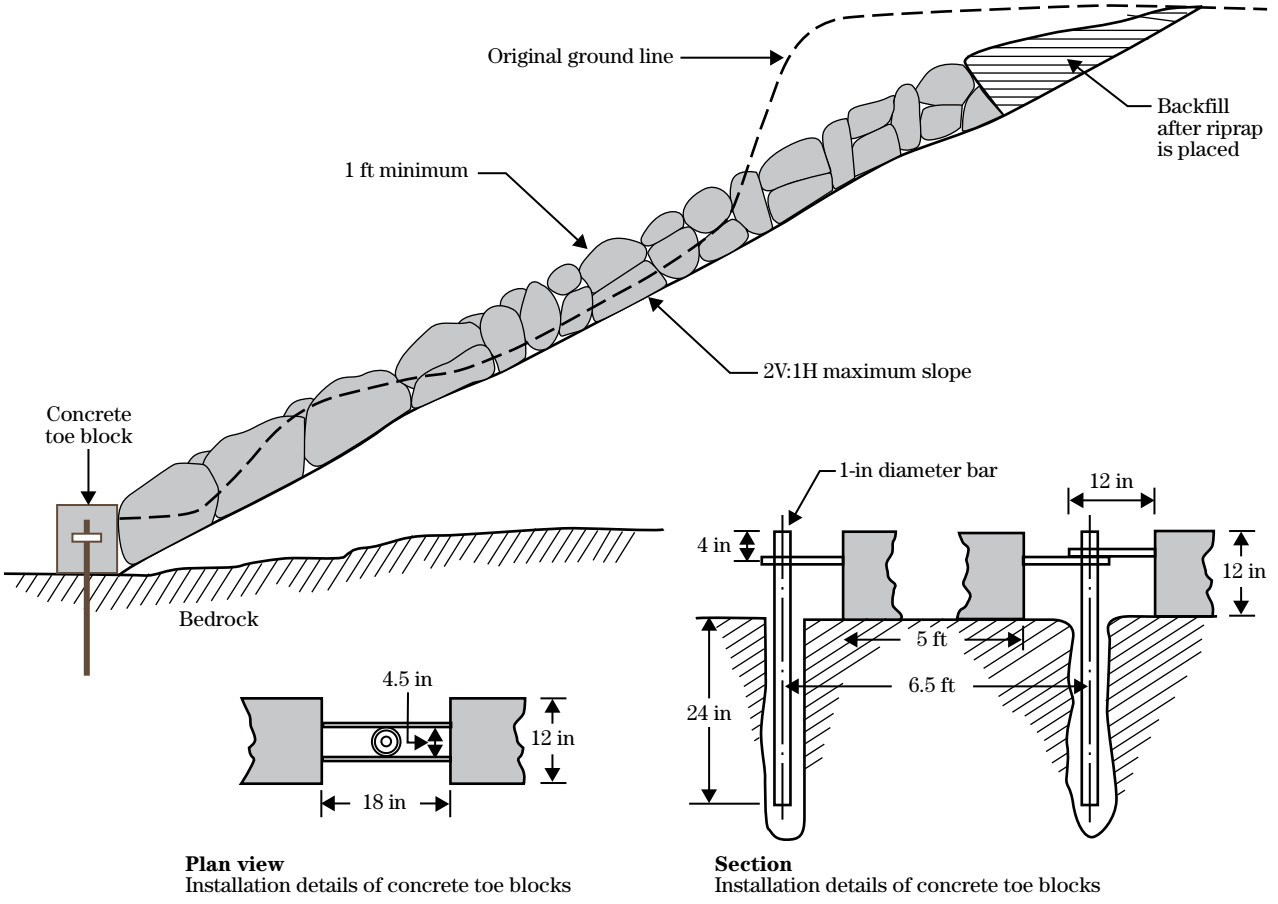


Figure TS14K–6 Precast toe block method



Plan view
Installation details of concrete toe blocks

Section
Installation details of concrete toe blocks

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.

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 (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

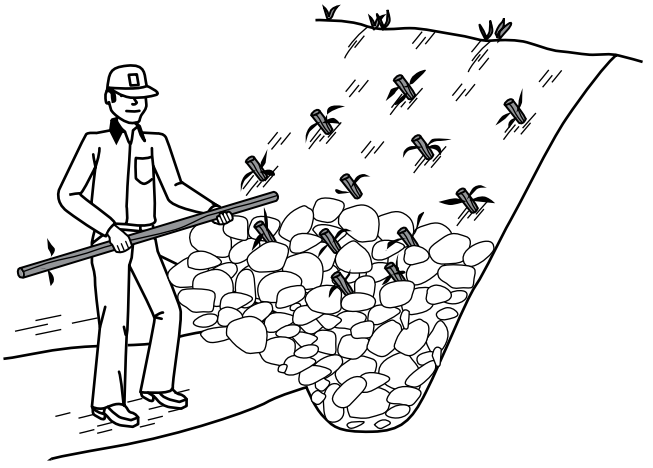


Figure TS14K–8 Brush layer over stone toe

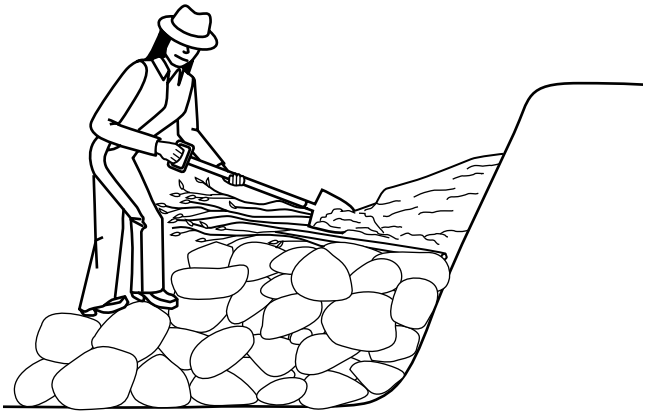


Figure TS14K–9 Vertical bundle and stone toe

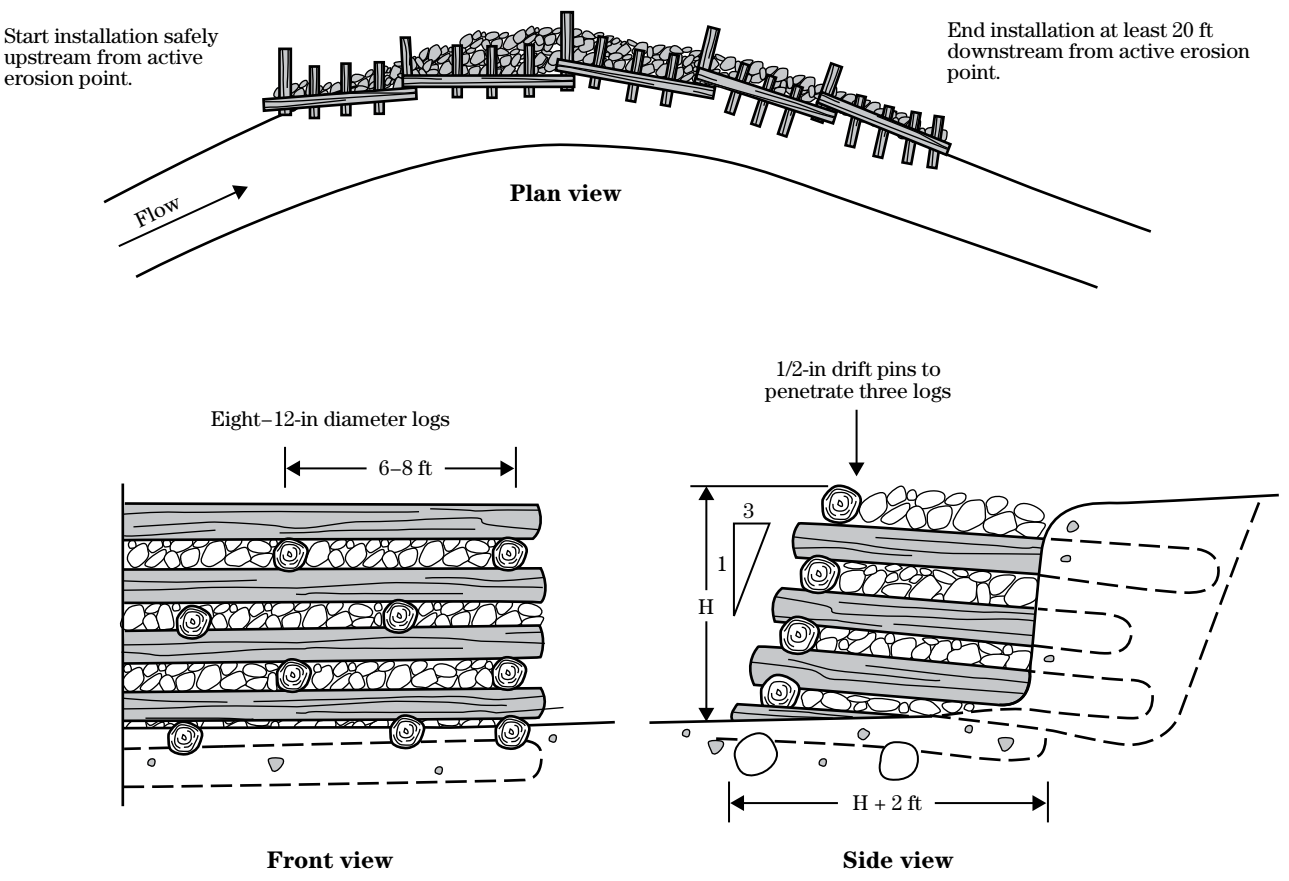
On moderate-sized tributaries along the Mississippi River, typical applications can be 1 to 2 tons per linear foot, resulting in a triangular peak between 3 and 5 feet above the streambed (Biedenharn, Elliott, and Watson 1997). Usually, this simple technique is constructed by dumping stone from the bank. 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 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.

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 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.

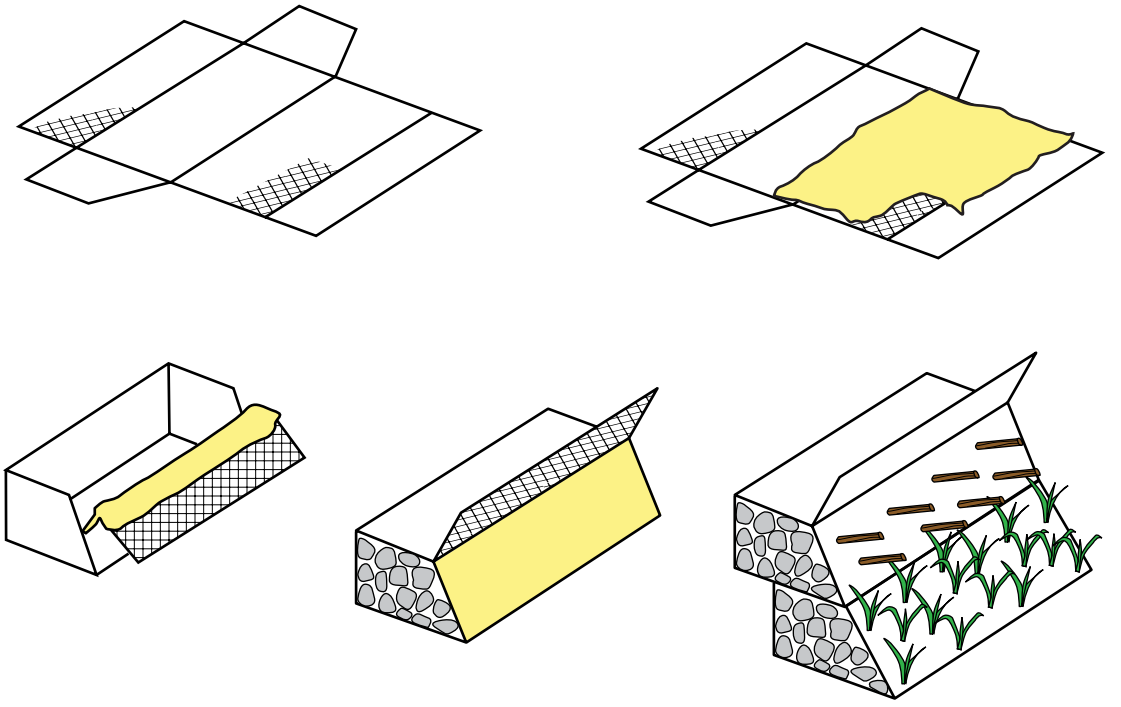
While the stone used for a grouted riprap installation can be smaller than what is required for a loose riprap installation, there is no available guidance that specifies a minimum size. Sizing is usually based on experience with similar projects in the area. The stone used should be as coarse as possible 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.

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.

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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.

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.

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

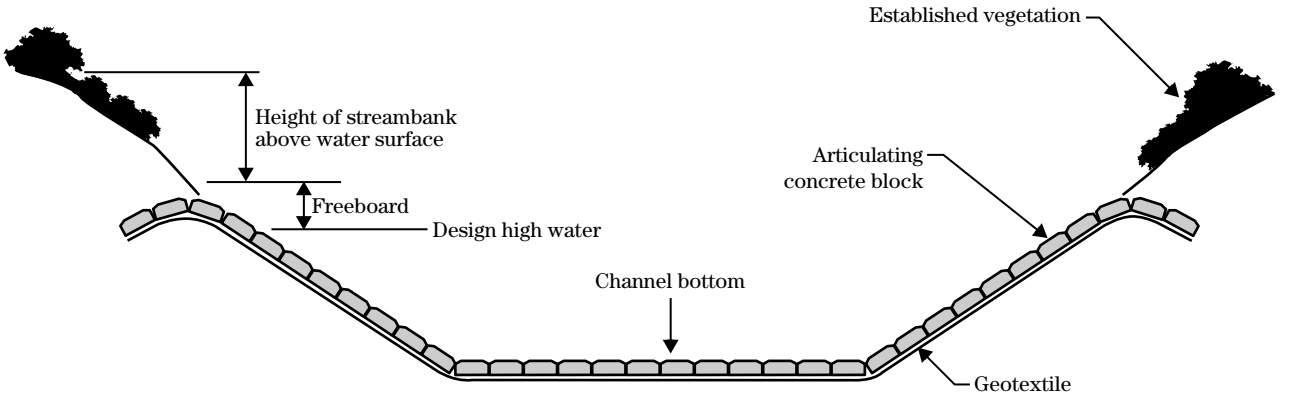
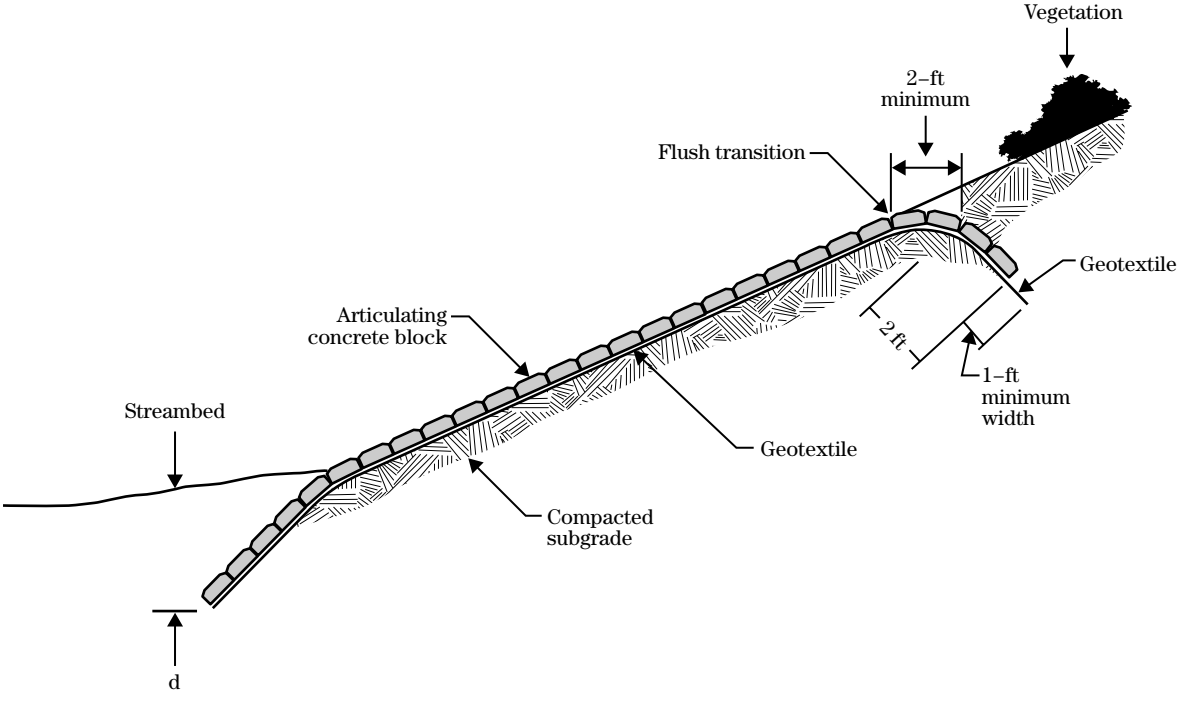
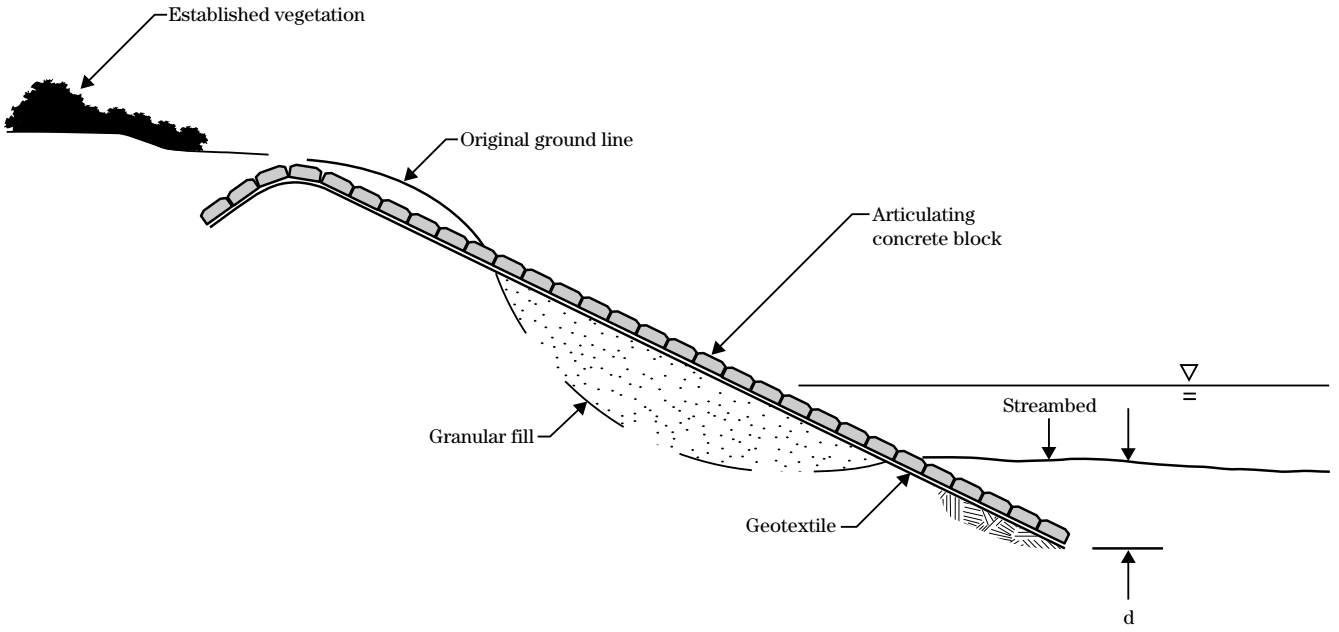


Figure TS14L–2 Armoring the toe and lower slope cross section



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

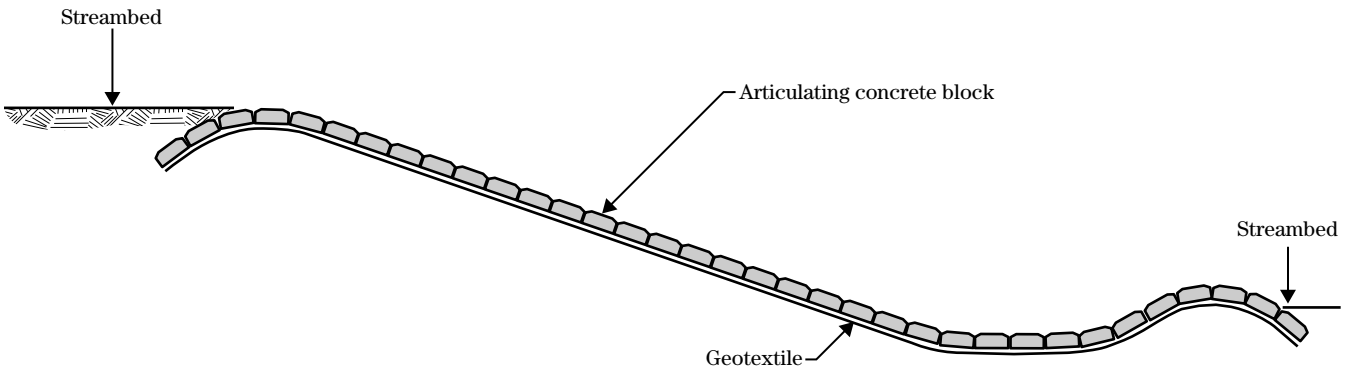


Figure TS14L–5 Armoring of pipe/culvert outlets profile

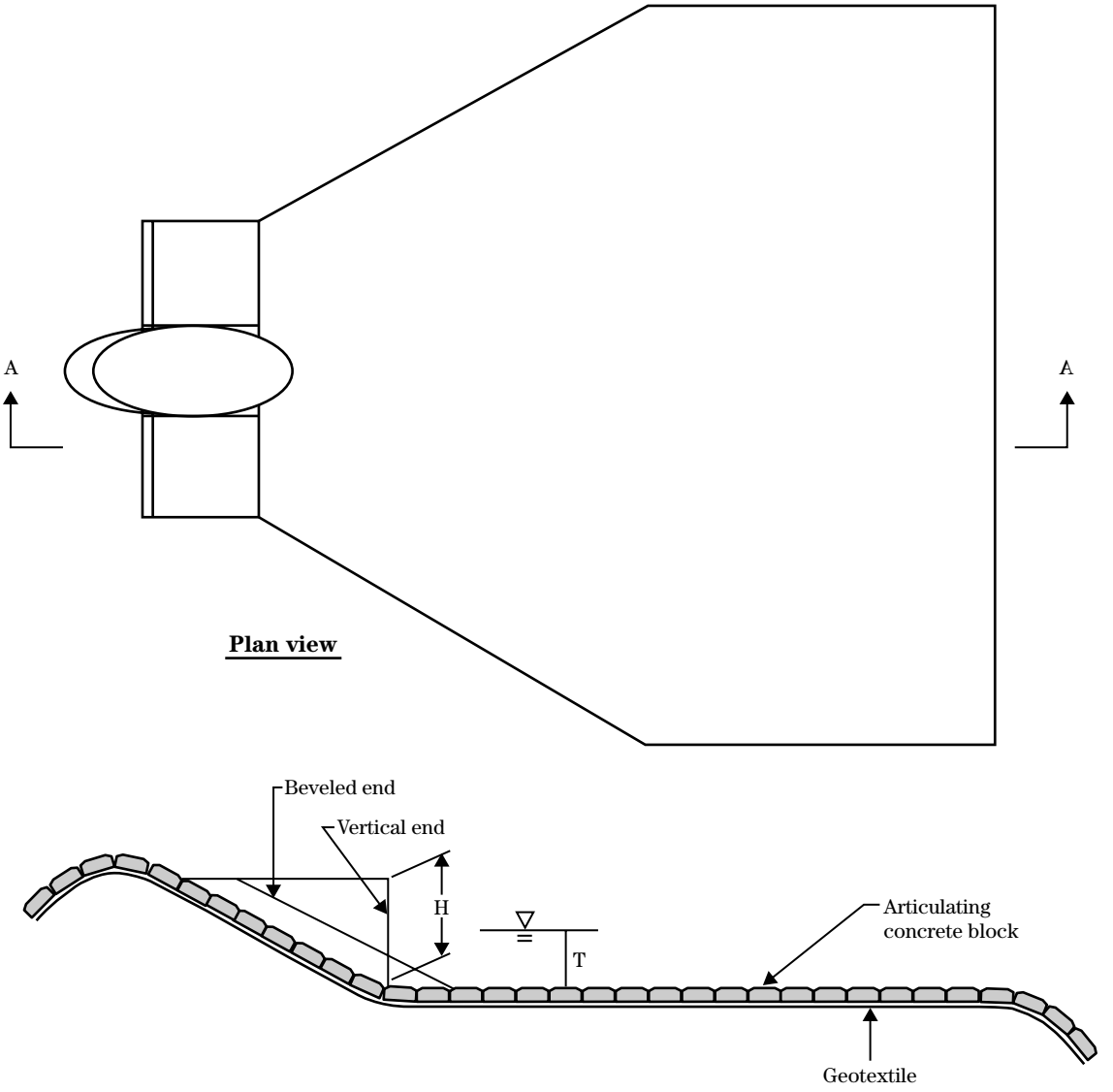
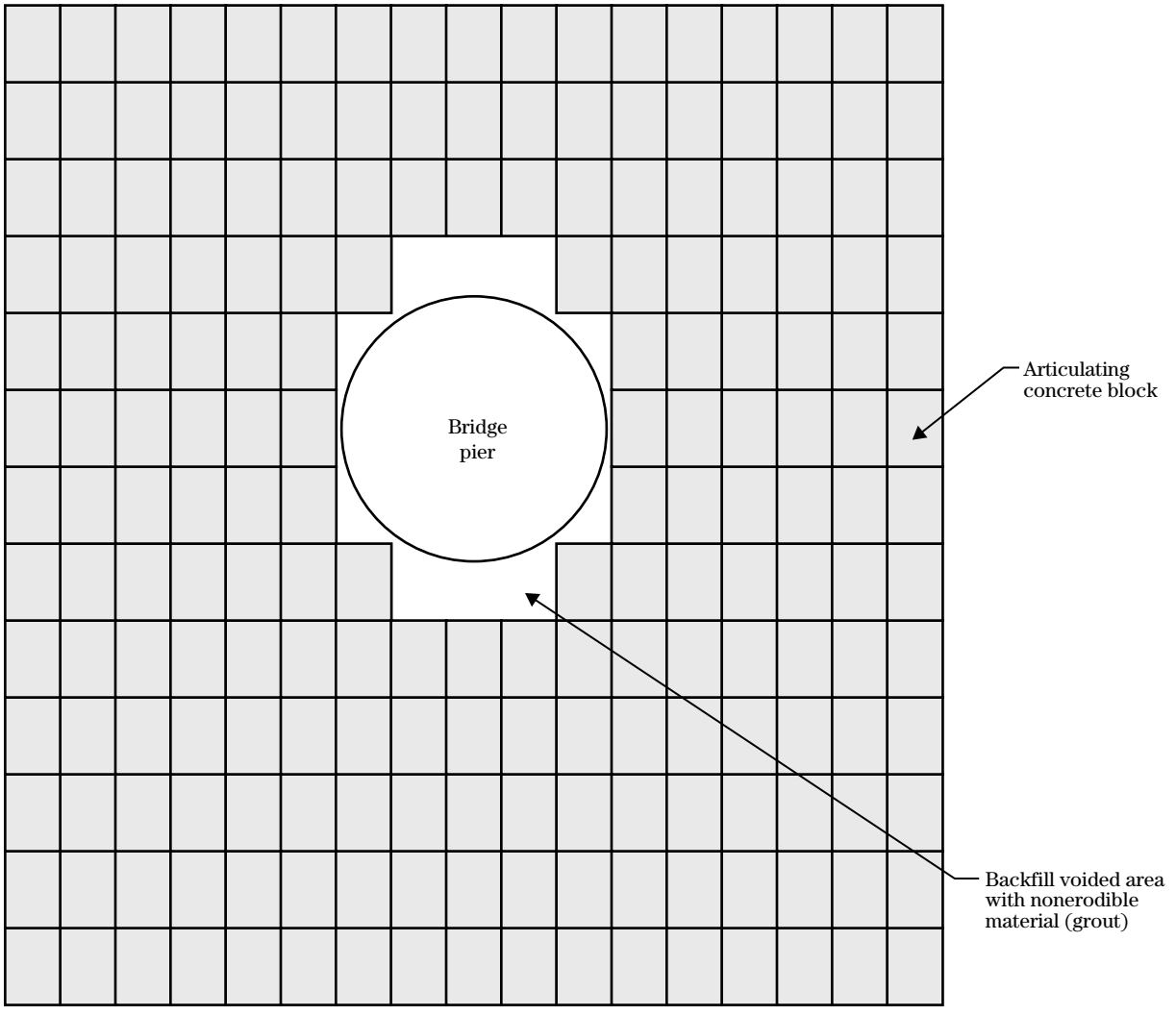


Figure TS14L–6 Scour protection around bridge pier plan



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

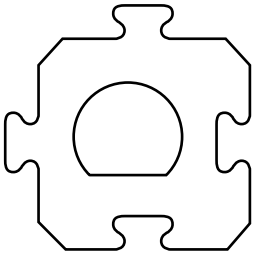
Geotextiles are typically used to retain the soil particles serving as the subgrade for the ACBs (fig. TS14L–10). Geotextiles may be woven or nonwoven and may be composed of multifilament yarns or monofilament yarns. Woven slit film (monofilament or multifilament) geotextiles should not be used as a 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

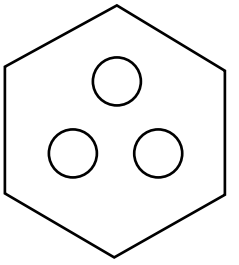
The purpose of the granular filter is to intercept water flowing through the pores of the subgrade soil, allowing for the passage of the water, while retaining the subgrade soil particles. Granular filters consist of sand, gravel, or a sand and gravel mixture and may contain some fine-grained particles.

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.

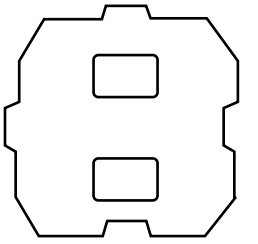
Figure TS14L–7 Examples of ACB revetment systems (*Figures courtesy of HCFCD*)



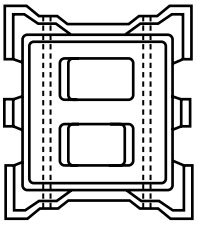
Channellock™



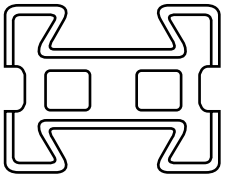
Hex lock™



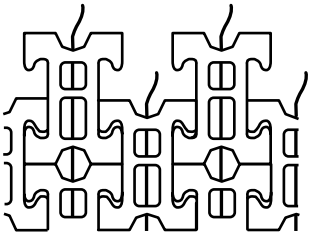
Petraflex™



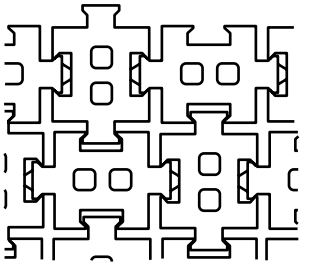
Armorflex™



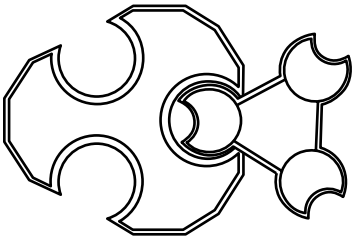
Armorloc™



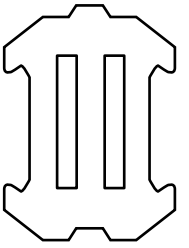
Conlock I™



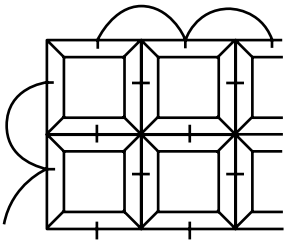
Conlock II™



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*)

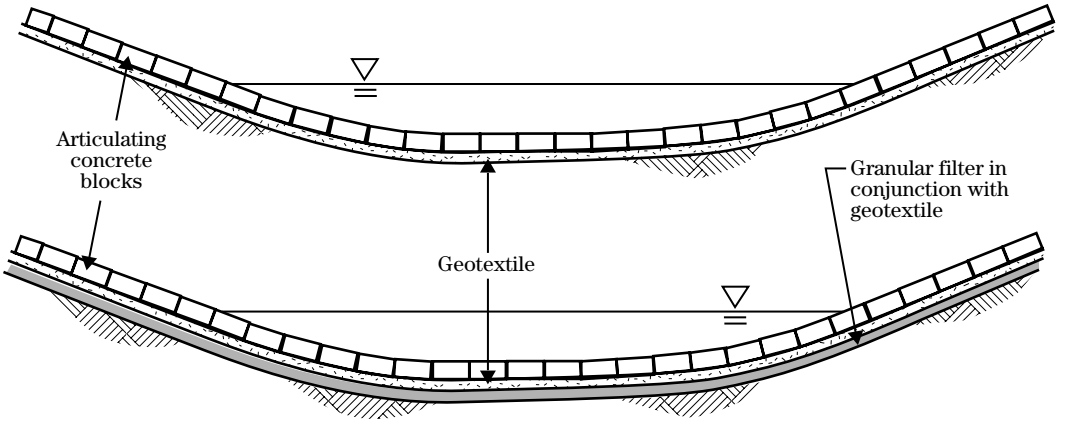


Figure TS14L–11 ACBs in combination with granular and geotextile filter

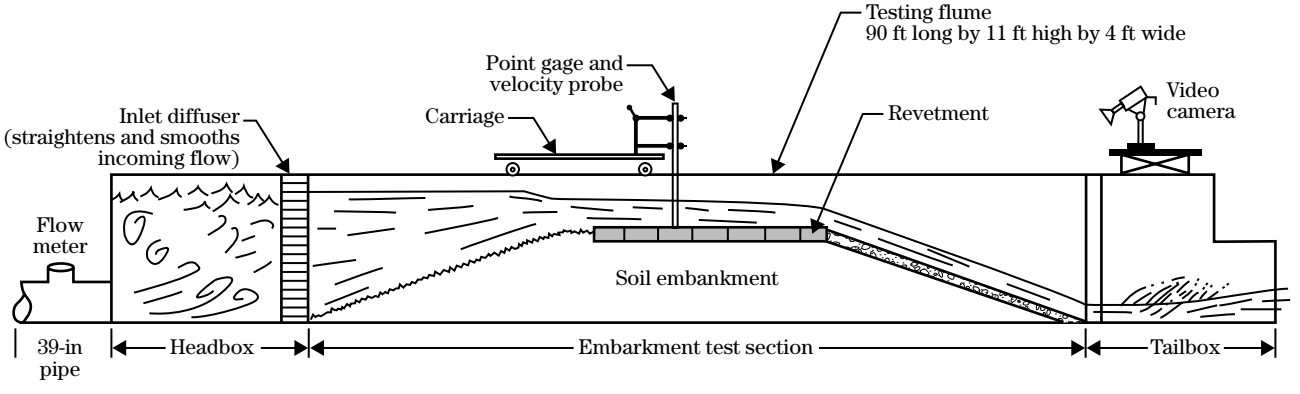
Due to the proprietary nature and unique characteristics of the ACB revetment systems available, a hydraulic stability test should be completed on each family of blocks. The hydraulic stability test should be conducted in accordance with U.S. Department of Transportation Federal Highway Administration (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*)

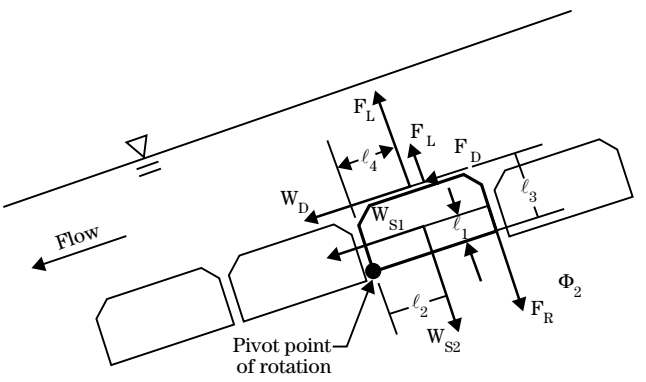
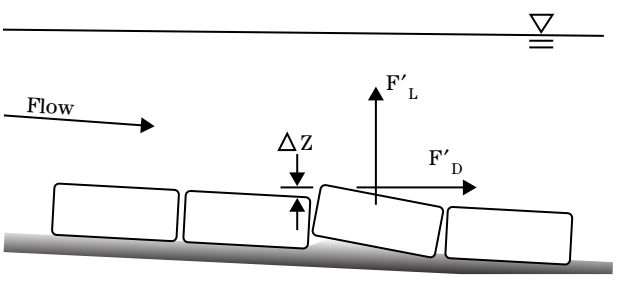


Figure TS14L–14 Forces on a protruding block (*Figure courtesy of HCFCD*)



$$F'_D = \frac{1}{2} \times C_D (\Delta Z) b \rho V^2$$

(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.

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.

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).

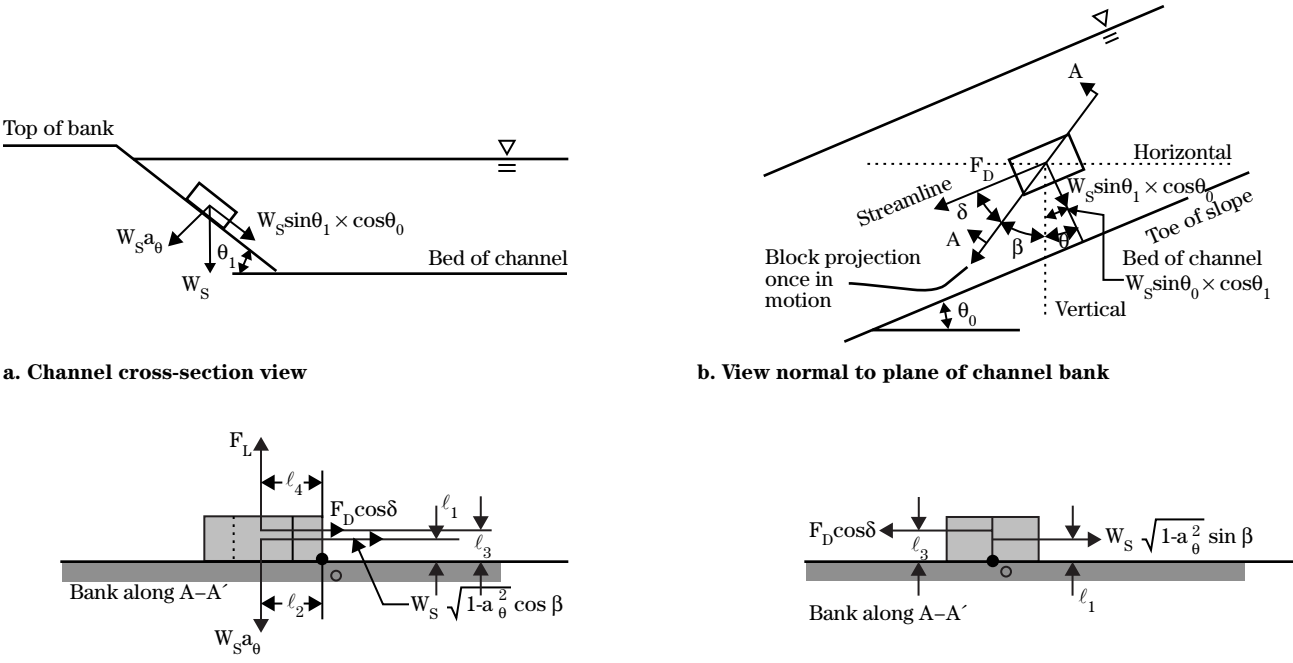
Failure (loss of intimate contact) is typically the result of the overturning of a block or group of blocks about the downstream contact point of the block. The hydraulic stability of a block on a channel side slope is a function of the magnitude and direction of stream velocity and shear stress, the 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).

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*)



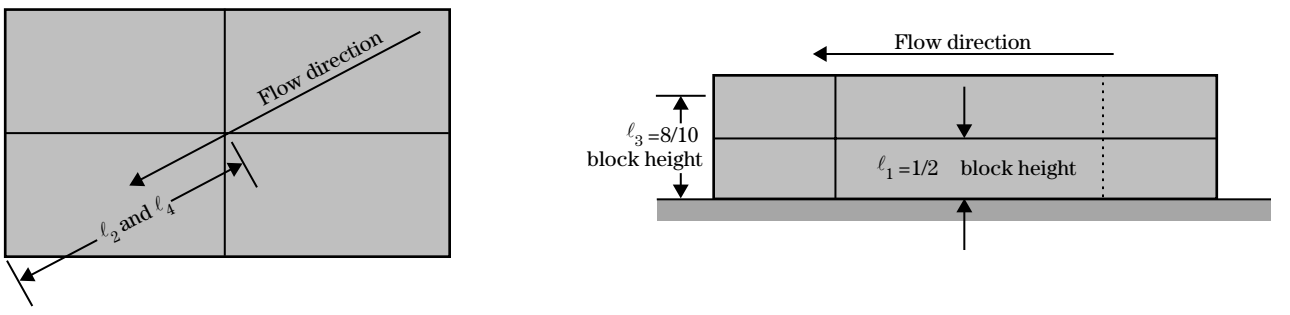
a. Channel cross-section view

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*)

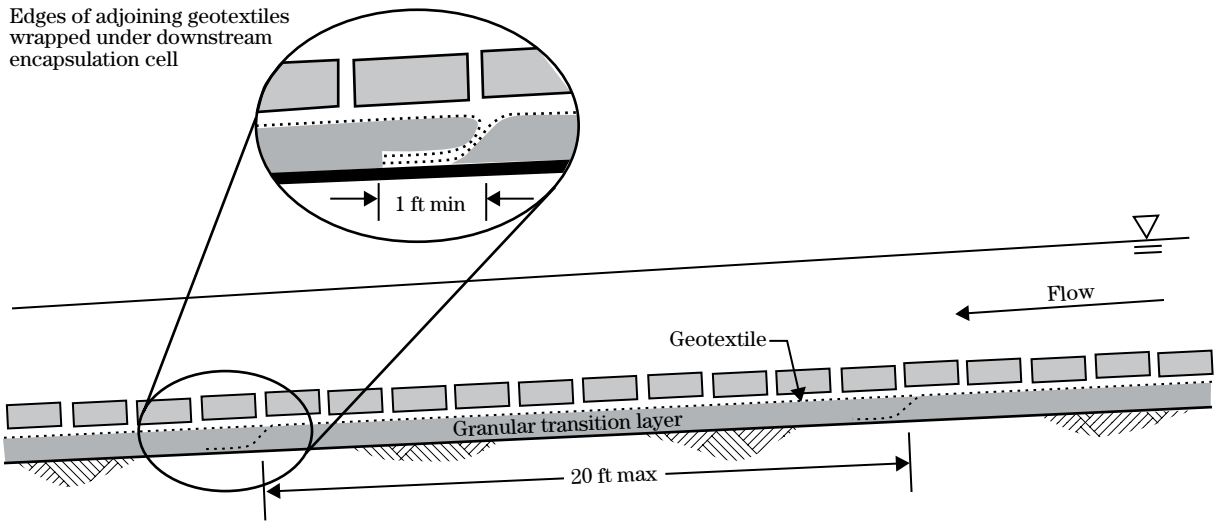


Figure TS14L–18 Spreader bar for placement of cabled mats

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*)

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.

$$\delta + \beta + \theta = 90^\circ \qquad \text{(eq. TS14L–17)}$$

$$\delta = 90^\circ - \beta - \theta$$

So, $= 90^\circ - 17.82^\circ - 3.43^\circ$ (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.

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.

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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.

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

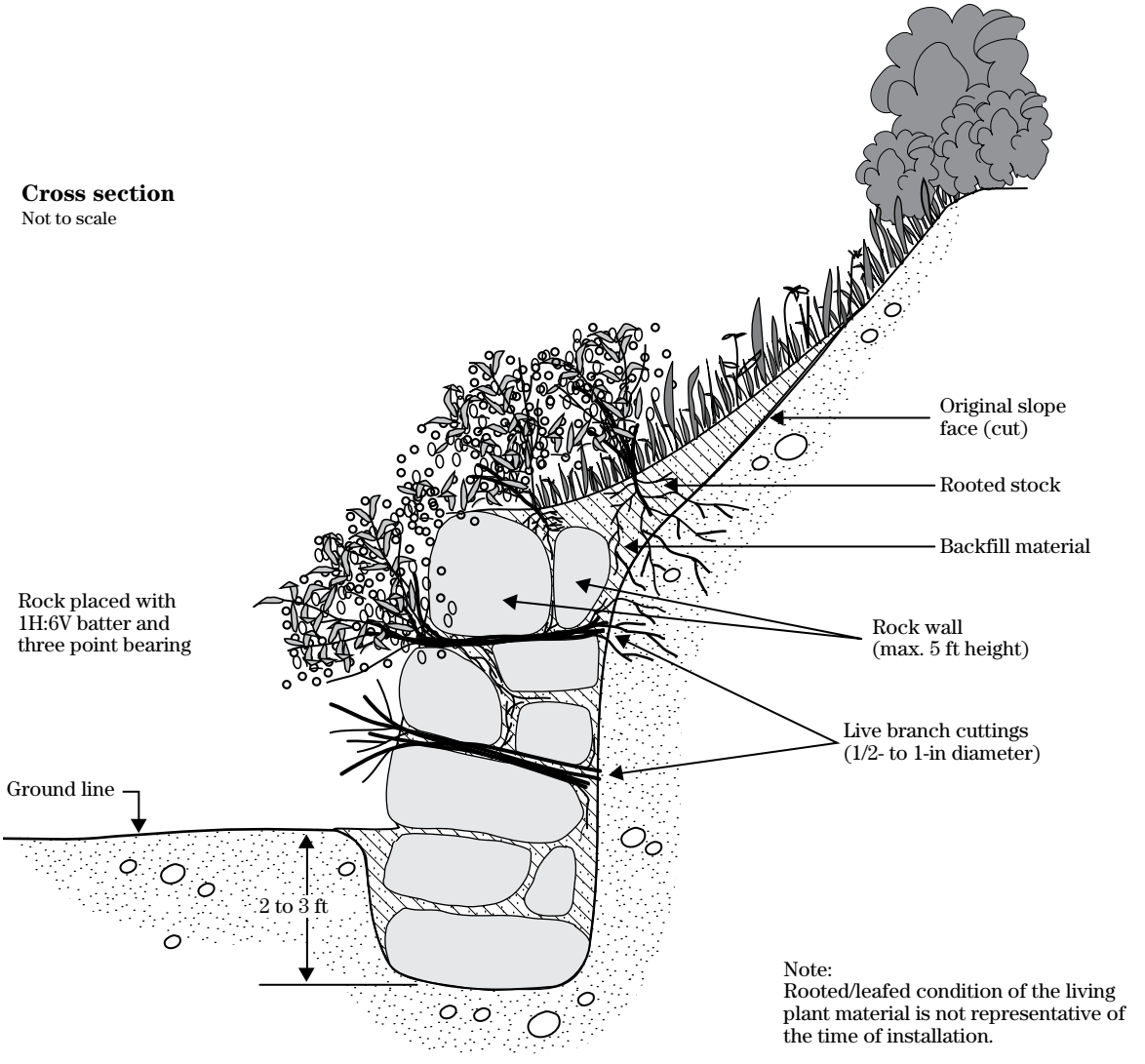
A sloping bench can be provided behind the wall to serve as a transition slope on which vegetation can be planted. Well tamped backfill should be placed behind the wall and in the spaces between the rocks as they are placed. Live branch cuttings can be placed in the interstices of the rock wall as it 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.

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.

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.

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

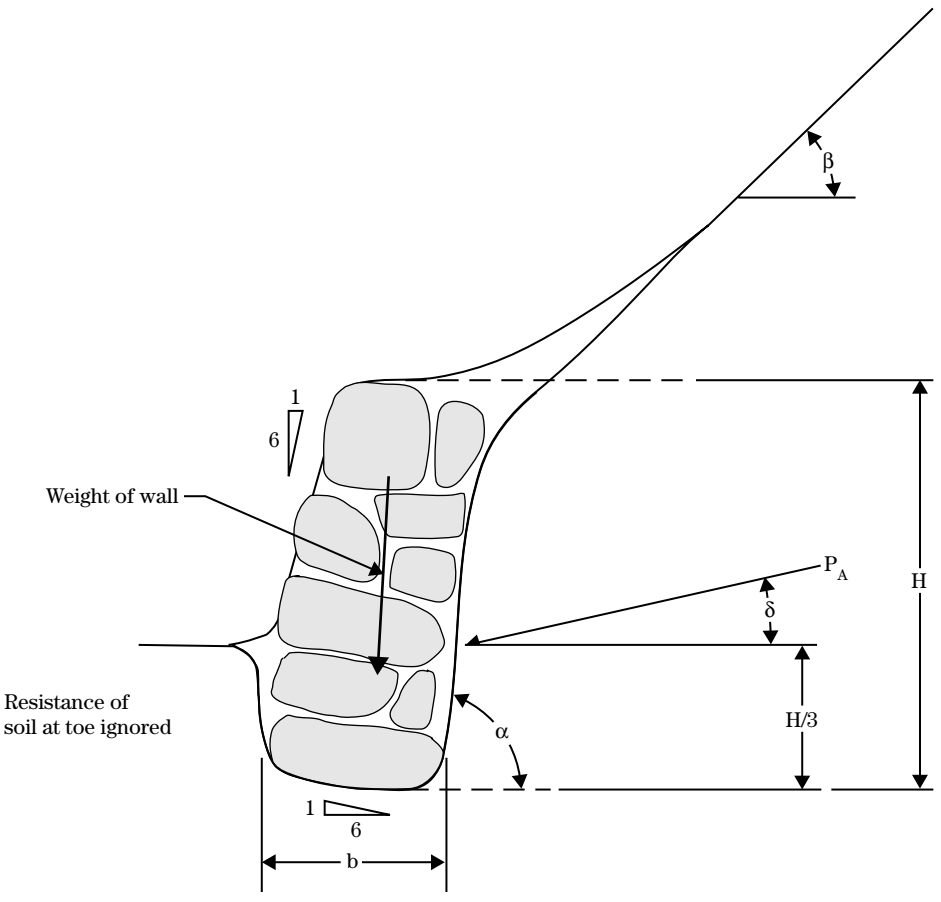


slope (increasing the factor of safety for slope stability). A visual reconnaissance should be followed up by a mechanical analysis of the slope to address its global stability related to slope failure.

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.

$$\frac{\Sigma F_v \tan\phi}{\Sigma F_H} \ge 1.5 \qquad \text{(TS14M–5)}$$

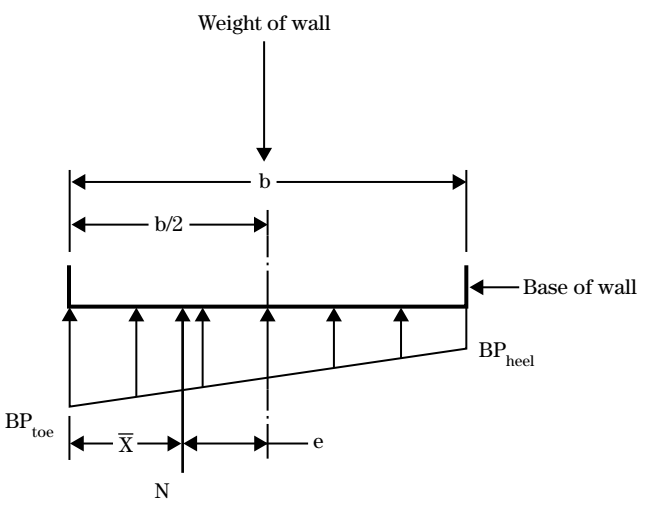
where:

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.

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.

Issued August 2007

Cover photo: Restoring fish migration may require culverts and other water conveyances to be redesigned or replaced.

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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.

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.

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 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.

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:

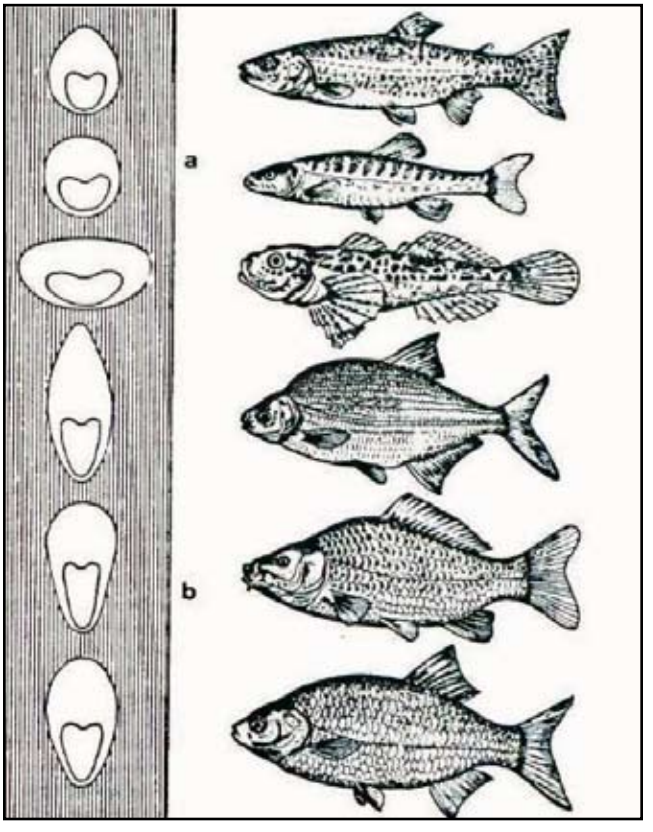
Burst (darting) speed—highest swimming speeds; endurance less than 20 seconds; ends in extreme fatigue

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

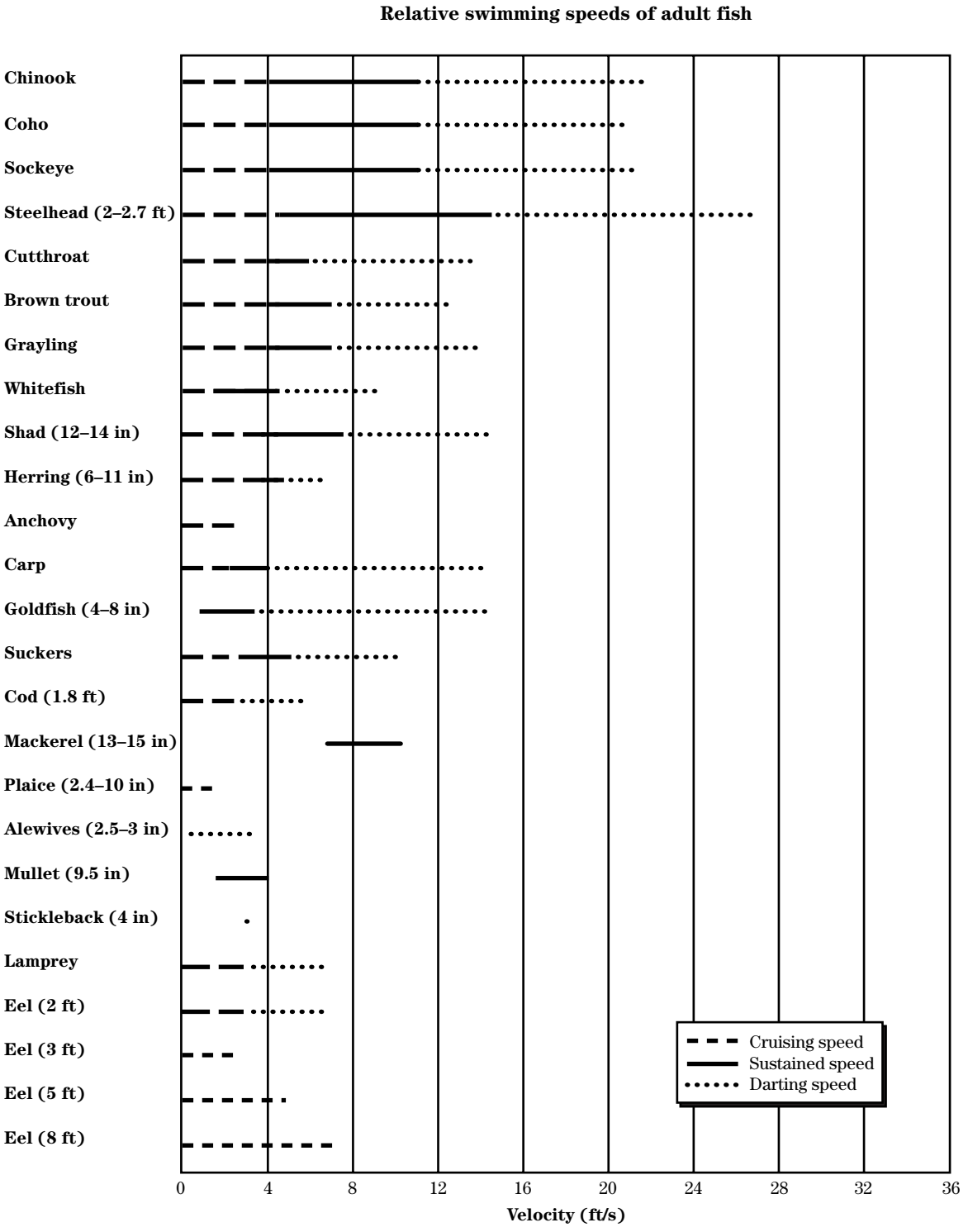
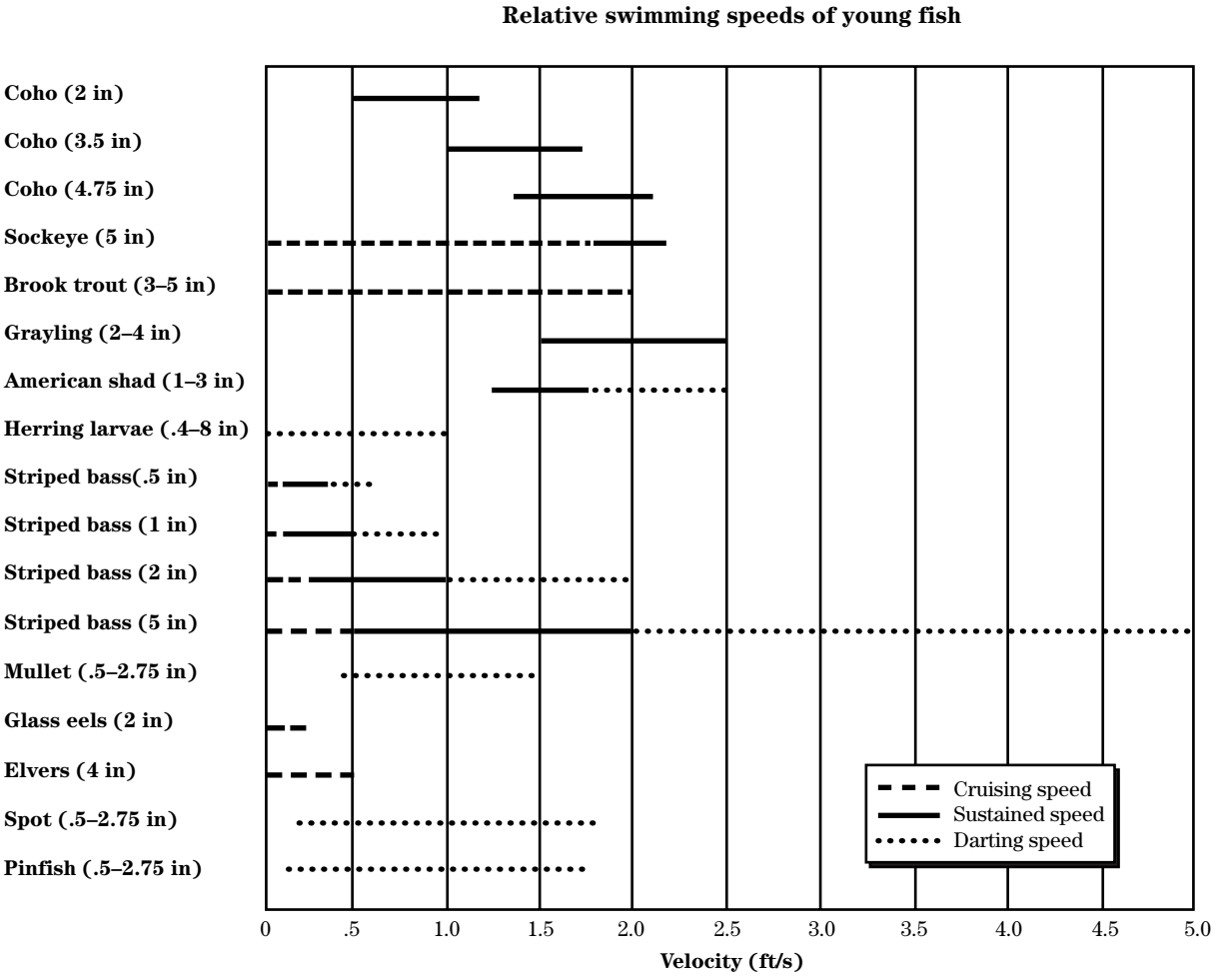


Table TS14N–5 Relative swimming speeds of young fish



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.

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.

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.

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 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:

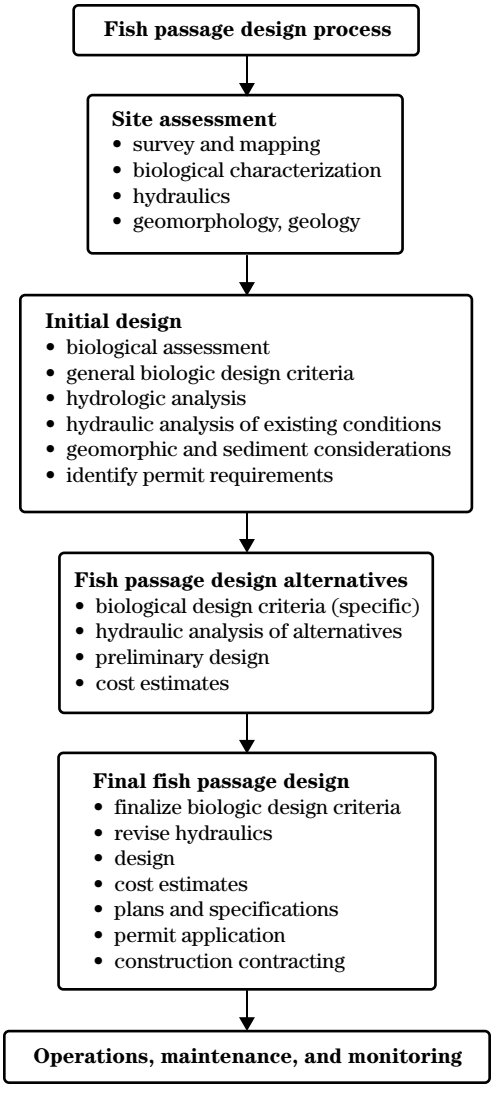
http://wdfw.wa.gov/hab/engineer/habeng.htm#dwnstrm

http://swr.nmfs.noaa.gov/hcd/fishscrn.htm

http://swr.nmfs.noaa.gov/hcd/pumpcrit.pdf

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 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.

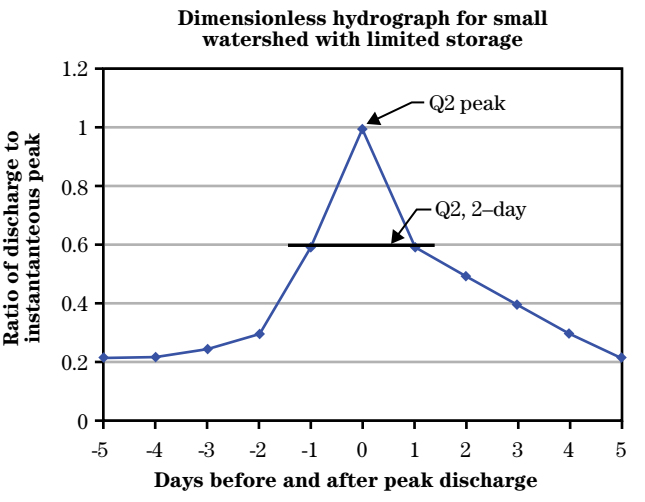
The first step in the engineering design is the hydrologic analysis. Typically, designs require defining the range of high and low discharges the fish passage facility will operate within. Hydrologic analysis must consider the period of interest when migration occurs and when statistical analyses for 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 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.

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 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:

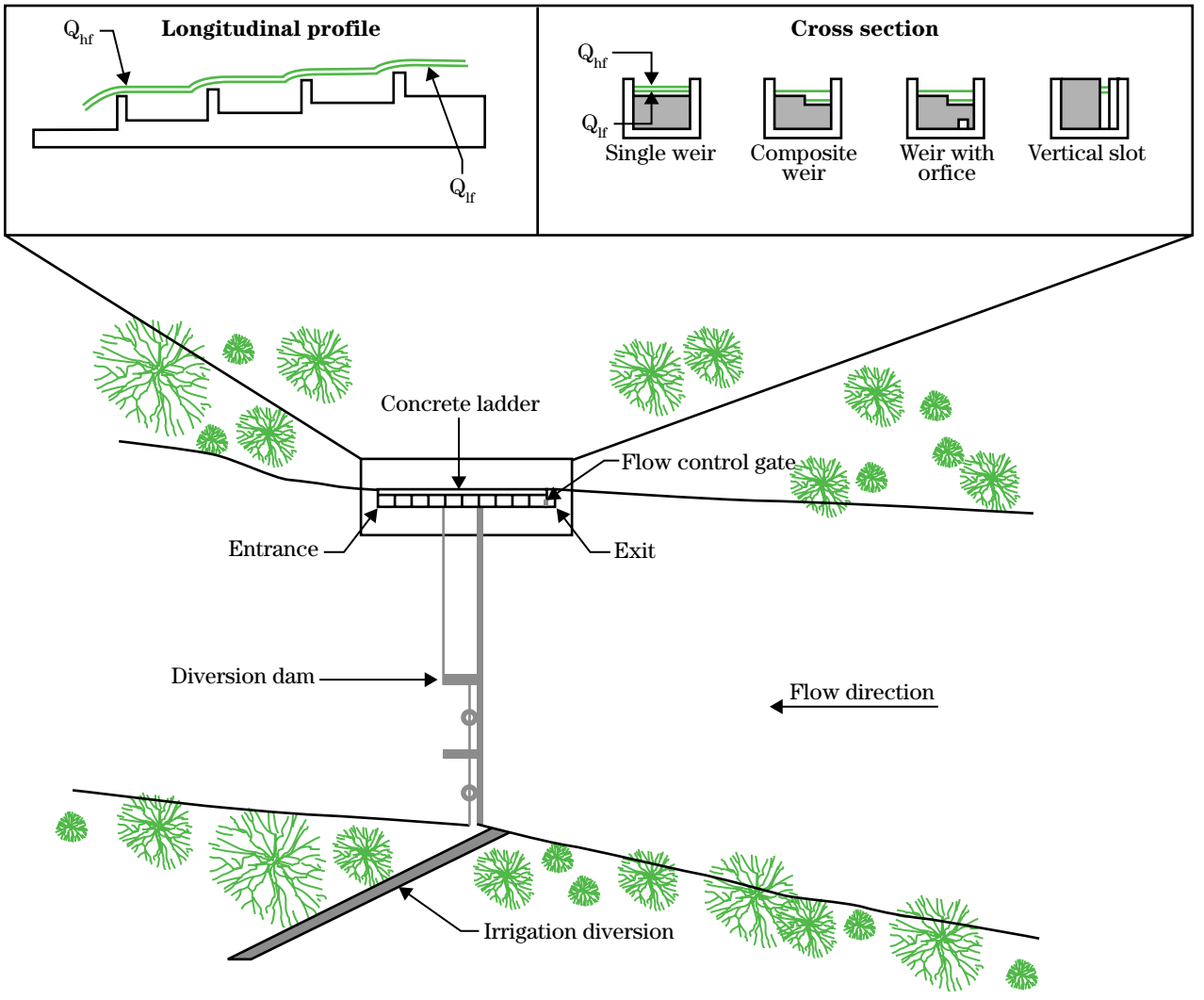
- excavated, earthen channels artificially roughened with large rocks
- seminatural channels equipped with stair-stepped resting pools held in place with rocks, logs, or stoplogs
- concrete and/or metal structures that slow water velocity enough to provide upstream passage

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 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)

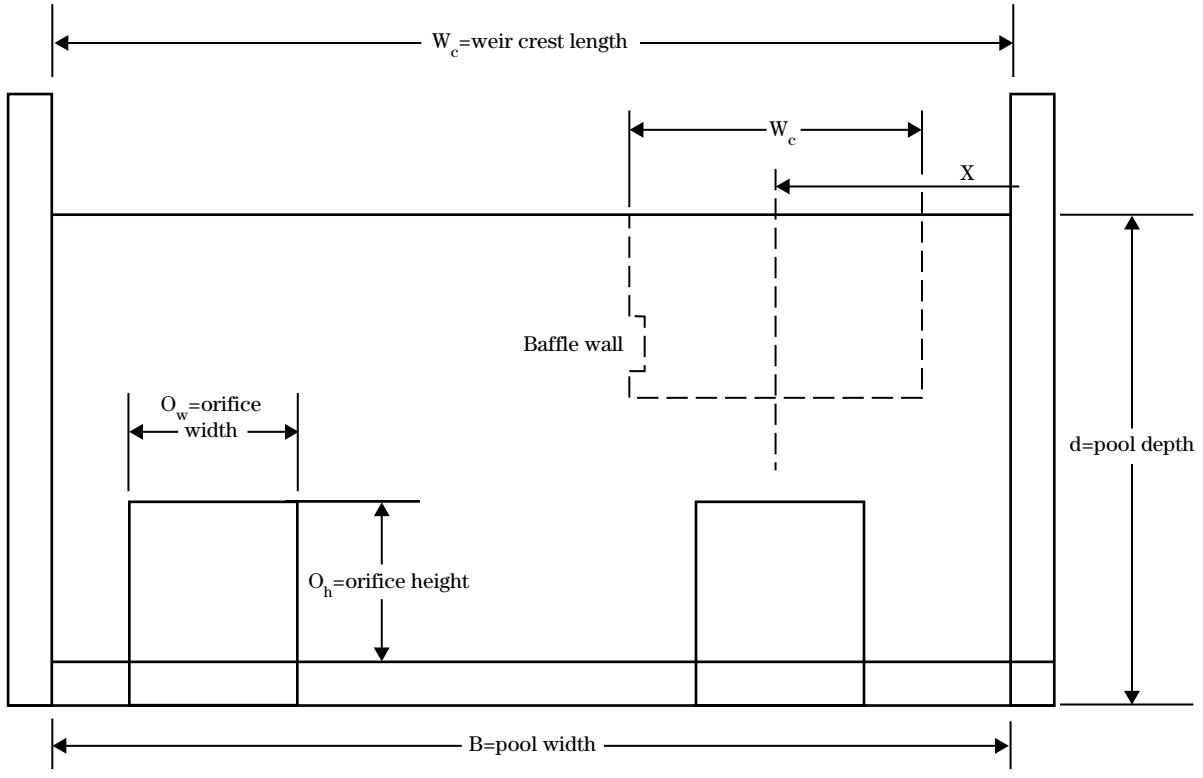
Equation TS14N–2 is the general form of a weir equation and can be used to estimate discharge, given water surface elevation and weir height, or back-calculate water surface elevations by rearranging the equation to solve for head on the weir (fig. TS14N–7).

$$Q = C_d L H^{1.5} \qquad \text{(eq. TS14N–2)}$$

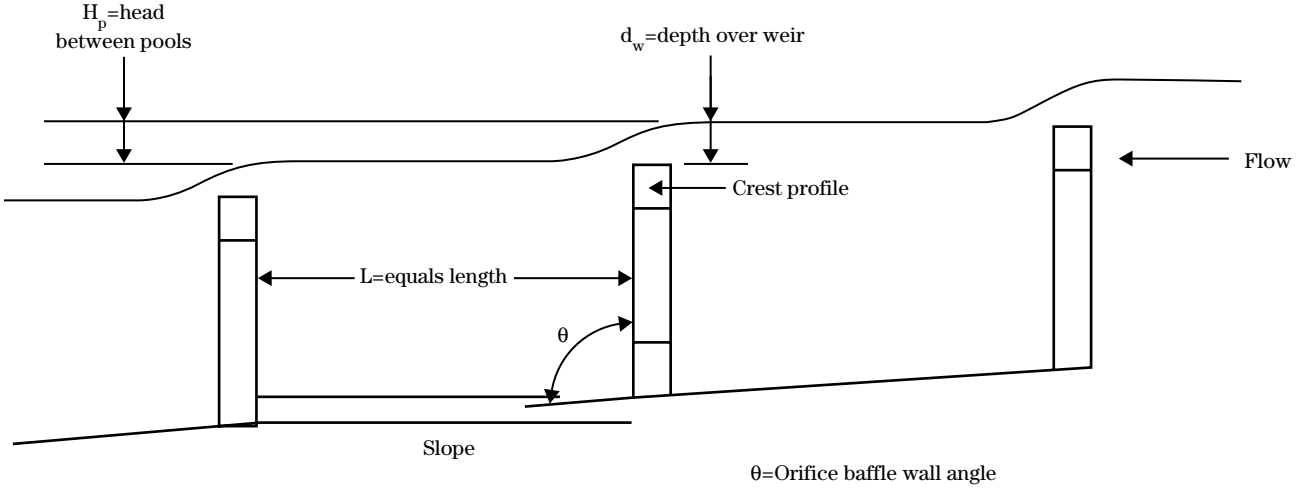
where:

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

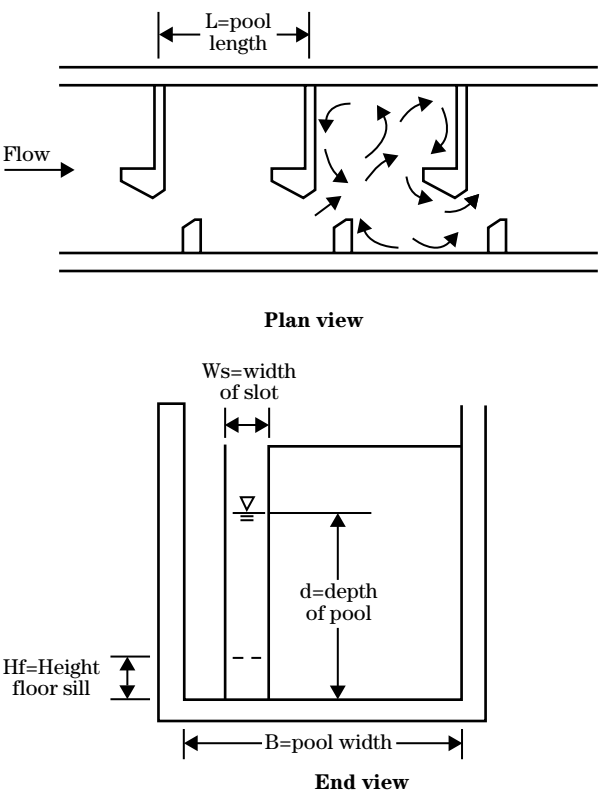


Figure TS14N–7 Weir schematic

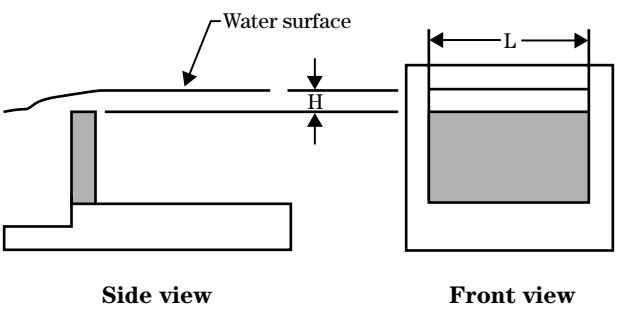
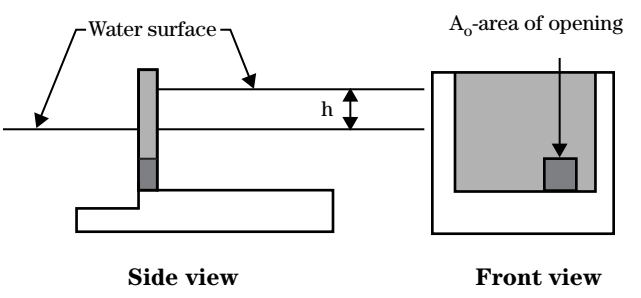


Figure TS14N–8 Orifice (gate) schematic



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

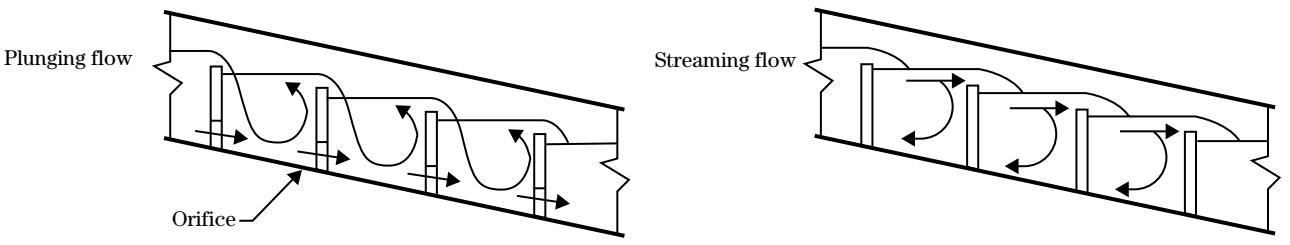
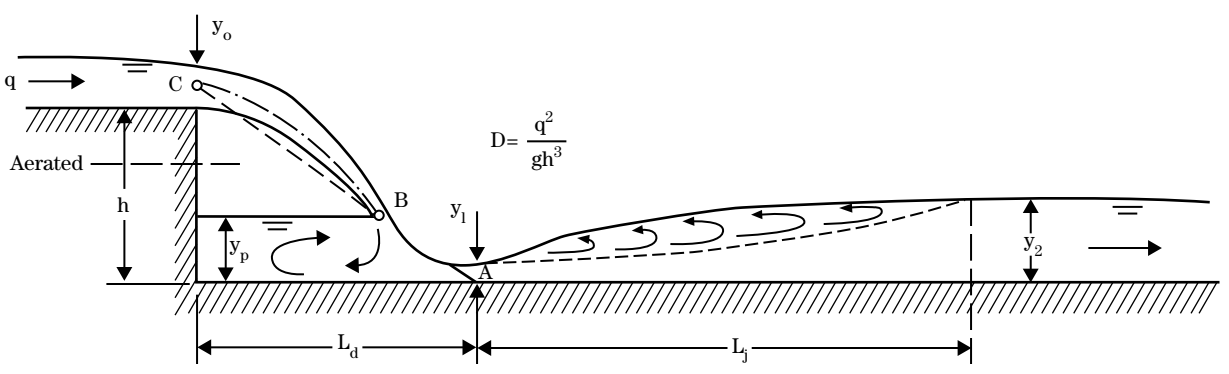


Figure TS14N–10 D number parameters for evaluating hydraulic jump geometry of a vertical drop spillway



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)

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

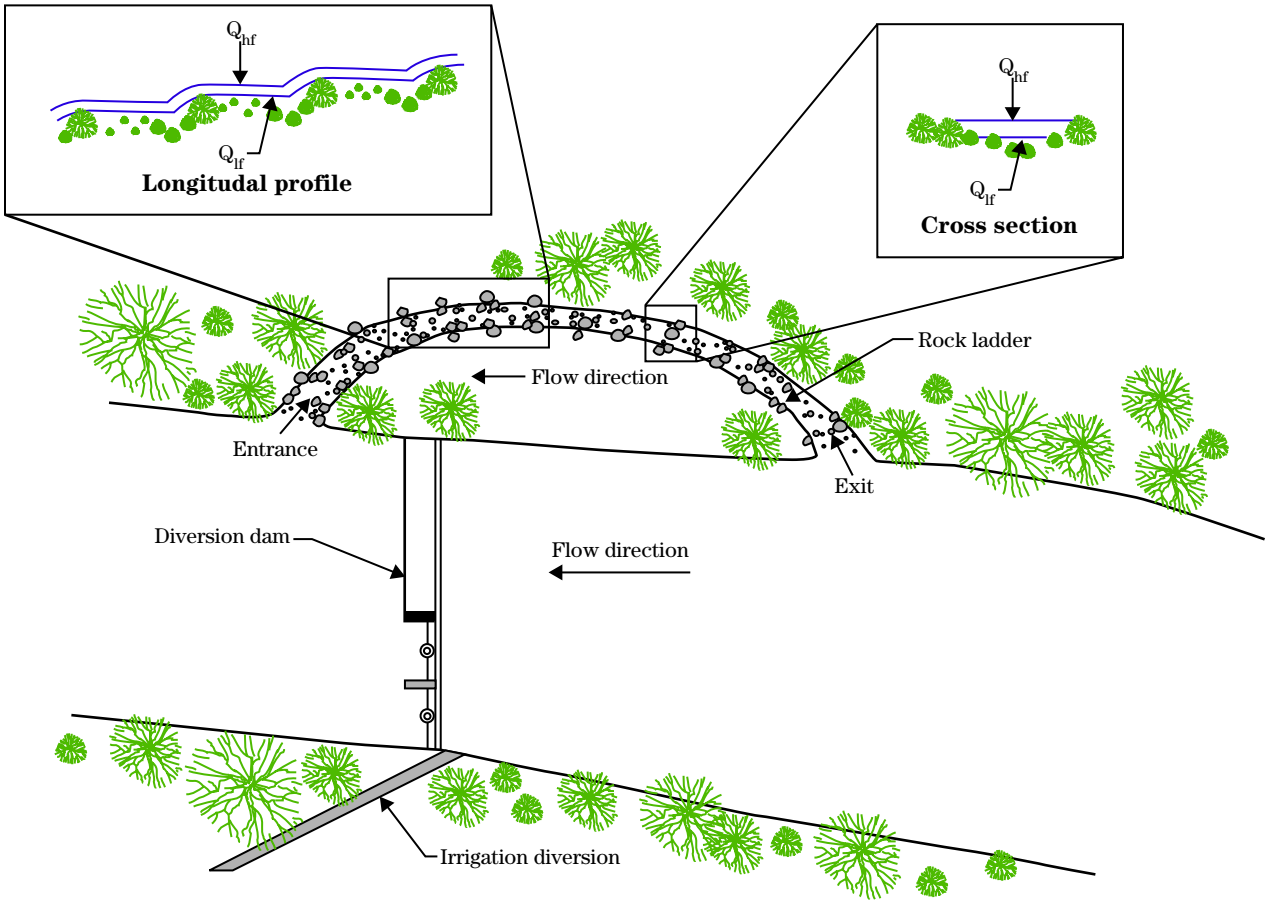
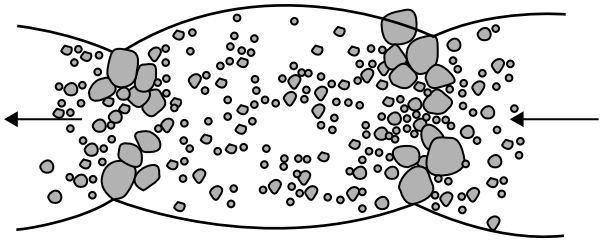
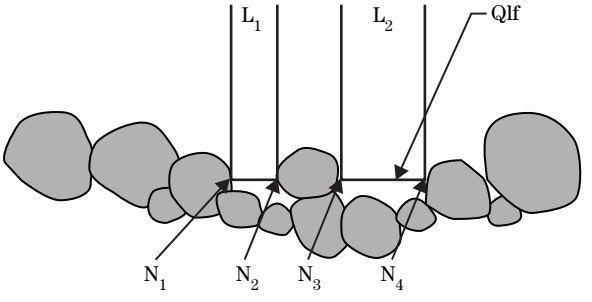


Figure TS14N–12 Boulder weir (plan view)



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

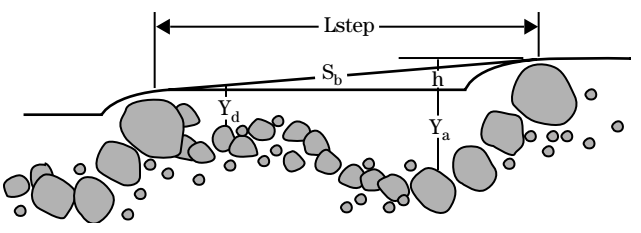
Boulder pools

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

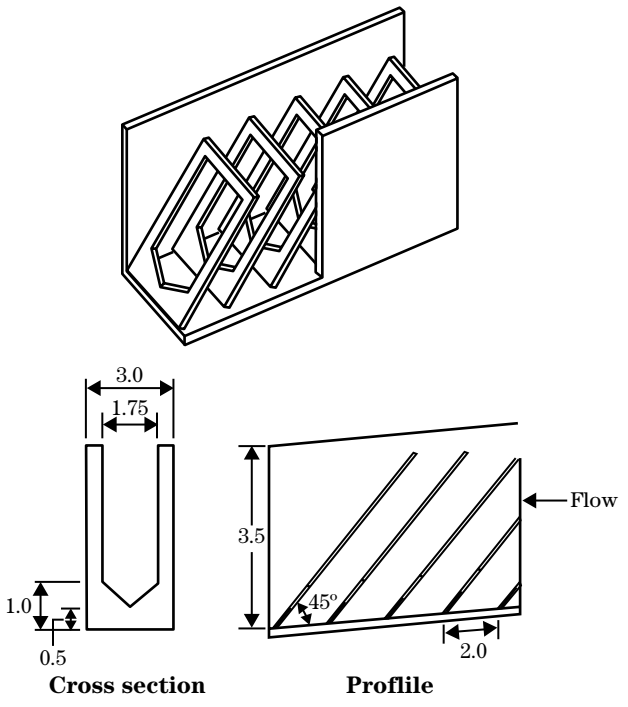
(a)

(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 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.

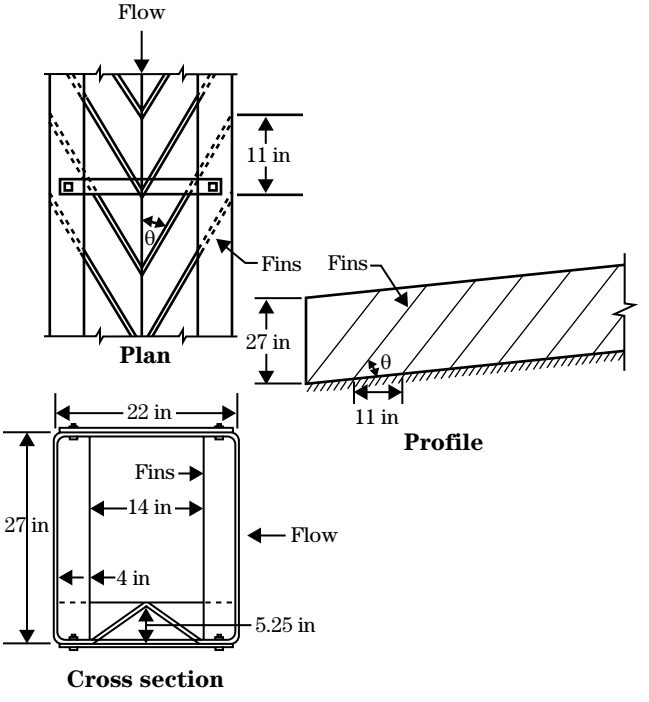
Baffle design in an Alaska Steeppass can be adjusted to fit the passage needs of target species. Although the floor fin angle (Ø) is generally 45 degrees, side fin angle (θ) adjustments from 45 to 90 degrees are commonly used to reduce air entrainment and structural turbulence to improve passage conditions.

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

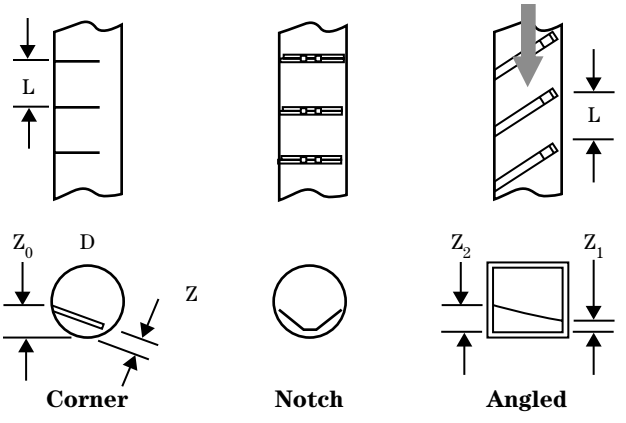


Figure TS14N–21 Round, corrugated metal culvert retrofitted with corner baffles to improve fish passage

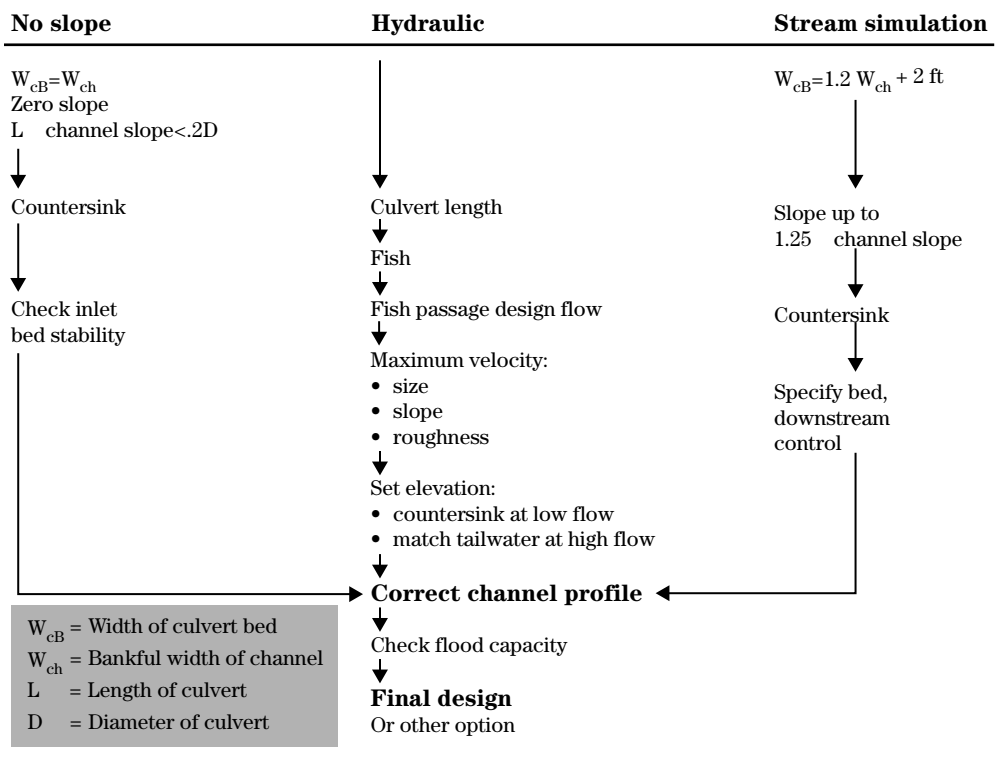
and expensive options such as rerouting a waterway or building a bridge may require consideration. Road abandonment can be an option if a culvert barrier to fish passage is along a poorly maintained and/or unused road.

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



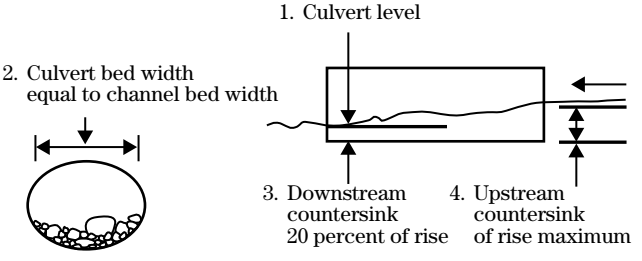
- upstream and downstream geomorphic instability (headcuts) is addressed
- adequate flood capacity exists

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

Maximum culvert length (ft) = 0.2H / channel slope / 12 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 = $1.0(W_{cb})$

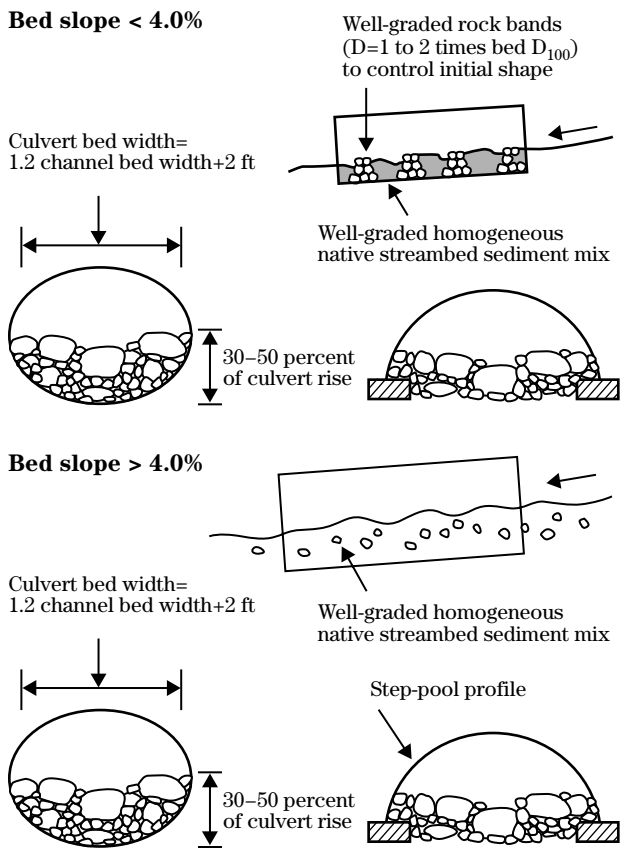
The preceding equation should be adhered to unless compelling evidence indicates otherwise. Deviations could lead to significant consequences, including inlet contraction scour, and smaller culverts will increase the chance of adverse outcomes over the design life of the crossing. If the stream is confined 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.

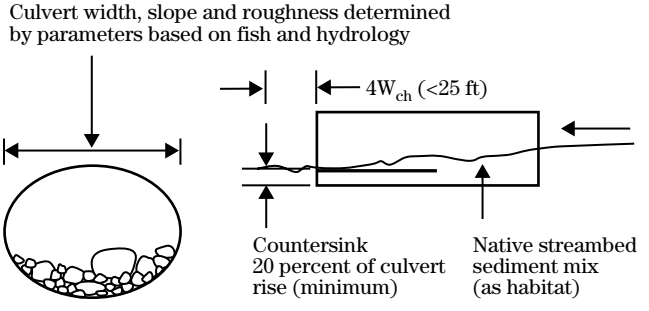
A general design sequence for developing a hydraulic design culvert is:

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

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 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).

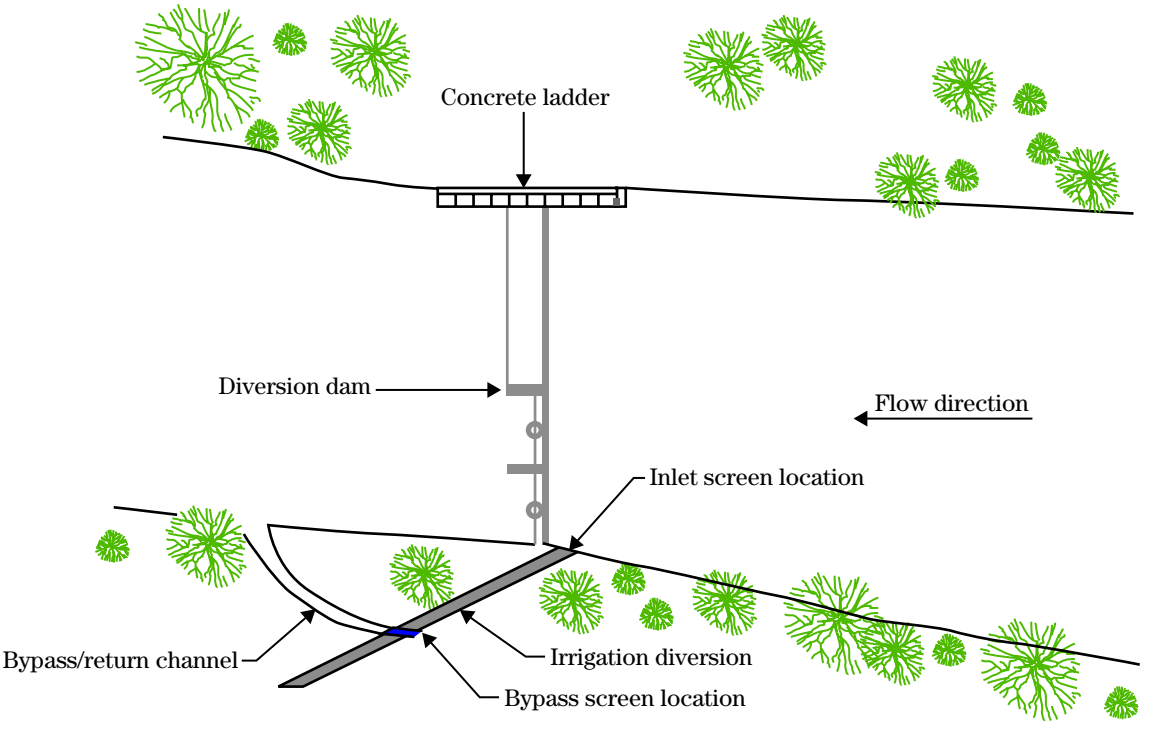
Fish screen biological design criteria

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

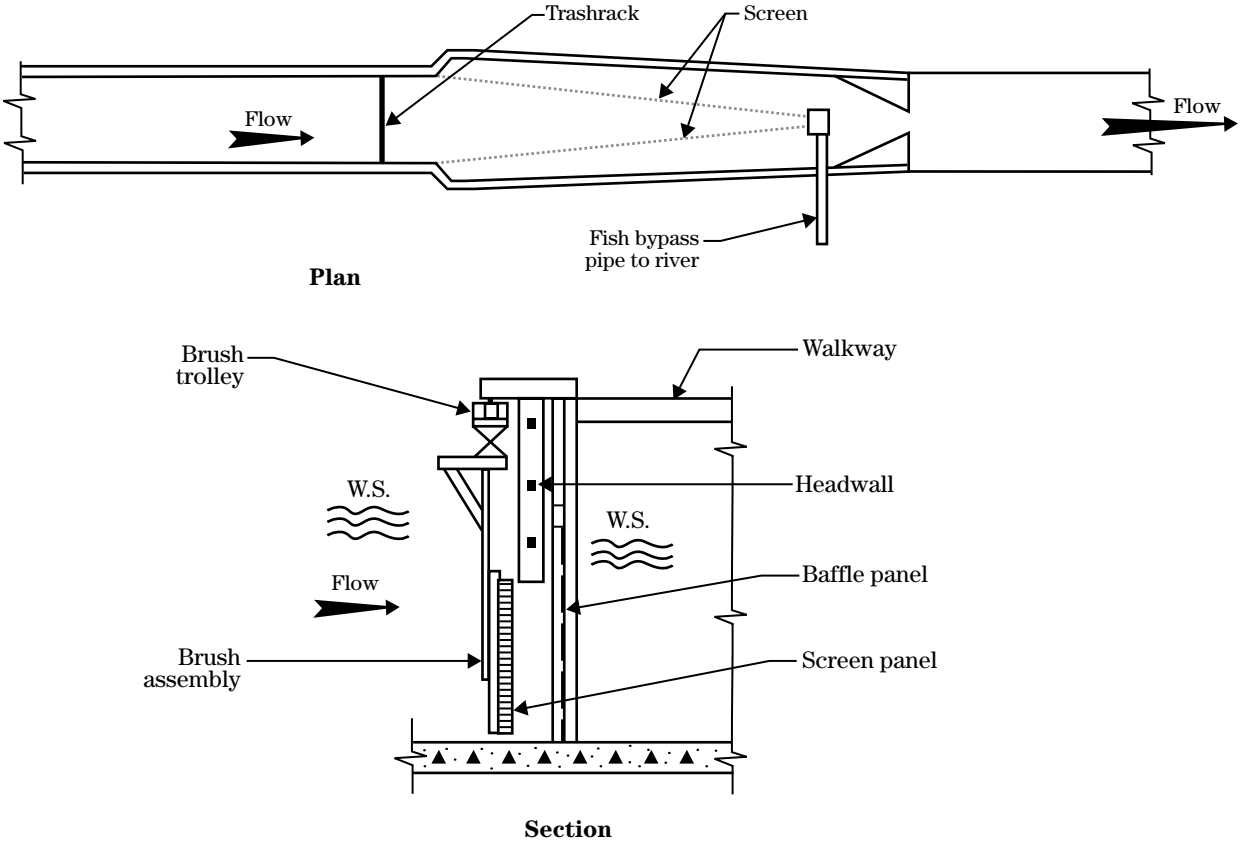
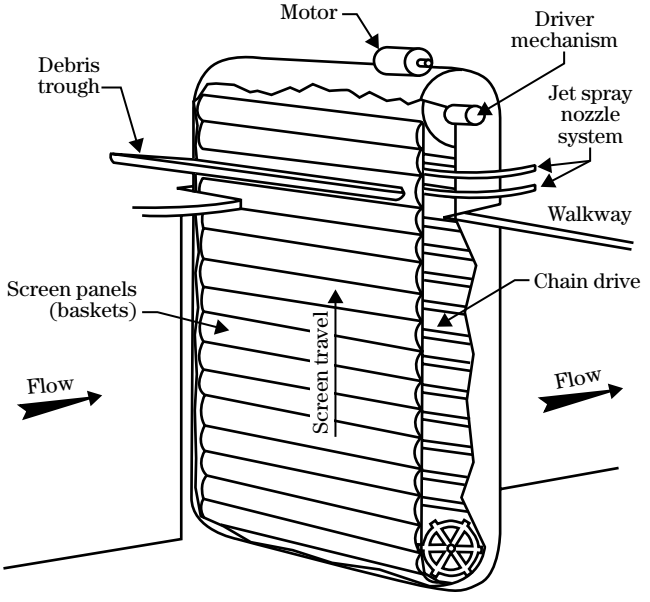


Figure TS14N–31 Vertical traveling screen



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.

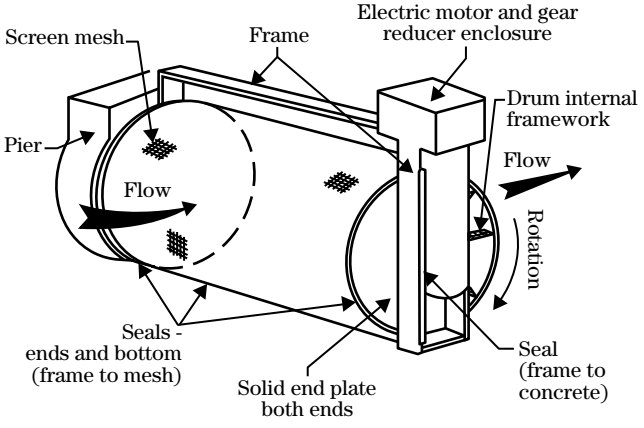
Rotary screens generally have less cleaning and maintenance requirements than a fixed or moving vertical plate screen, but side and bottom seals must be regularly inspected and replaced to ensure a fish tight facility. Each rotary drum is driven by a motor or mechanized propulsion system (commonly, a 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.

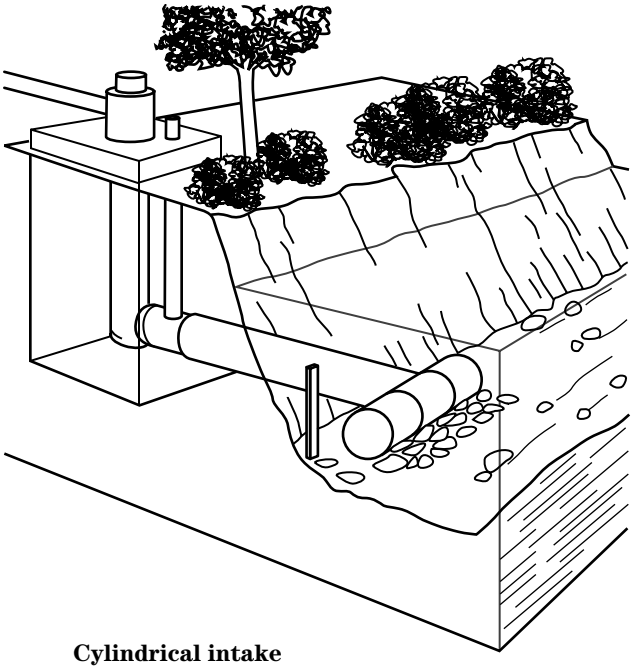
Infiltration galleries—Infiltration galleries can be used as a natural approach to pump or diversion intake design. Water infiltrates riverbed substrate, which acts as the fish screen, into a system of perfo-

Figure TS14N–32 Rotary drum screen



Isometric view

Figure TS14N–33 Pump intake screens



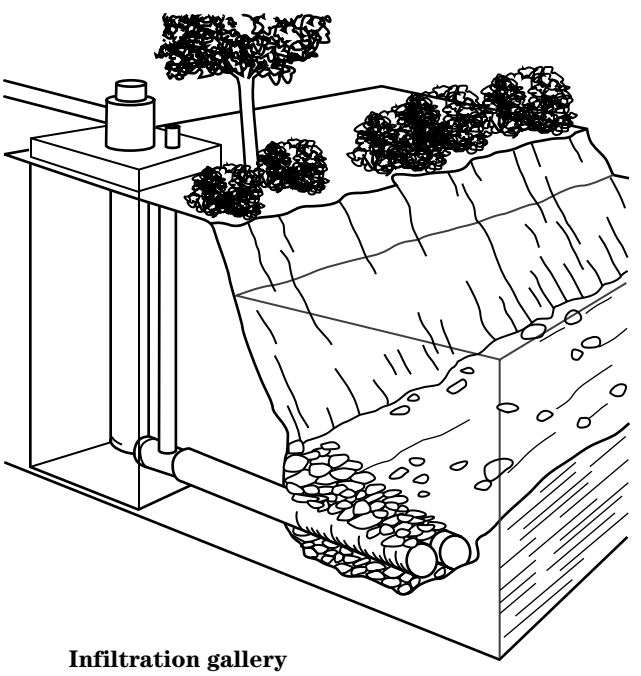
Cylindrical intake

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

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).

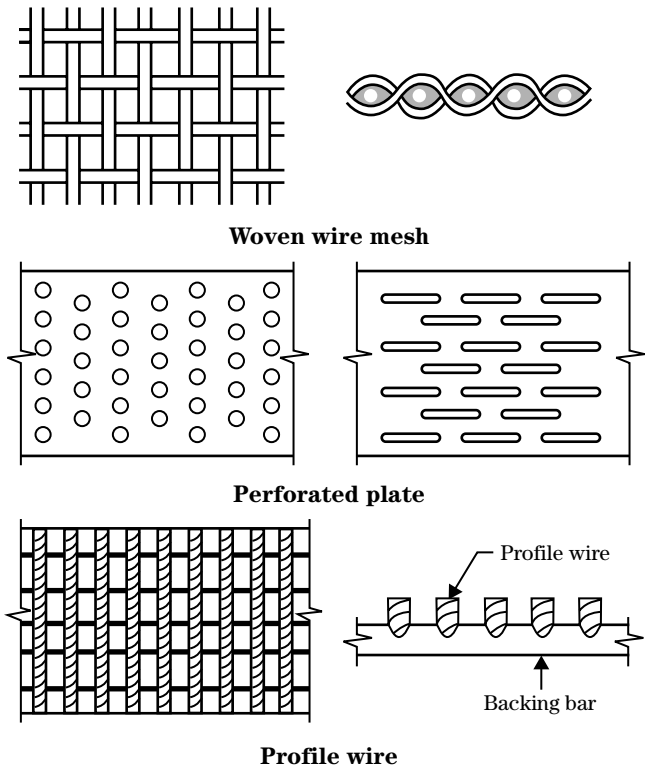
A primary screen parameter in evaluating screen design is the relationship between screen mesh size ($A_{opening}$) and the overall area of the screen (A_{screen}). For the purposes of this handbook, the ratio between the screen mesh size and the overall screen area is called the screen size ratio (eq. 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.

$$1 < \frac{V_{sweep}}{V_{approach}} \qquad \text{(eq. TS14N–29)}$$

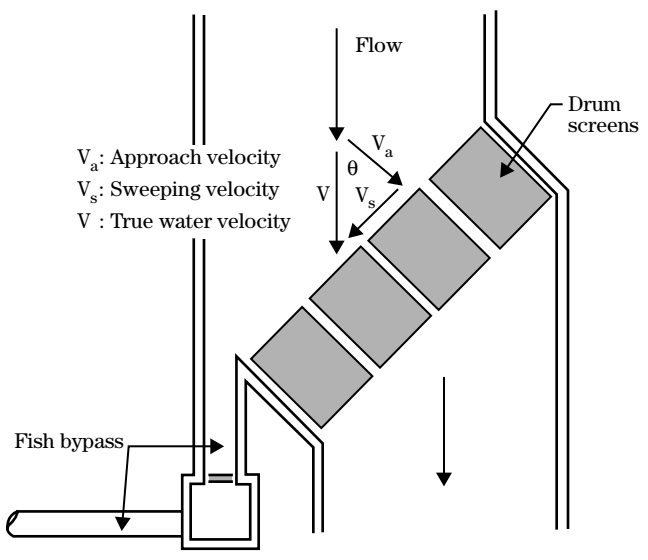
$$V_{sweep} = V \cos\theta \qquad \text{(eq. TS14N–30)}$$

$$V_{approach} = V \sin\theta \qquad \text{(eq. TS14N–31)}$$

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)

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.

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.

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.

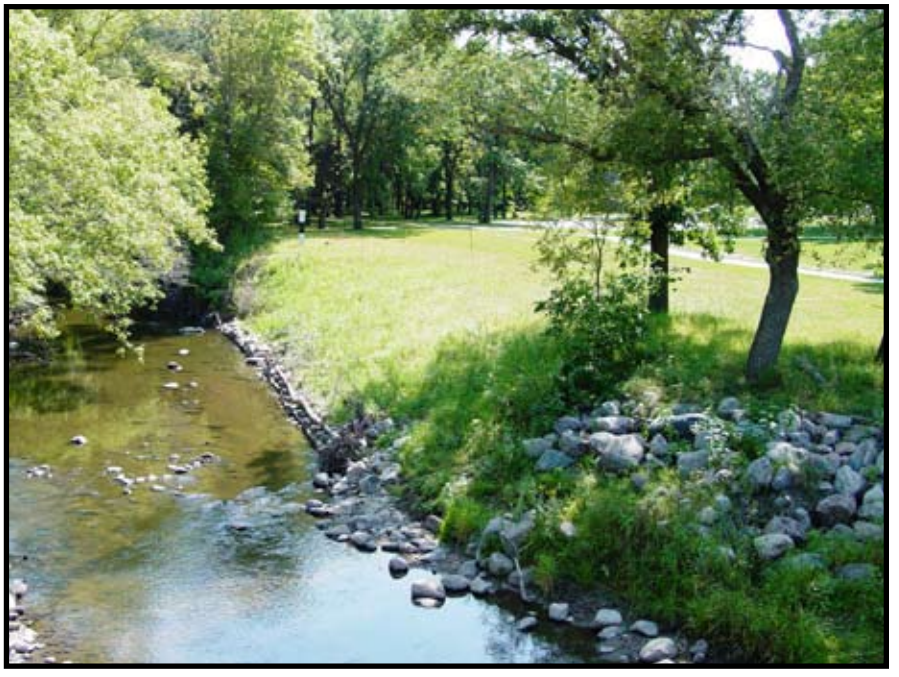
$$D_{50} = \left(\frac{V_1}{2.57}\right)^2$$

$$D_{50} = \left(\frac{10.1 \text{ ft/s}}{2.57}\right)^2 = 15.4 \text{ ft}$$

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.

$$V_{weir} = \frac{100}{(12)(0.67)(2)} = 6.2 \text{ ft/s}$$

$$D_{50} = \left(\frac{6.2 \text{ ft/s}}{2.57}\right)^2 = 5.8 \text{ in} = 0.5 \text{ ft}$$



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.

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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.

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.

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.

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.

Figure TS14O–2 LUNKERS installed over a stone base

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"

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.

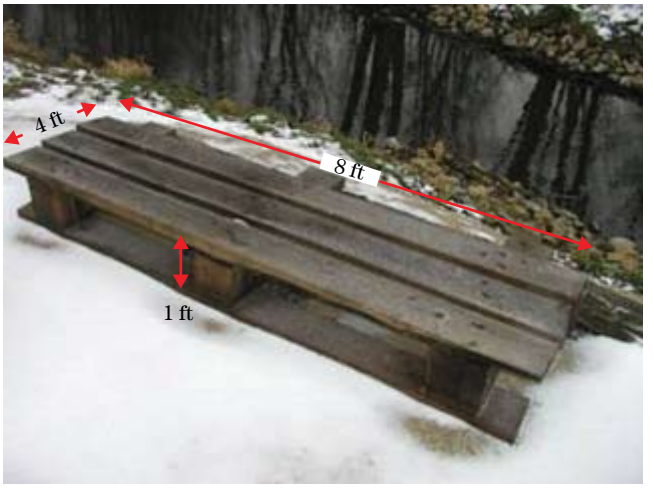
If needed, drill two 9/16-inch holes in the top plank of the outer two spacers on the *bankside*. These holes should be placed on the inside of the *bankside* plank, as close to the *bankside* rectangular blocks as possible.

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

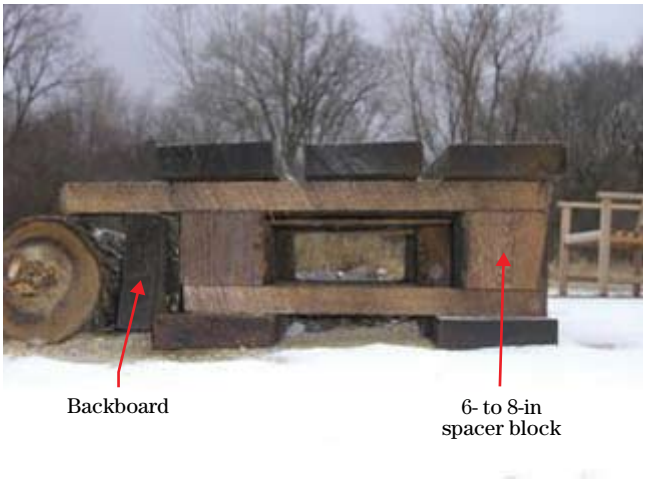
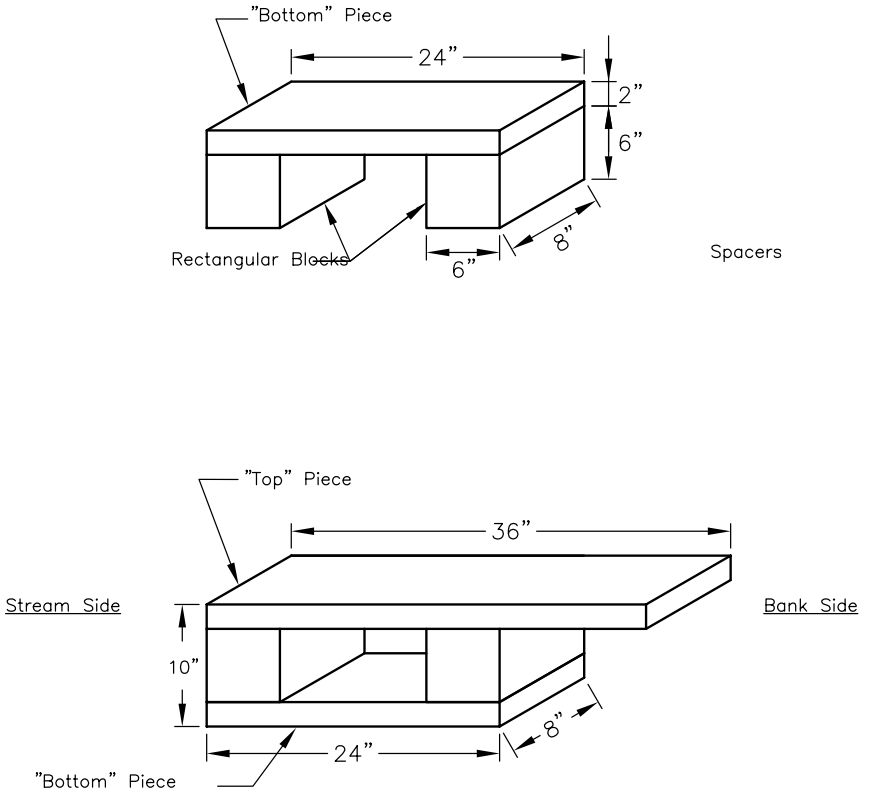


Figure TS14O–5 LUNKERS construction concept plan

(a) Step 1



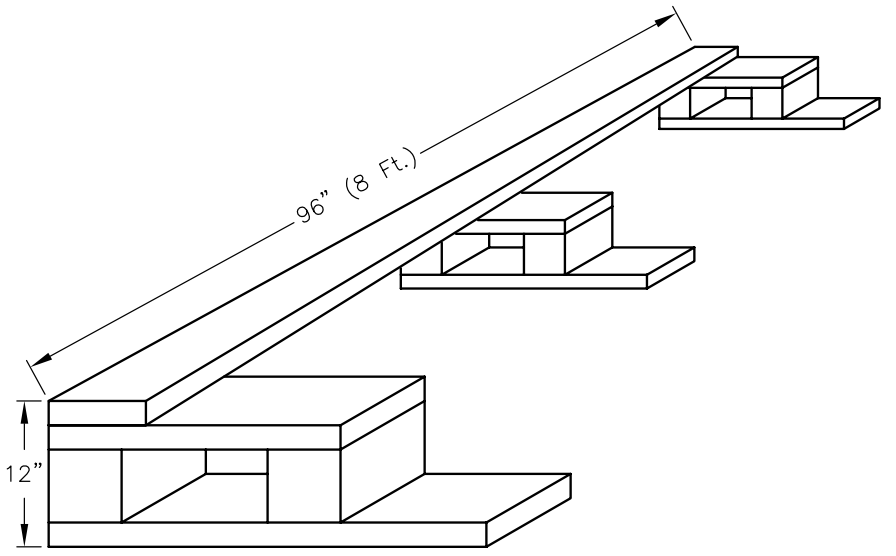
Step 1: Bulid SPACERS

Build 3 equal spacers as shown using oak or other wood that is strong and rot resistent. Use 20d nails min.

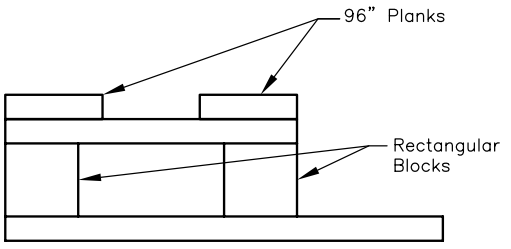
NRCS Natural Resources Conservation Service United States Department of Agriculture	LUNKER ASSEMBLY	J. Fripp Date Designed C. Haag, K. Worster 08/05 Drawn K. Miller, J. Renteria 08/05 Checked Approved	File Name Lunker–1.dwg Drawing Name Lunker Assembly 09/15/05 Sheet of

Figure TS14O–5 LUNKERS construction concept plan—Continued

(b) Step 2



Perspective Side View (One Plank in Place)



Simple Side View

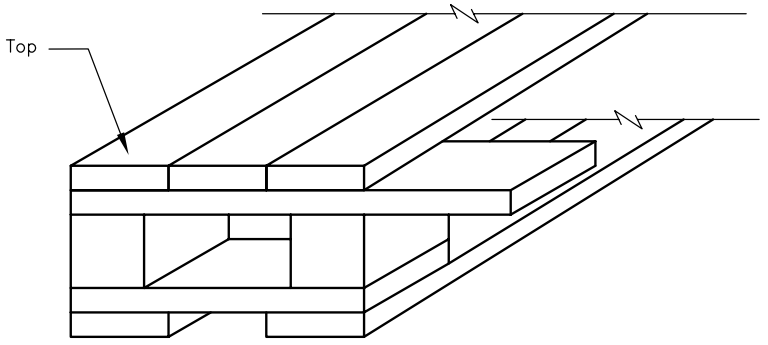
Step 2: FORM THE LUNKER BOTTOM

Bridge the spacers with two 8 ft. planks.
The spacers should be evenly spaced.
Secure each plank with 20d nails.

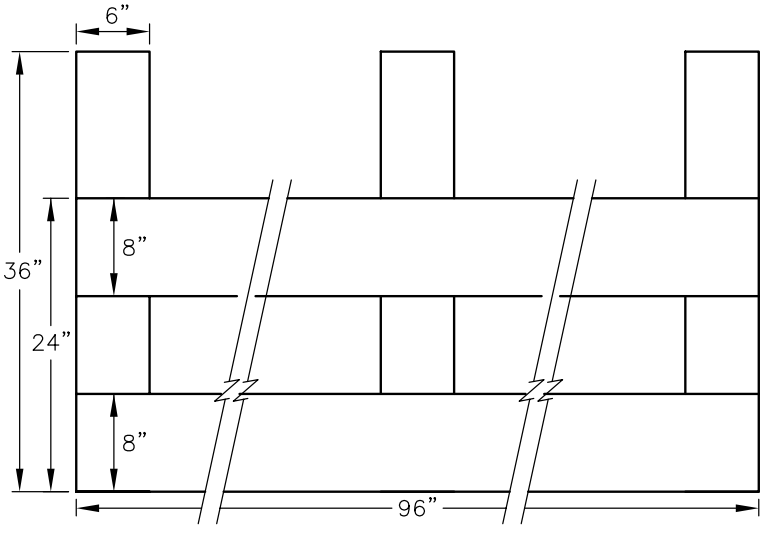
NRCS Natural Resources Conservation Service United States Department of Agriculture	LUNKER ASSEMBLY	Designed J. Fripp C. Haag, K. Worster Date 08/05 Drawn K. Miller, J. Renteria 08/05 Checked Approved	File Name Lunker–1.dwg Drawing Name Lunker Assembly Step 2 09/15/05 Sheet of

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)

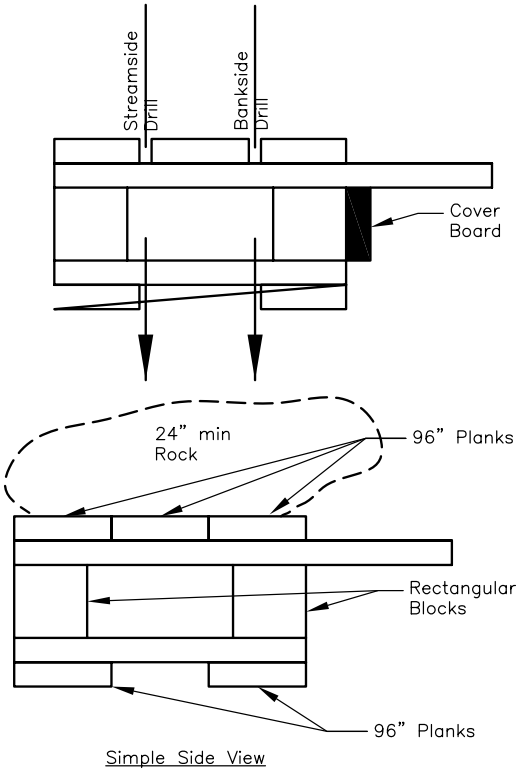
Step 3: FORM THE LUNKER TOP

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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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.

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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.

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).

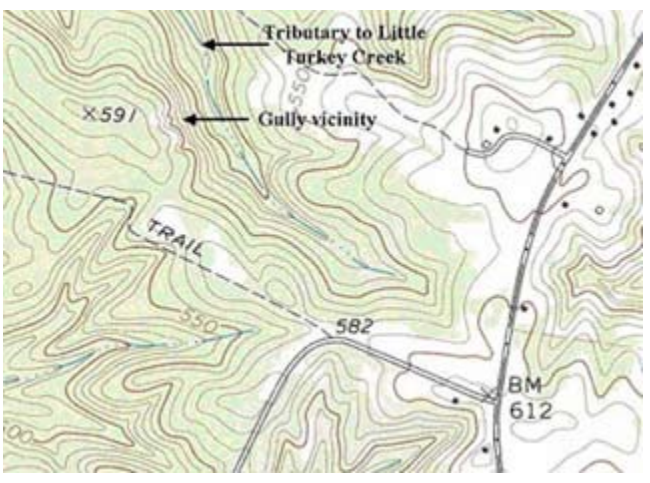
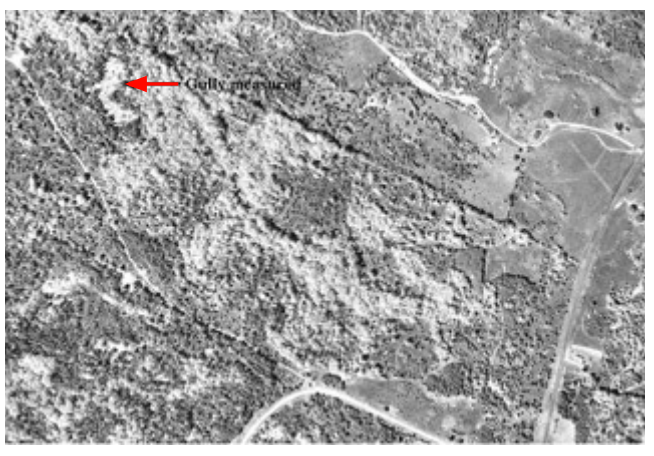


Figure TS14P–2 Aerial photo taken in 1974 highlights vicinity of gully marked in figure TS14P–1. The long gully (arrow) is about 0.25 acres and has delivered more than 53 tons of sediment over the last decade.



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

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.

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–6 Ephemeral gullies and severe sheet and rill erosion in unprotected northwestern IA field

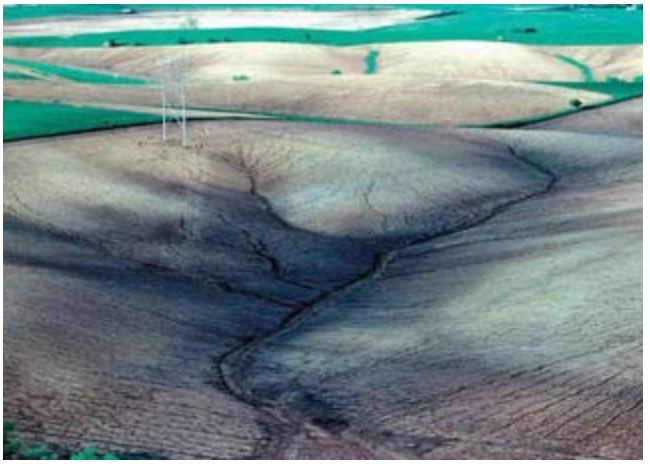


Figure TS14P–7 Perennial gully channel

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.

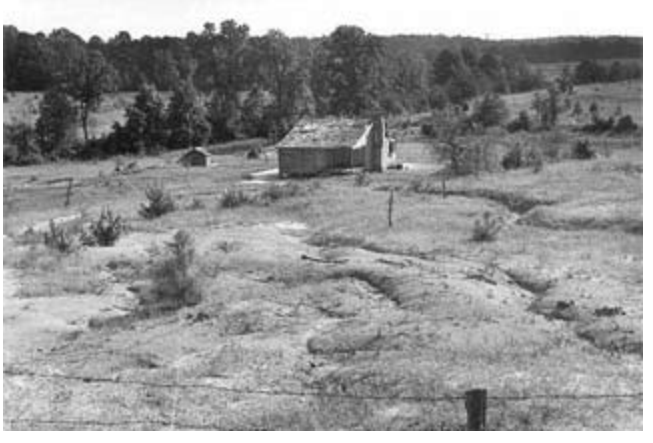
Some critical conditions (alone or in combination) could cause rejuvenation of gullies and channel forming processes with rapid erosion and expansion of the drainage network (Schumm, Harvey, and Watson 1984). Land use, soil, climate, rainfall, and hydrology are some of the leading considerations in evaluating 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 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).

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.

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 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.

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.

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.

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.

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 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 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-filledgeotextiles 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.

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

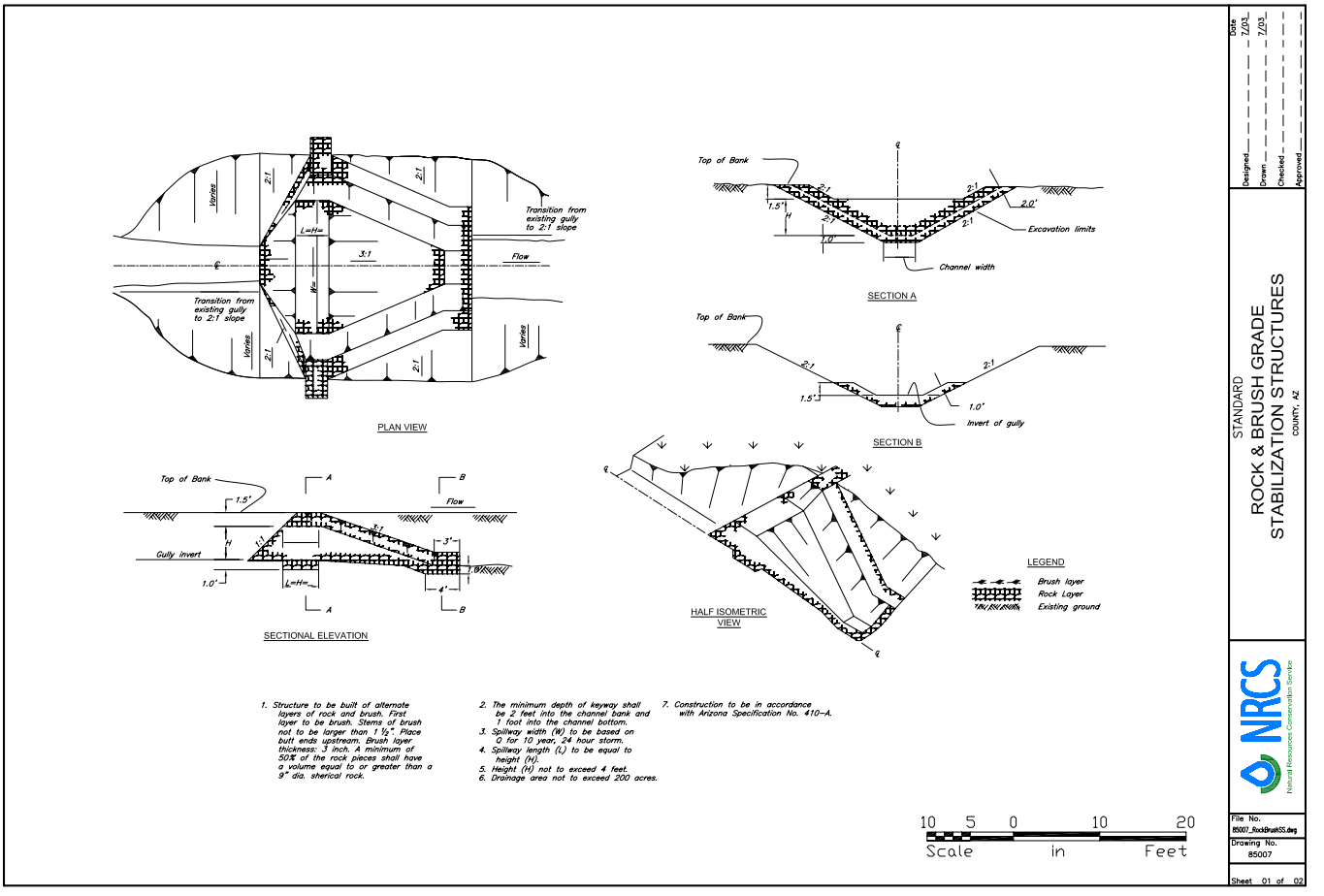


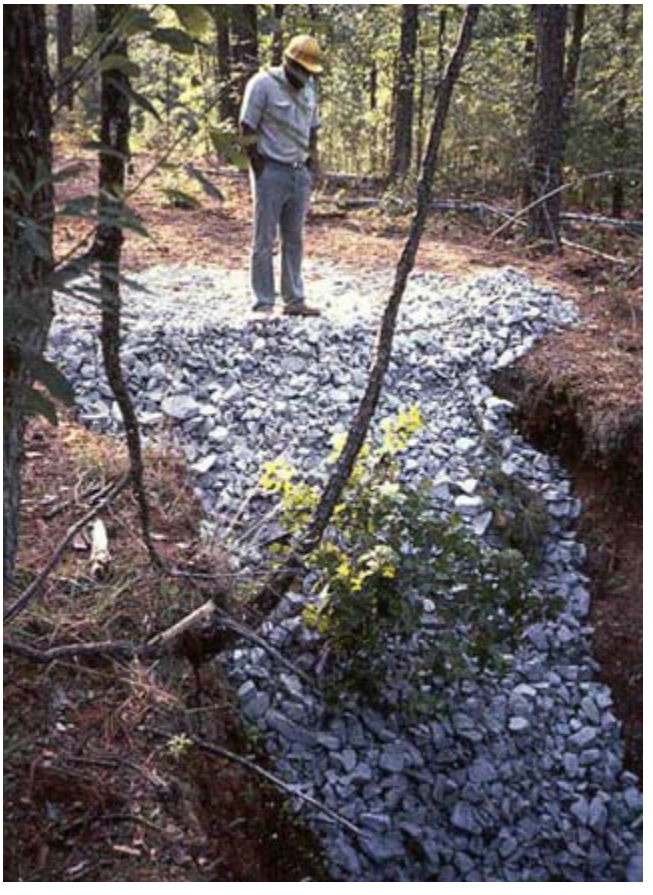
Figure TS14P–27 Soilcrete used to construct a small grade control structure

Figure TS14P–28 Reshaped gully with terrace failure, Abbeville County, SC

Figure TS14P–29 Grade stabilization structure

Rock 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

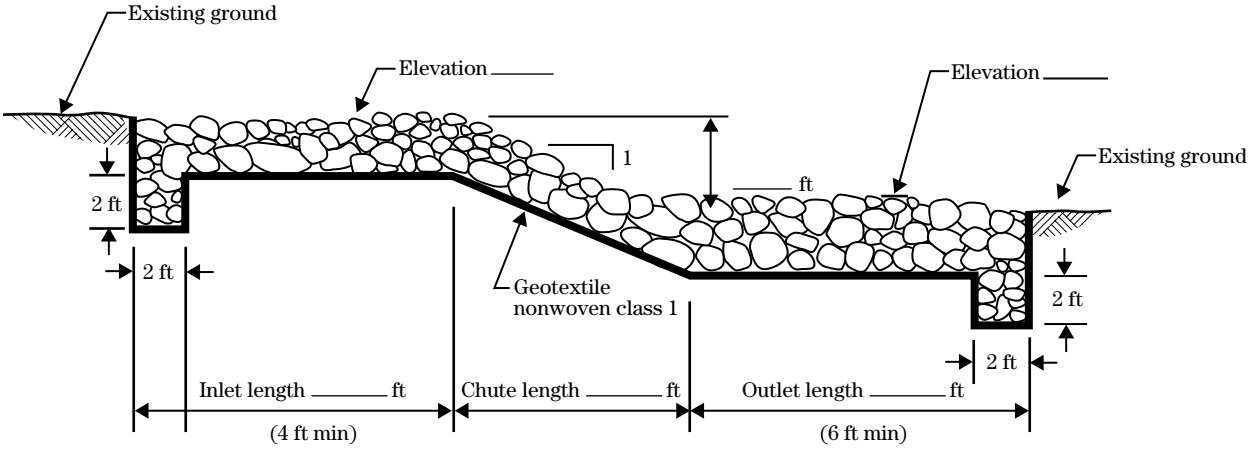


growth will develop in 3 to 5 years to control runoff. Rock placement provides immediate benefits, but can be costly when materials are not readily available and where hand labor is required.

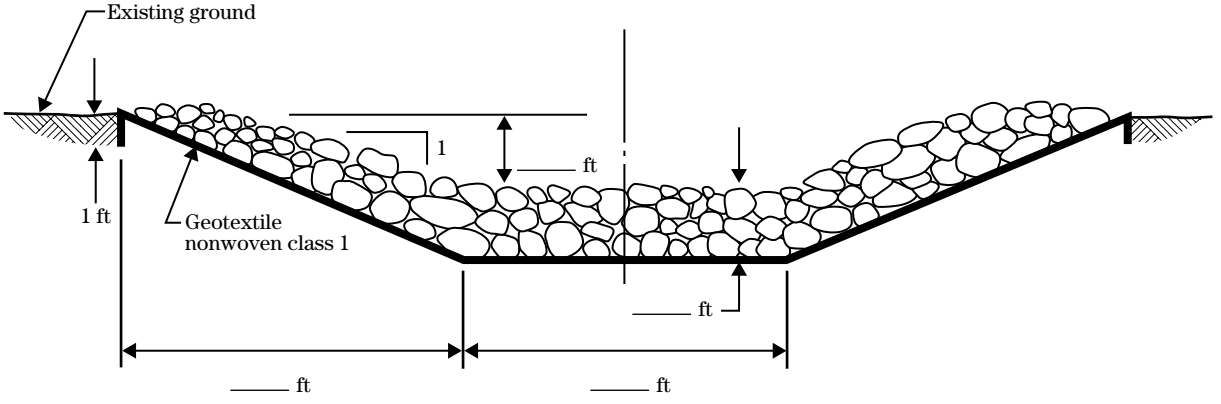
Rock chutes are grade control structures that are designed to reduce instream gradients or provide stable entry for surface water drainage to a stream. An example of a rock chute is shown in figure TS14P–31, and example design drawings for a rock chute are shown in figure TS14P–32 (NRCS–MI). Rock chutes 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 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 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.

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 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.

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.

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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

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).

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):

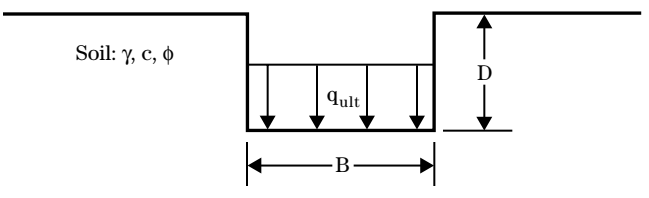
Figure TS14Q–1 Farm bridge with steel I-beam structural members and timber strip footing (*Photo courtesy of Ben Doerge*)

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.

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.

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

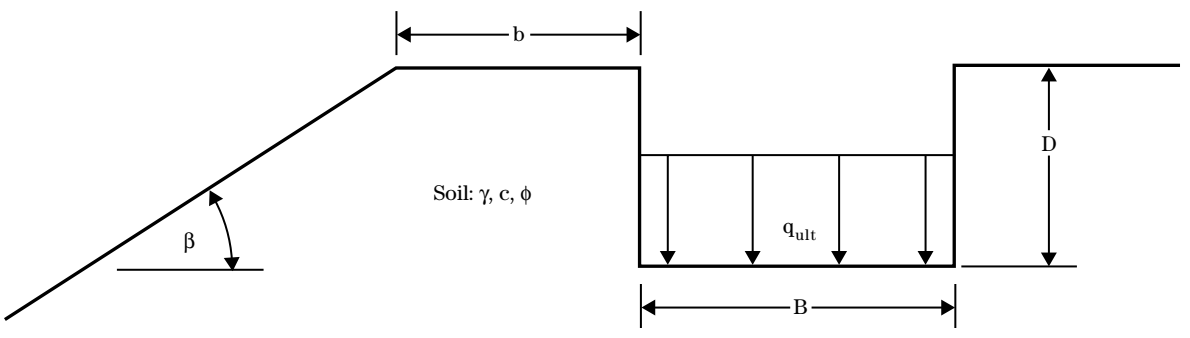
compared to the flat ground case. The reduction is the result of the loss of some of the available shear resistance on the sloping side due to the shortening of the failure surface and decrease in the total weight acting on the failure surface. The bearing capacity for footings adjacent to slopes is 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.

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.

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.

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.

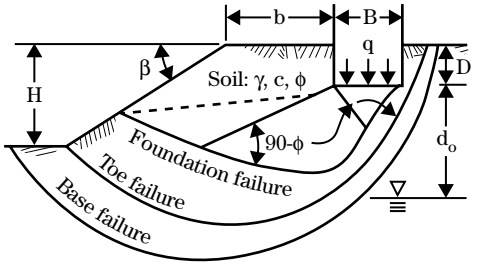
A qualified soils engineer should be consulted when selecting shear strength parameters for design.

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

$$N_o = \frac{\gamma H}{c}$$

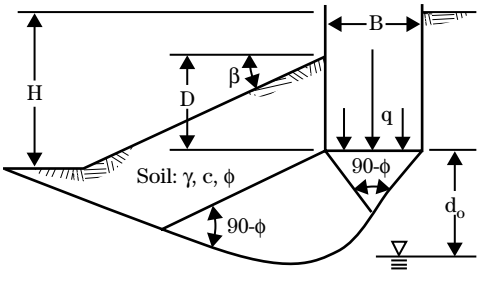
Interpolate for values $0 < D/B < 1$ for $0 < N_o < 1$. If $N_o \geq 1$, stability of slope controls ultimate bearing pressure.
Interpolate q_{u1t} between eq (a) and (b) for water intermediate level between ground surface and d_o = B. for water at ground surface and sudden drawdown: Substitute ϕ' for 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

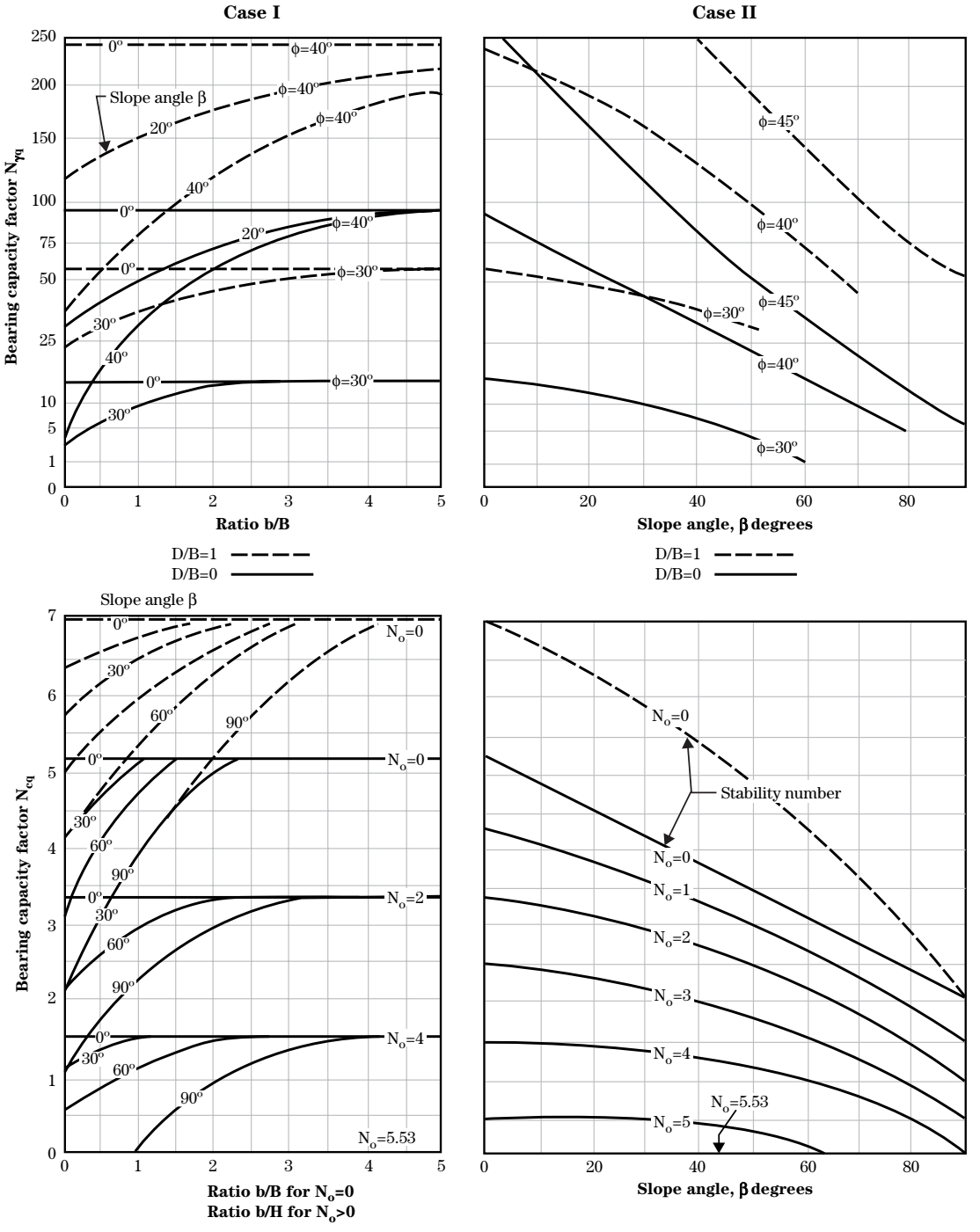


Table TS14Q–3 Meyerhof method design table (cohesionless soils)

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,

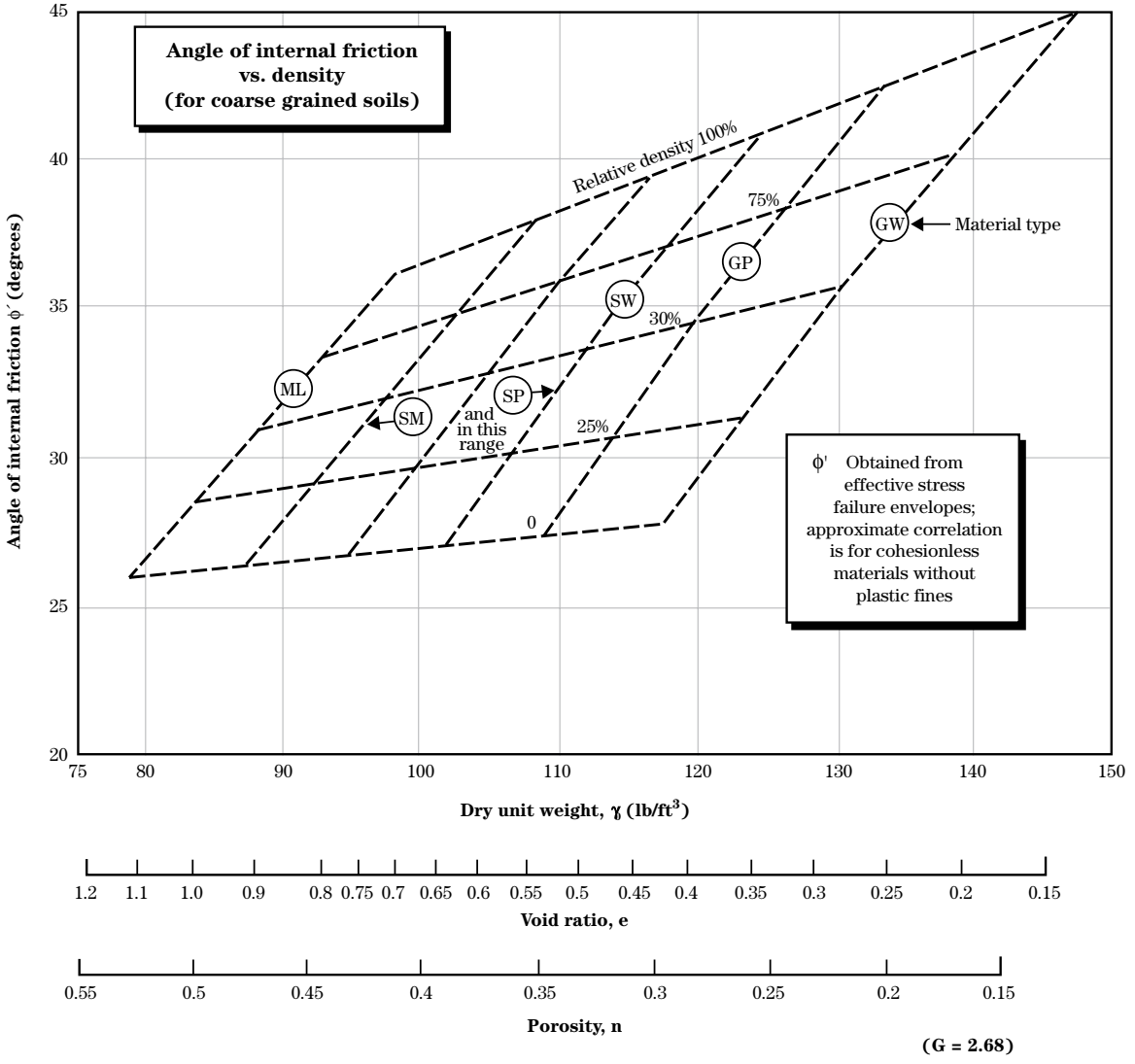
$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

$$e = \frac{G}{\gamma_d}\gamma_w - 1$$

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



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 $\phi = 35^\circ$, $N_{\gamma q} = 85$

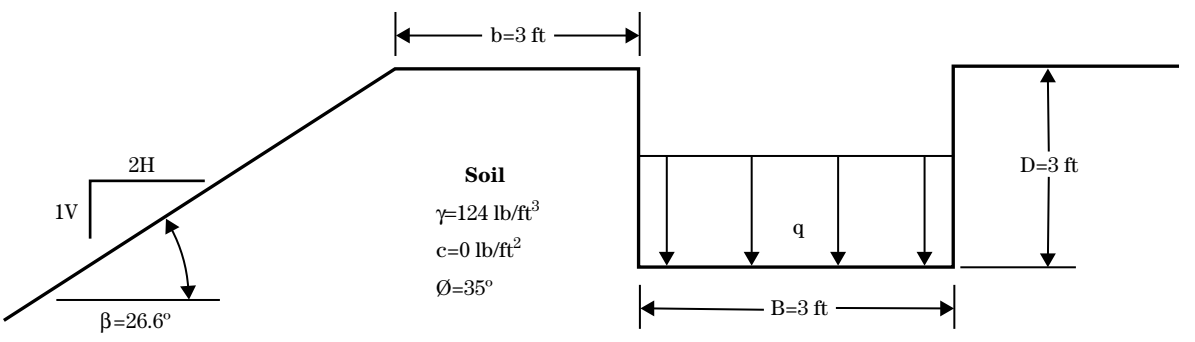
Solve for ultimate bearing capacity, q_{ult}, using equation TS14Q–2:

$$\begin{aligned} q_{ult} &= \frac{1}{2} \times \gamma \times B \times N_{\gamma q} \\ &= (0.5) \times \left(124\text{ lb/ft}^3\right) \times (3\text{ ft}) \times (85) \\ &= 15{,}810\text{ lb/ft}^2 \\ &= 15.8\text{ k/ft}^2 \end{aligned}$$

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



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.

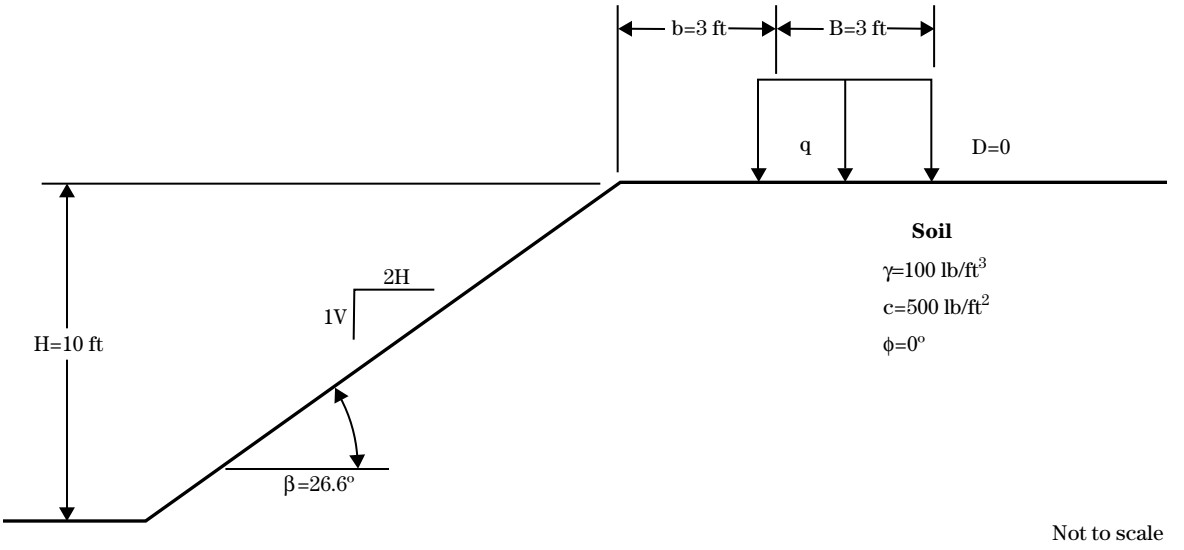
b/H	N_{cq}
0.50	3.18
0.33	3.11
0.00	2.97

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



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.

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Figure TS14R–5	Bank retaining wall cross section	TS14R–2
Figure TS14R–6	Sheet pile	TS14R–3
Figure TS14R–7	Sheet pile sections	TS14R–3
Figure TS14R–8	Concrete piles	TS14R–4
Figure TS14R–9	Wood pile sheets	TS14R–5
Figure TS14R–10	Empirical correlation between effective phi angle and PI	TS14R–7
Figure TS14R–11	Active and passive rotation	TS14R–8
Figure TS14R–12	Differential water load	TS14R–9
Figure TS14R–13	Surface surcharge loading	TS14R–9
Figure TS14R–14	Deep-seated failure	TS14R–10
Figure TS14R–15	Wall rotational failure	TS14R–10
Figure TS14R–16	Overstressed pile	TS14R–12
Figure TS14R–17	CWALSHT output file	TS14R–28

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.

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.

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

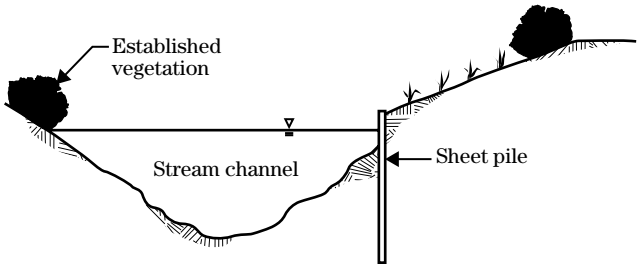


Figure TS14R–2 Flanking and undermining protection profile

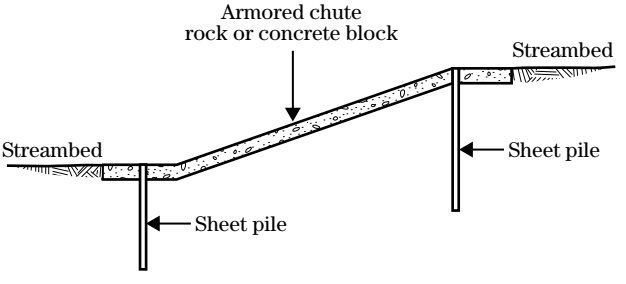
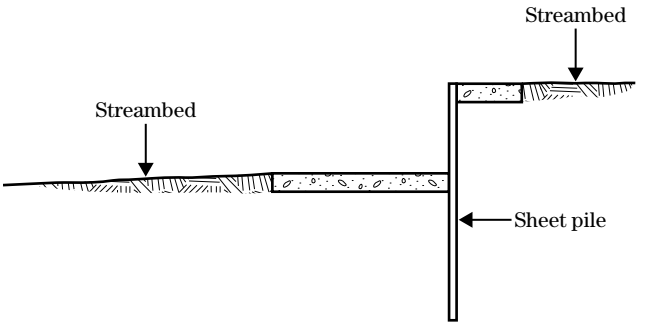
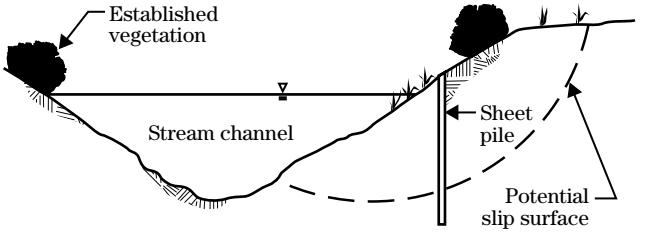


Figure TS14R–3 Streambed grade stabilization profile



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.

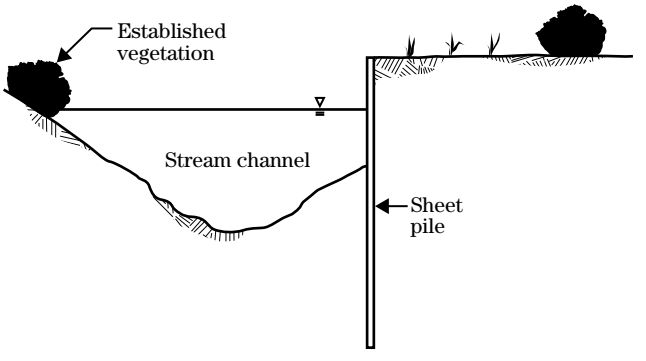
Figure TS14R–4 Bank slope stability cross section



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.

Figure TS14R–5 Bank retaining wall cross section



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

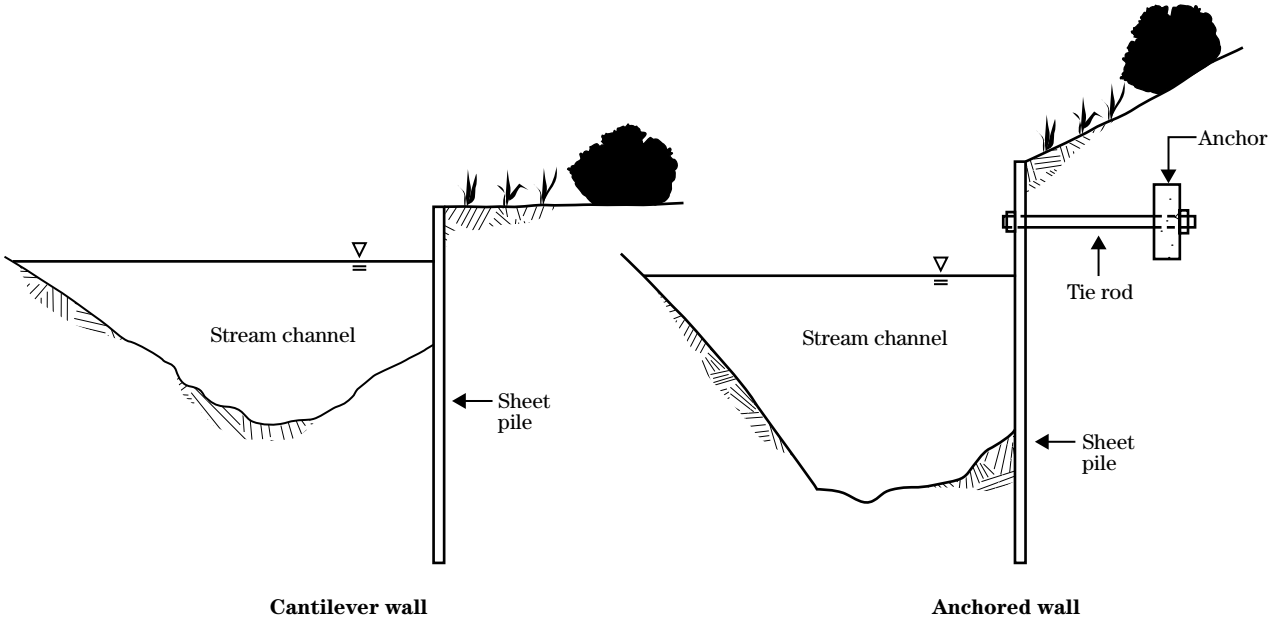


Figure TS14R–7 Sheet pile section

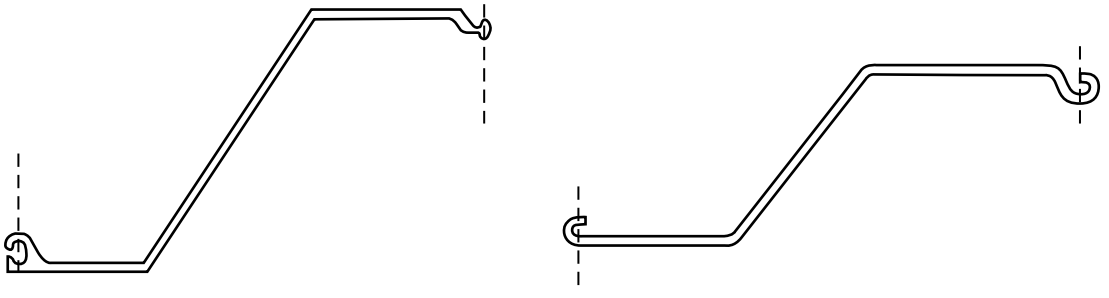
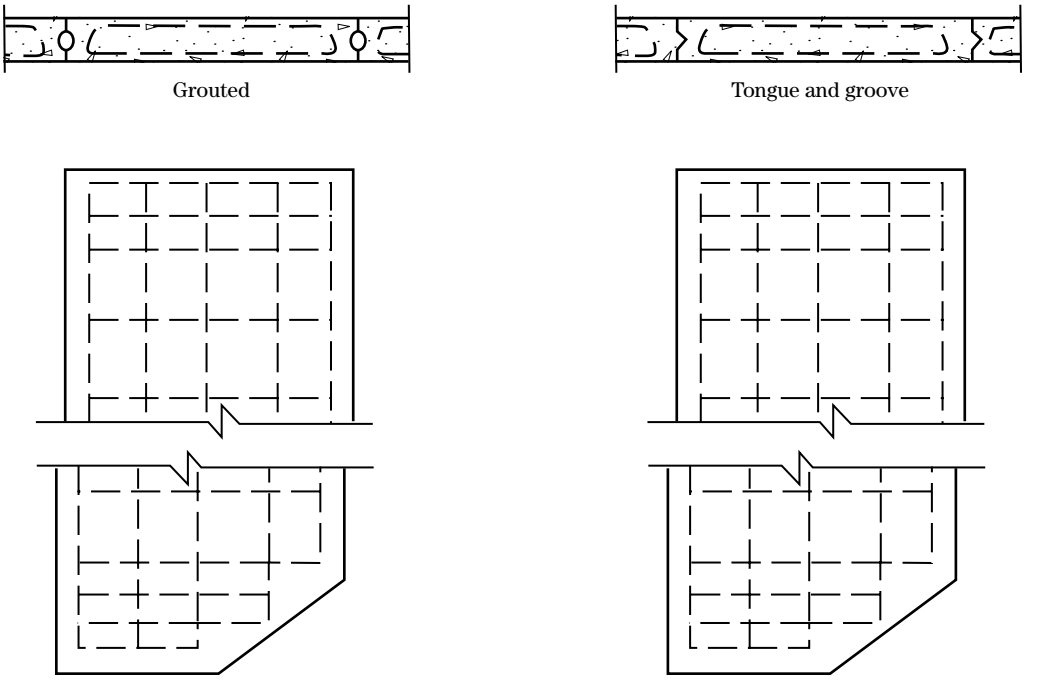


Figure TS14R–8 Concrete piles

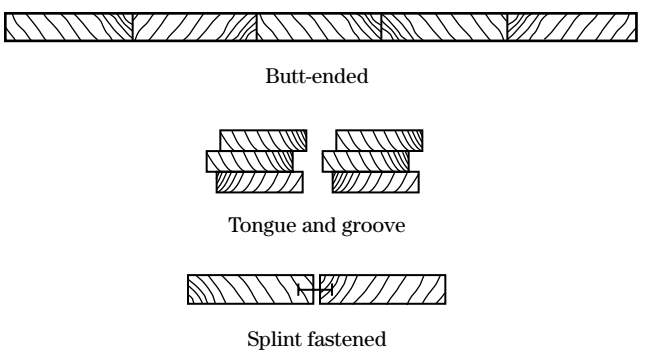


Wood piles may be either independent sheets or tongue-and-groove interlocking sheets (fig. TS14R–9 (USACE 1994c)). Wood piles are used on short wall heights and often anchored with wood walers and vertical wood piles. All wood components should be treated to reduce rot or damage due to wood-destroying insects or water borers.

Vinyl 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.

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



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 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.

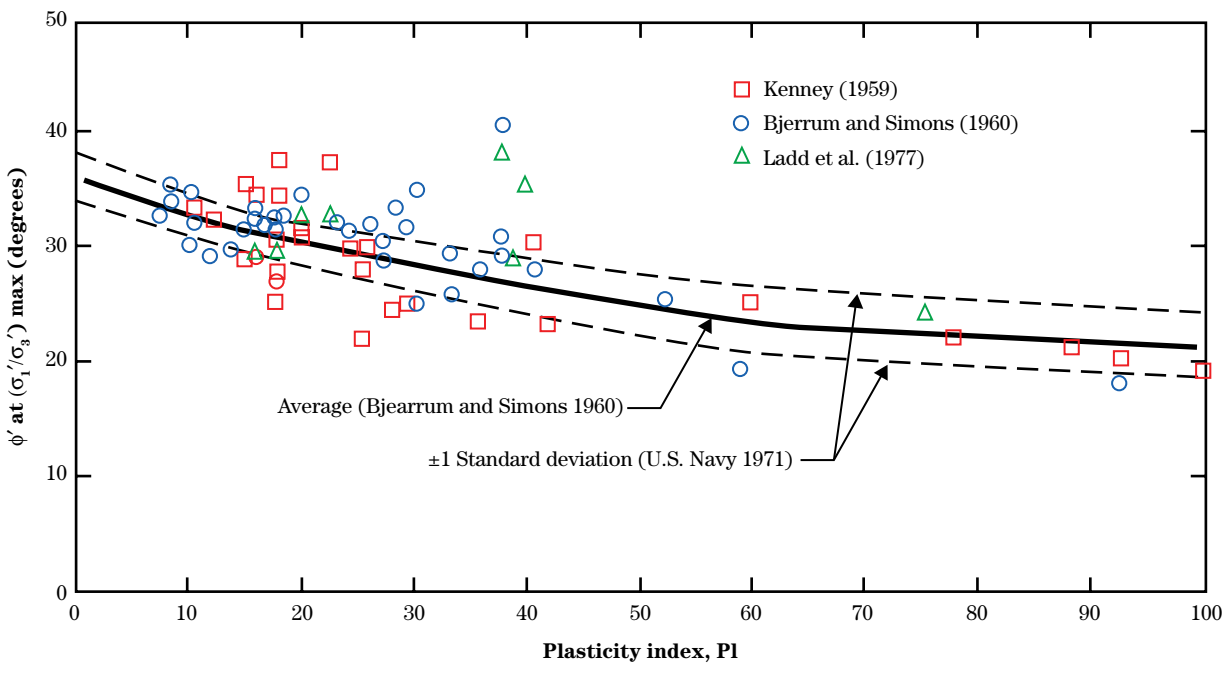
Table TS14R–2 Description of coarse-grain soil density

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



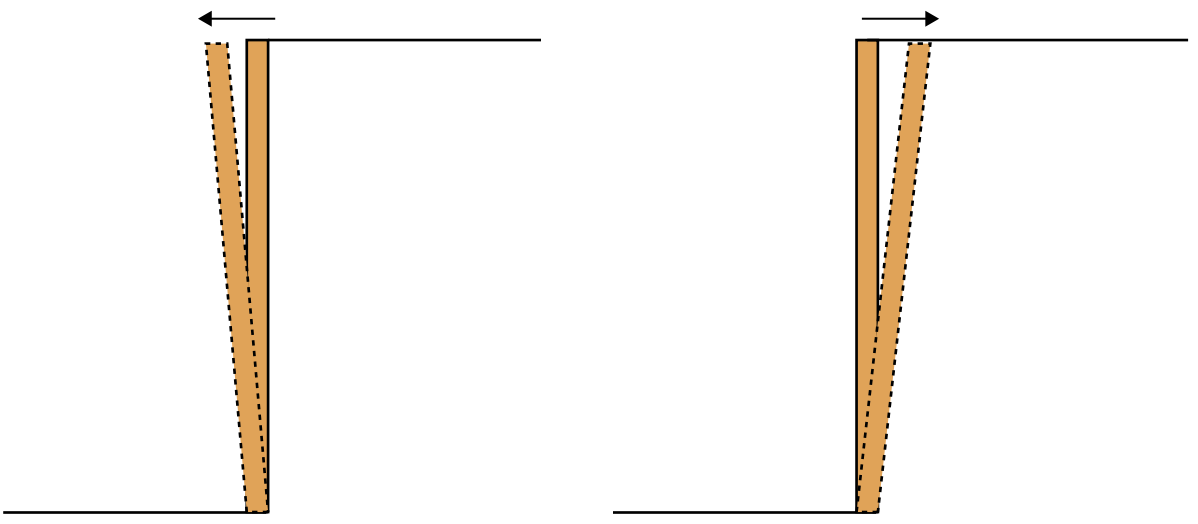
The lateral (horizontal) earth pressure is a function of the soil properties (cohesion, phi angle, and unit weight), height of overburden, and the elevation of the water table. Earth pressures varies from an initial state referred to as at-rest, K_o, to a minimum limit state referred to as active, K_A, 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

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.

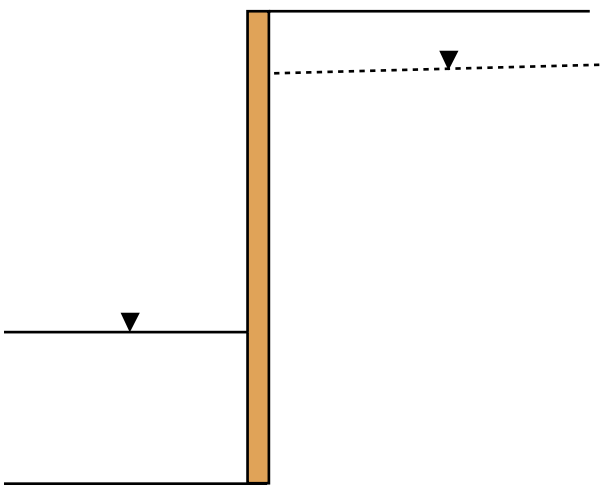
Surface surcharge (fig. TS14R–13) also exerts lateral pressure on the wall, forcing the wall outward. Typical surcharge loadings may be due to equipment (parked or traveling), storage areas, construction materials, vehicles, and others. Surcharge loads are often estimated to be 100 to 200 pounds per square foot. Other surcharge loads include spoil, snow, or ice.

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

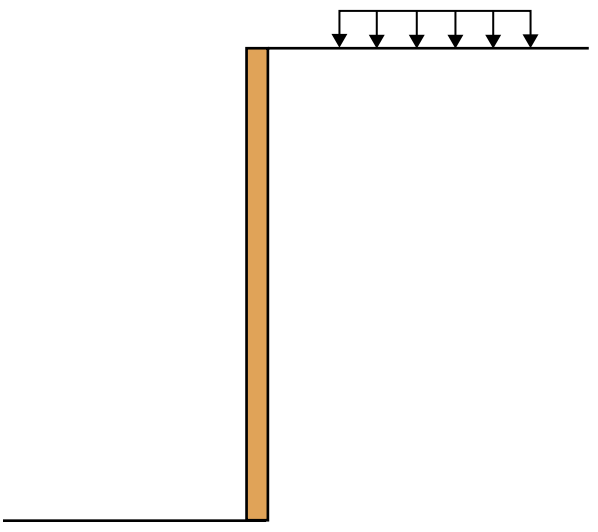
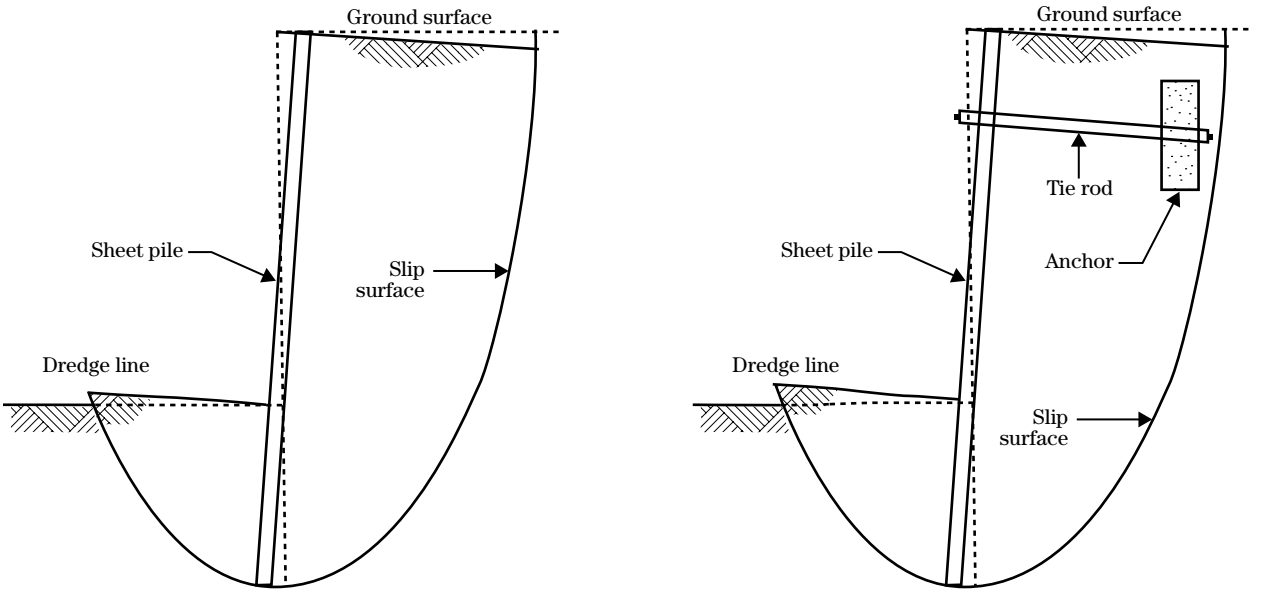
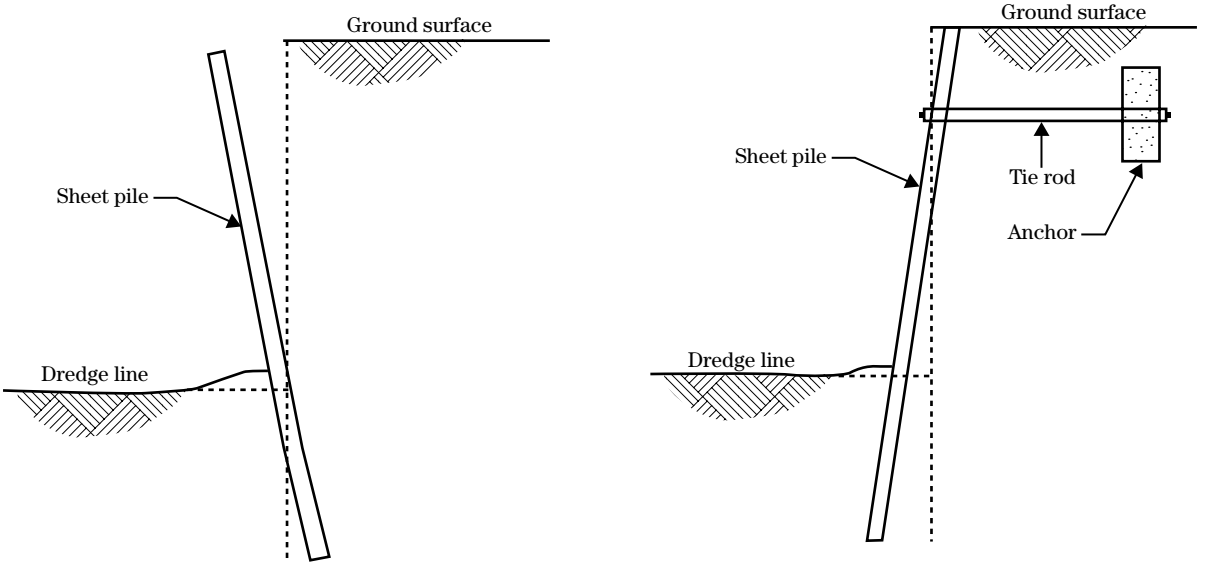


Figure TS14R–14 Deep-seated failure



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.

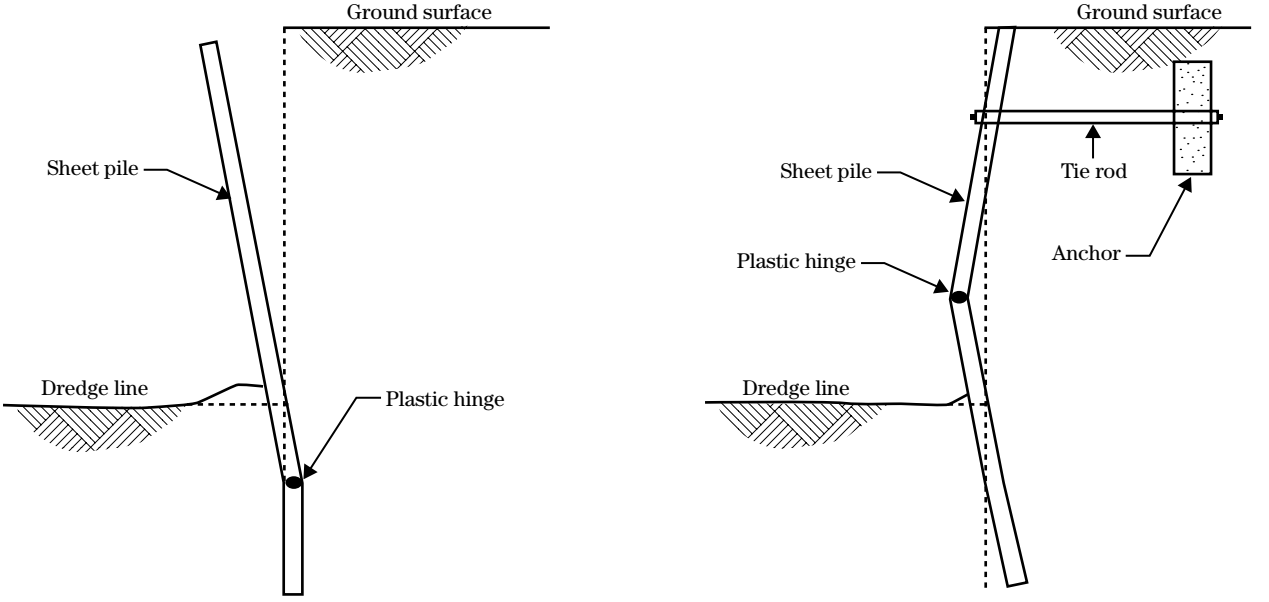
Moment—The maximum moment may be evaluated using methods presented in the Retaining Wall Design Guide or Steel Sheet Piling Design Manual. The moment along the length of the pile may be evaluated with CWALSHT.

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**

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).

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.

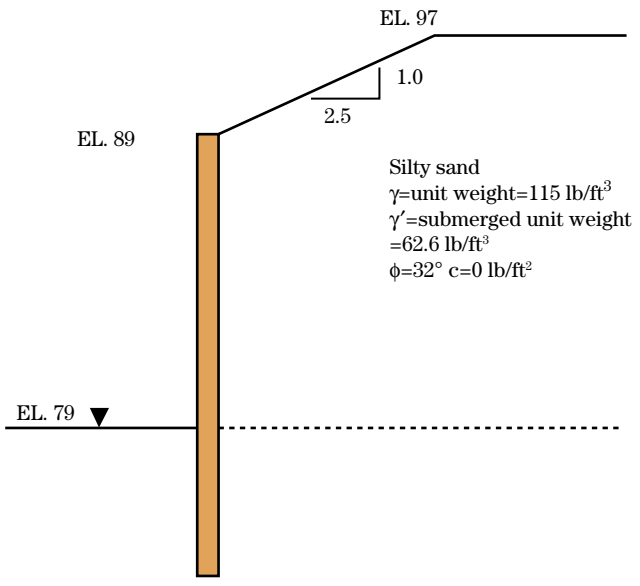
Alignment—Piles should be maintained in alignment during driving. Sheet pile should not be driven more than an eighth inch per foot out of plumb in the plane of the wall or perpendicular to the plane of the wall.

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.



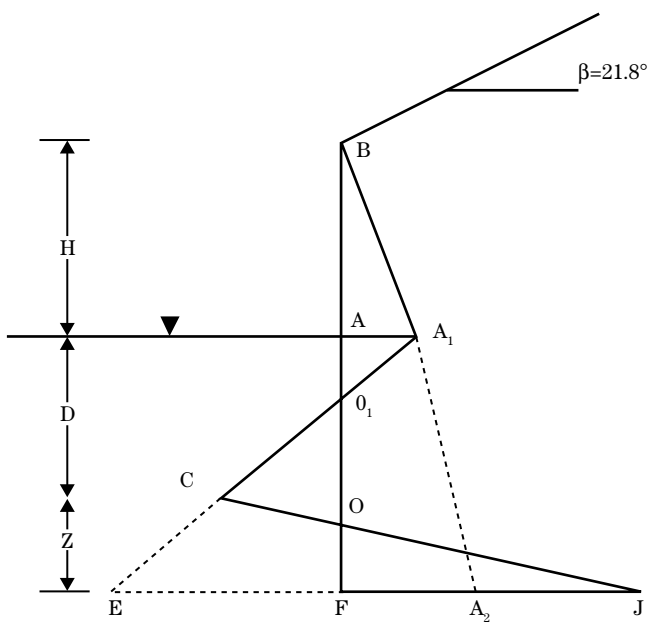
Determine:

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

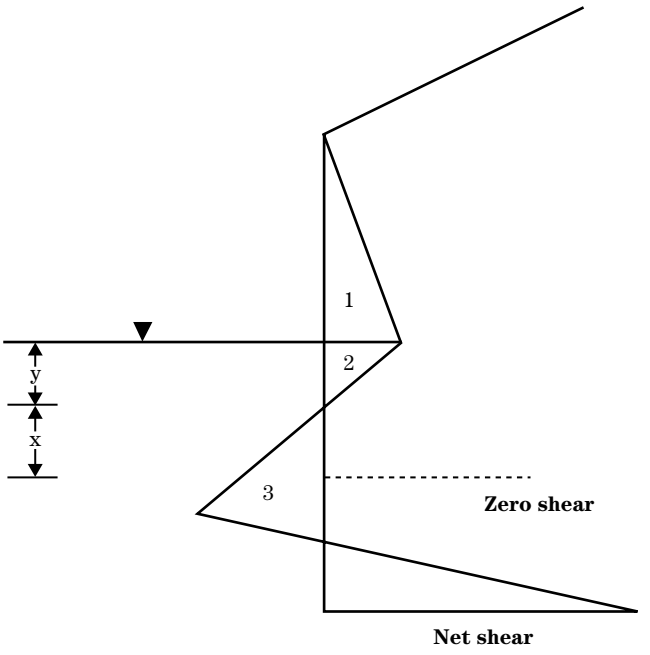
$$Z = 3.68 \quad \text{(eq. TS14R–32)}$$

$$\sum M_F = 164 \text{ ft-lb} \quad \text{(eq. TS14R–33)}$$

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

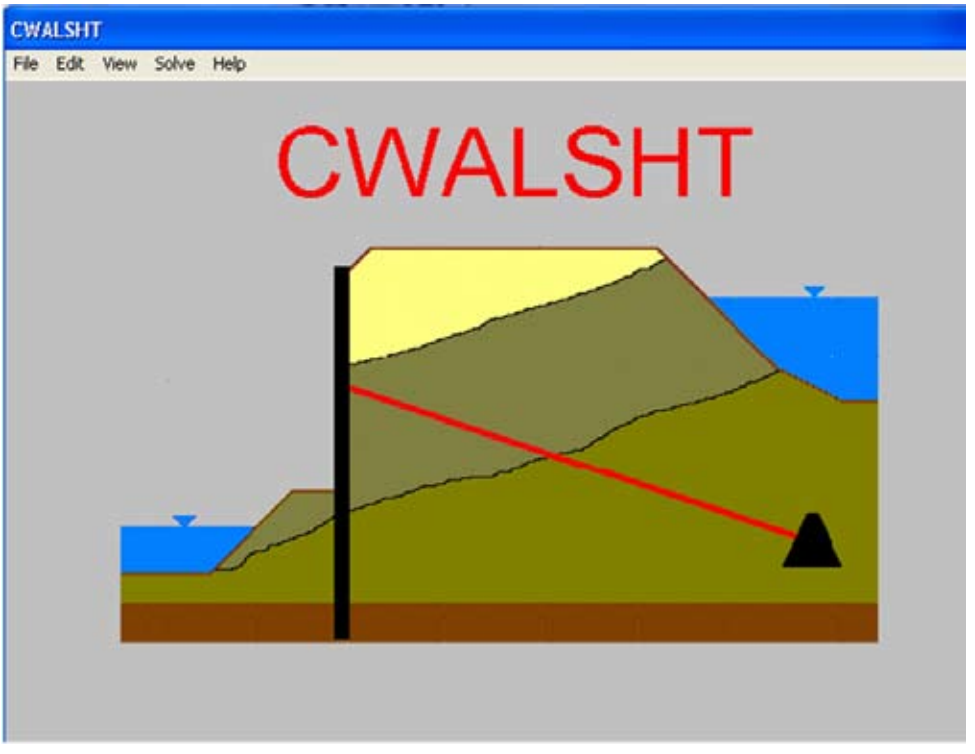
Maximum moment and shear in the pile

Minimum recommended steel sheet pile wall properties

Estimate the deflection of the top of the pile

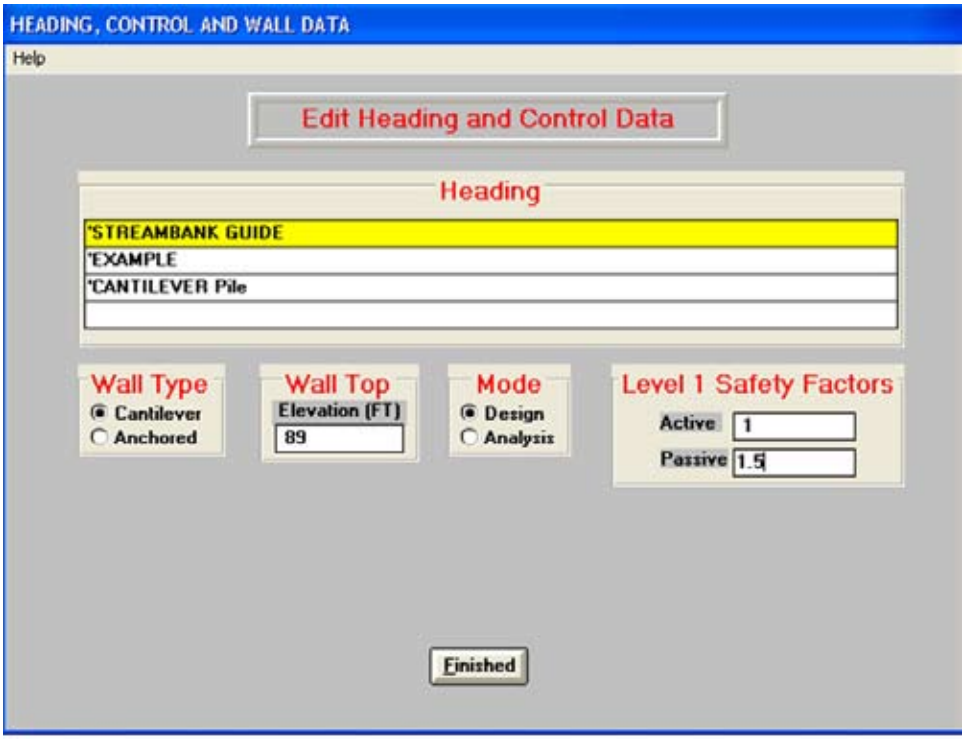
Solution:

Using CWALSHT, enter the data for the analysis.

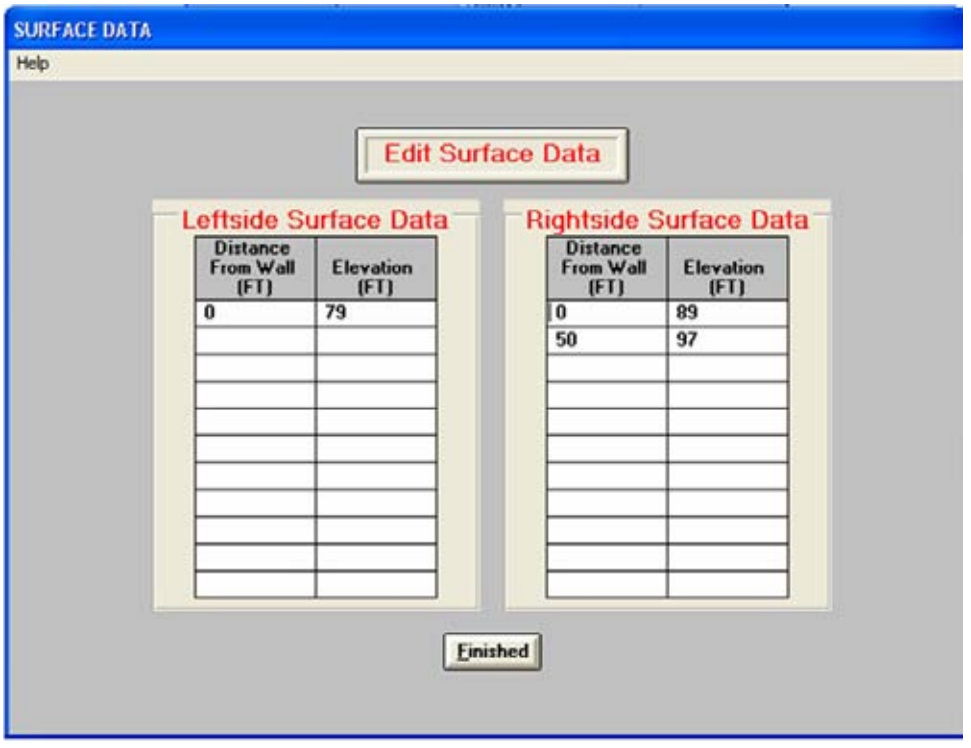


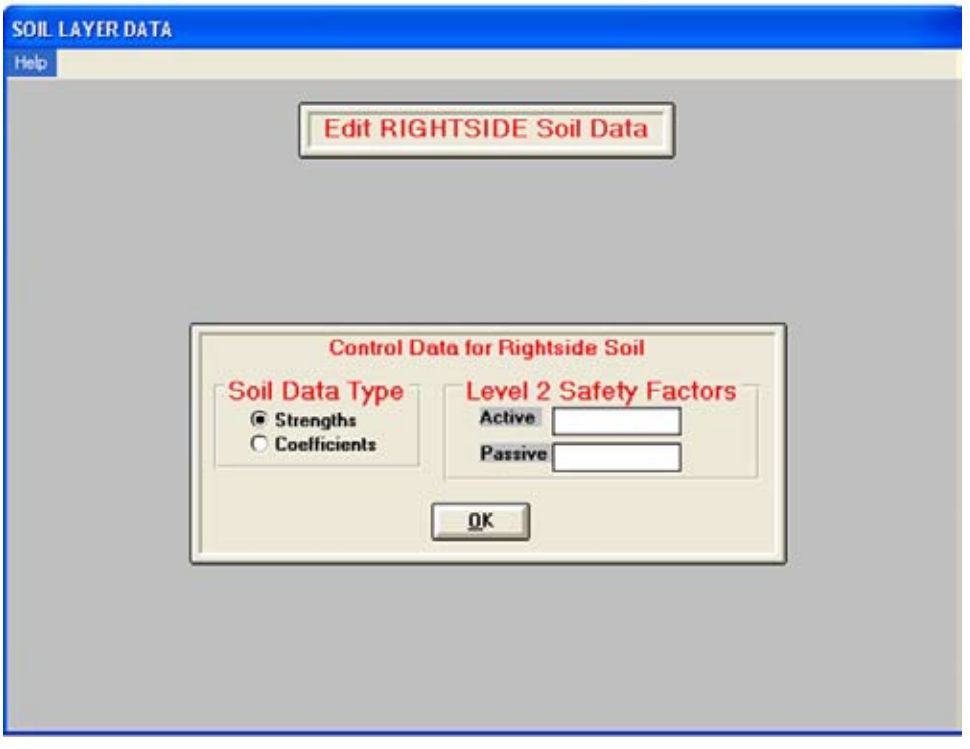
A factor of safety of 1.5 on the passive soil pressure is recommended for this usual load condition. Since the soils are silty sand with a moderate permeability, they will exhibit drained (long-term, effective) behavior.

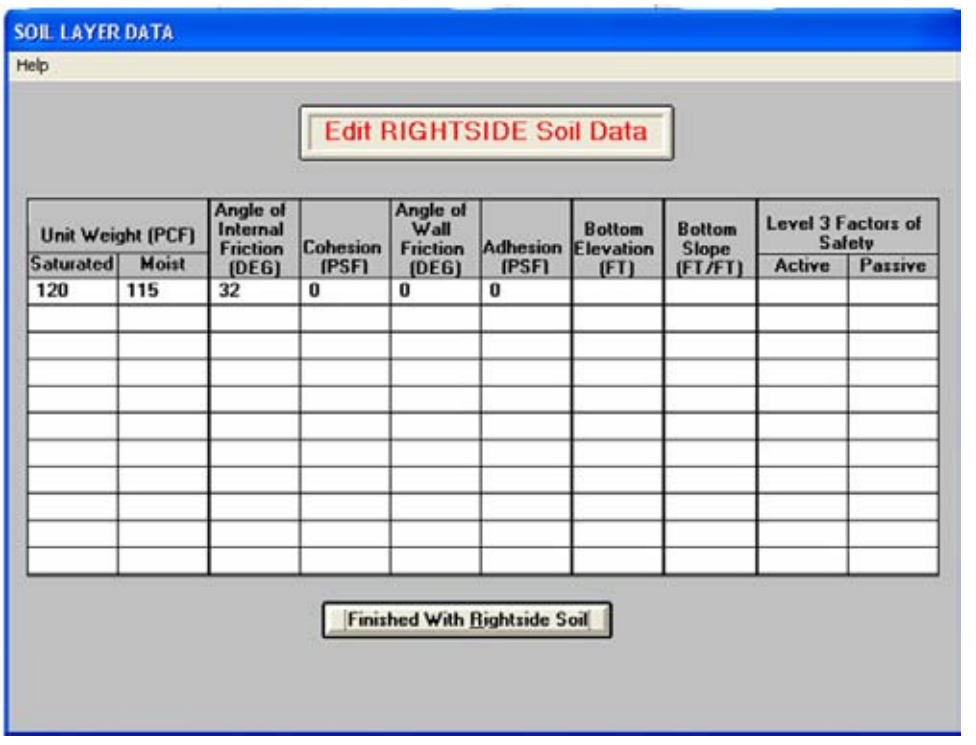
Enter the surface data for both the left side and right side. CWALSHT requires the pile to be loaded from the right side.



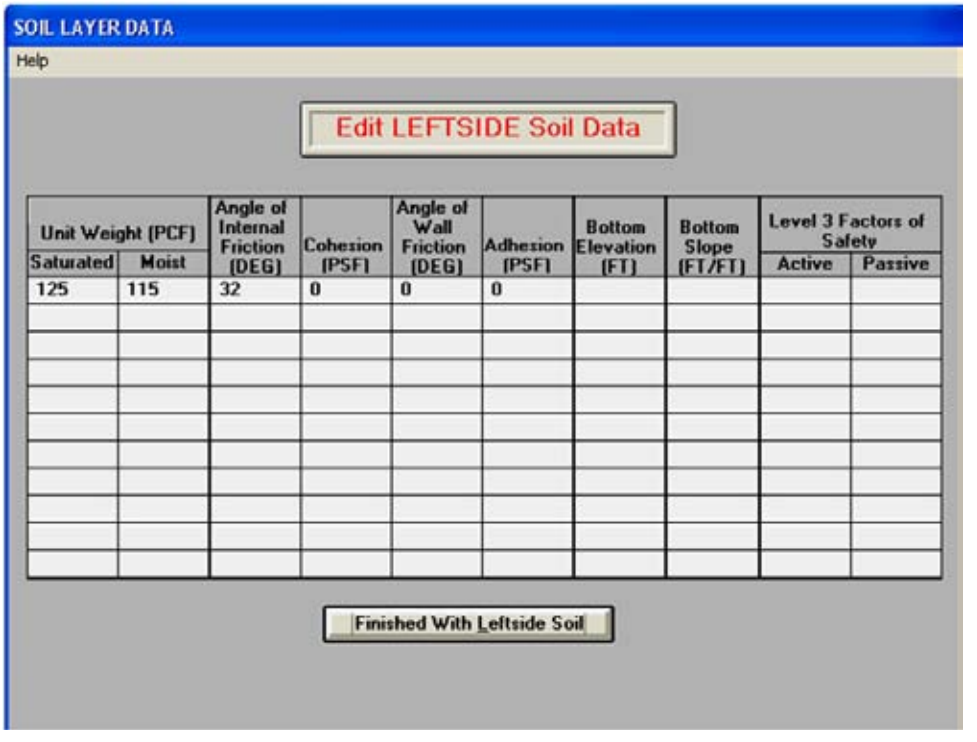
Enter the soils data for both the left and right side. The level 1 factors of safety input previously will apply to the soils data unless level 2 factors are input to override level 1.





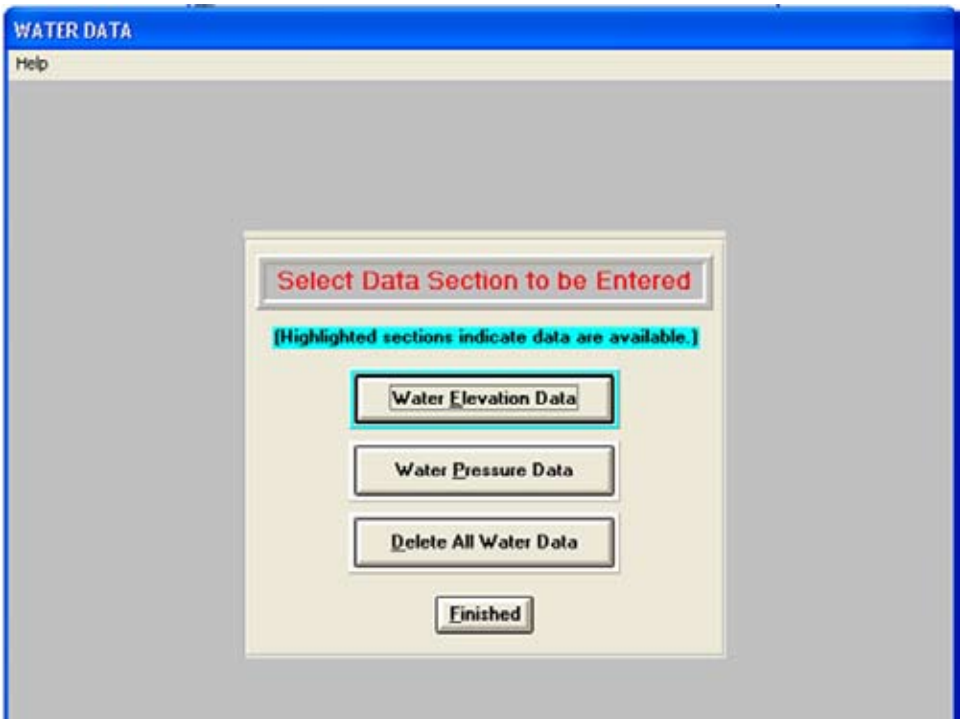


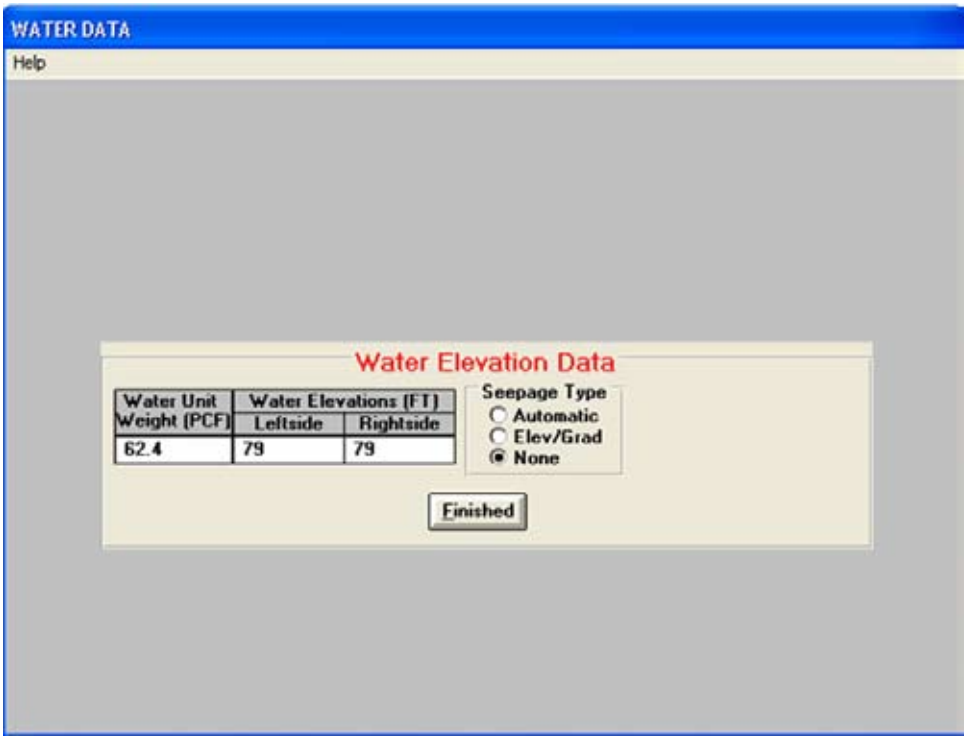
Unit Weight (PCF)		Angle of Internal Friction (DEG)	Cohesion (PSF)	Angle of Wall Friction (DEG)	Adhesion (PSF)	Bottom Elevation (FT)	Bottom Slope (FT/FT)	Level 3 Factors of Safety	
Saturated	Moist							Active	Passive
120	115	32	0	0	0				



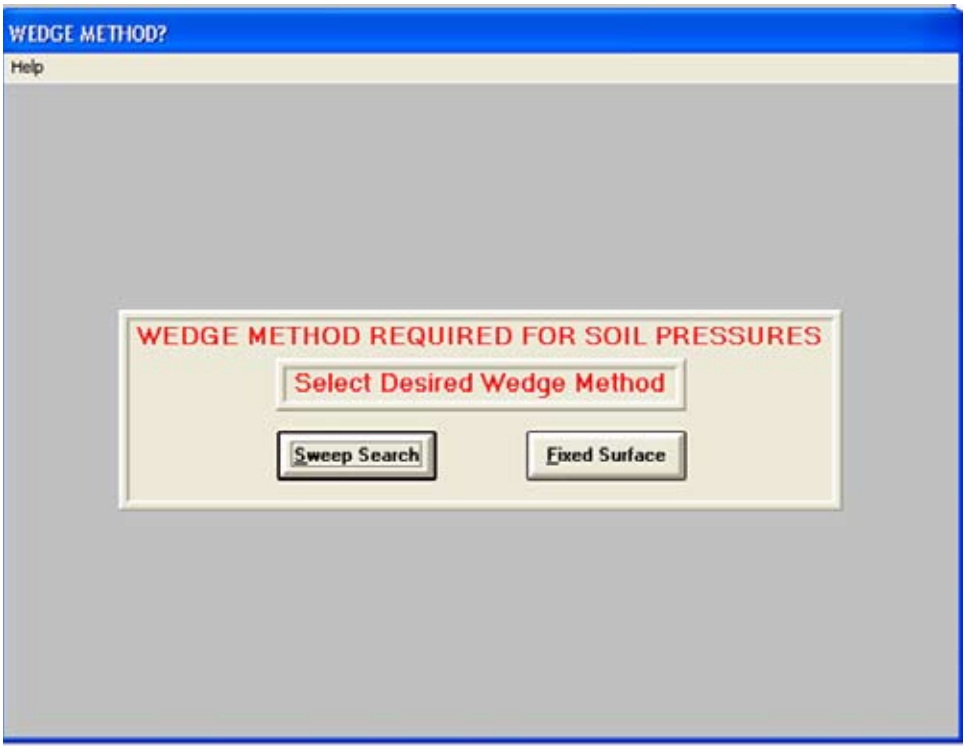
Unit Weight (PCF)		Angle of Internal Friction (DEG)	Cohesion (PSF)	Angle of Wall Friction (DEG)	Adhesion (PSF)	Bottom Elevation (FT)	Bottom Slope (FT/FT)	Level 3 Factors of Safety	
Saturated	Moist							Active	Passive
125	115	32	0	0	0				

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).

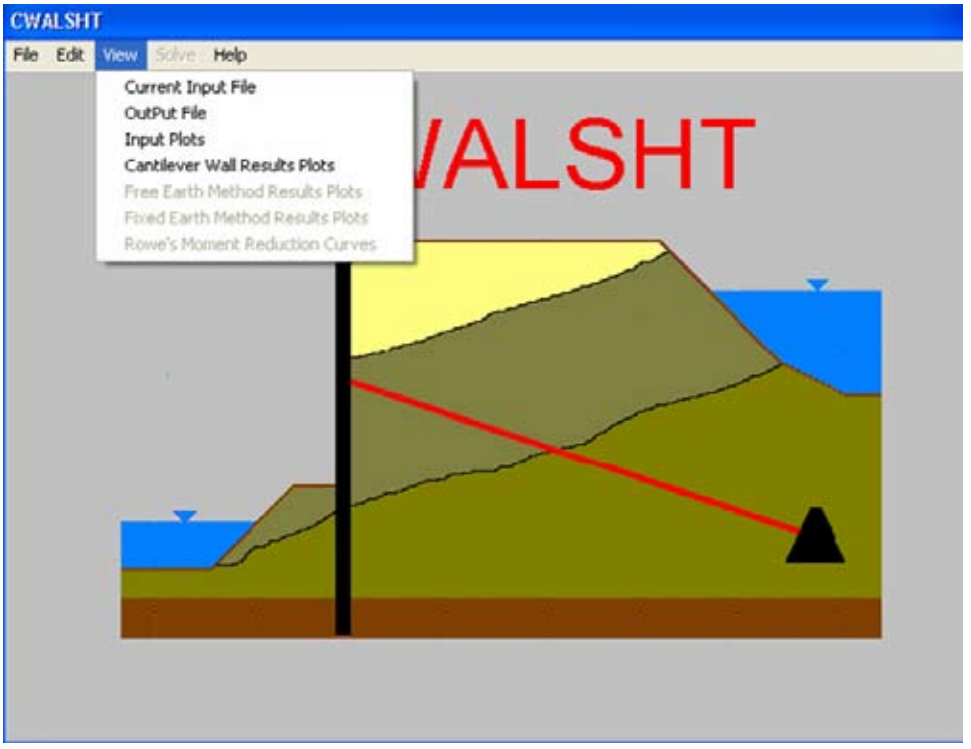


Figure TS14R–17 CWALSHT output file

```
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

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SCALED SHEAR (LB)	NET DEFLECTION (LB-IN^3)	PRESSURE (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.

$$\begin{aligned} \text{Deflection} &= \frac{\text{Scaled deflection}}{(E \times I)} \\ &= 5.1147E + \frac{9}{(29{,}000{,}000 \times 53.63)} \\ &= 3.28 \text{ in} \end{aligned} \qquad \text{(eq. TS14R–57)}$$

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.

Contents

Purpose	**TS14S–1**
Introduction	**TS14S–1**
Flood plain function	**TS14S–1**
The need for setbacks	**TS14S–1**
Calculating the streamway width	**TS14S–1**
Conclusion	**TS14S–3**

Tables

Table TS14S–1 Recommended width for vegetated riparian zones TS14S–2

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.

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 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

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).

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.

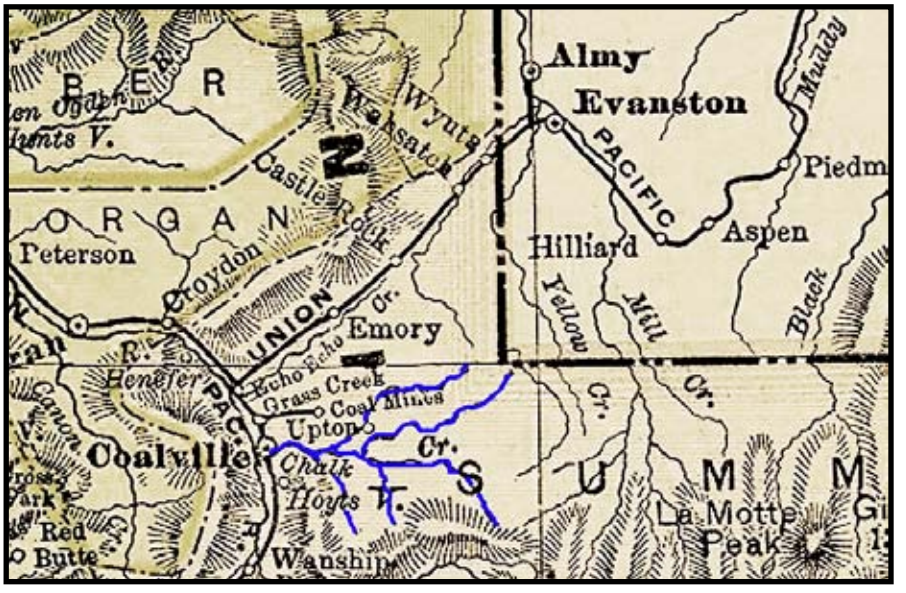
For the setback to accomplish the goal of the impacted stream to sustain dynamic-equilibrium also requires the incorporation of:

- 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

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.

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.


Almy
Evanston
Muddy
PACIFIC
Piedm
Aspen
Hilliard
Yellow
Mill
Black
Cr.
Cr.
Cr.
Wyuta
Weksatch
Castle Rock
ORGAN
Peterson
Croydon
UNION
Cr.
Emory
Henefer
Echo
Grass Creek
Coal Mines
Upton
Coalville
Chalk
Cr.
Hoyts
Wanship
Red
Butte
La Motte
Peak

Issued August 2007

Cover photo: Stream restoration practices were used to prevent streambank erosion and improve fish habitat in this stream.

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.

By Shane Green, Area Range/Riparian Specialist, U.S. Department of Agriculture, Natural Resources Conservation Service, Utah

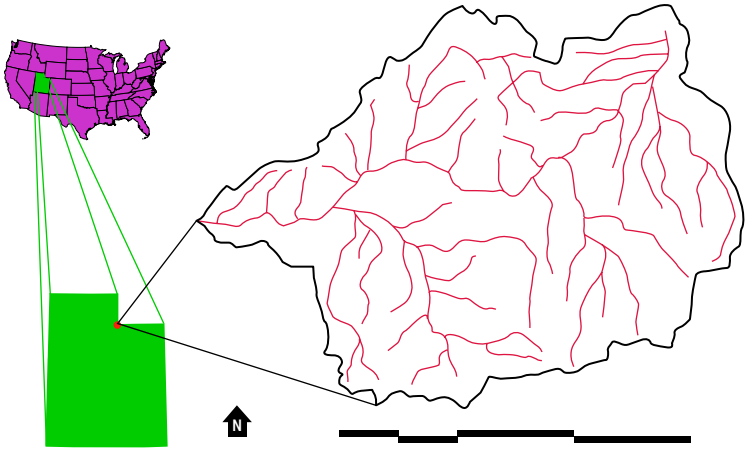
The Chalk Creek Watershed (fig. CS1–1) encompasses 172,000 acres in Summit County, Utah, 95 percent of which is rangeland. The Chalk Creek Nonpoint Source Water Quality Project began in 1991, when the Summit Soil Conservation District (SCD) organized a local steering committee to provide planning guidance 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.

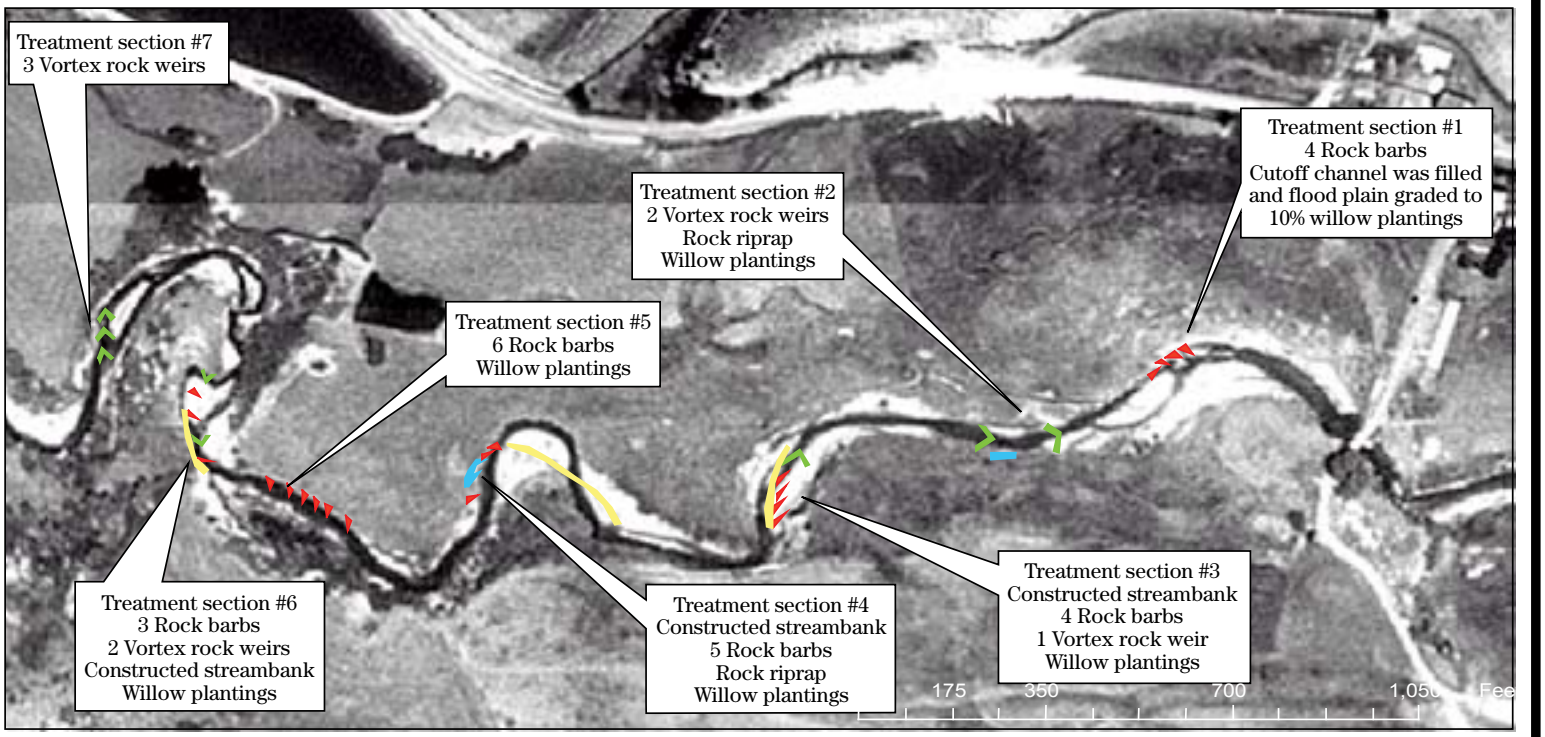
Chalk Creek Case Study design data

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



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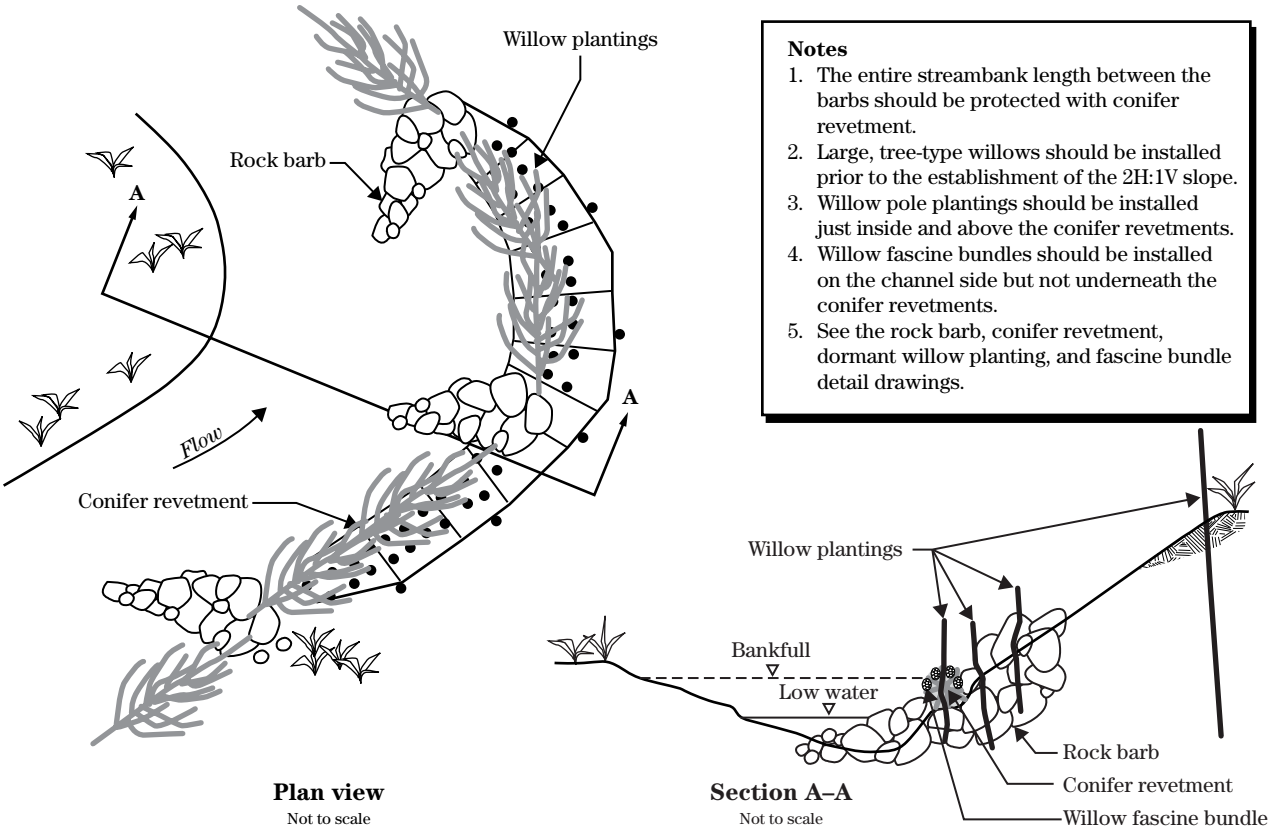
Treatment section #1 - the actively eroding bank was protected with a combination of rock barbs and willow plantings. A braided channel condition apparent in the photo was corrected by filling the cutoff channel and grading the flood plain to 10%.
Treatment section #2 - vortex rock weirs were installed 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.

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.

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

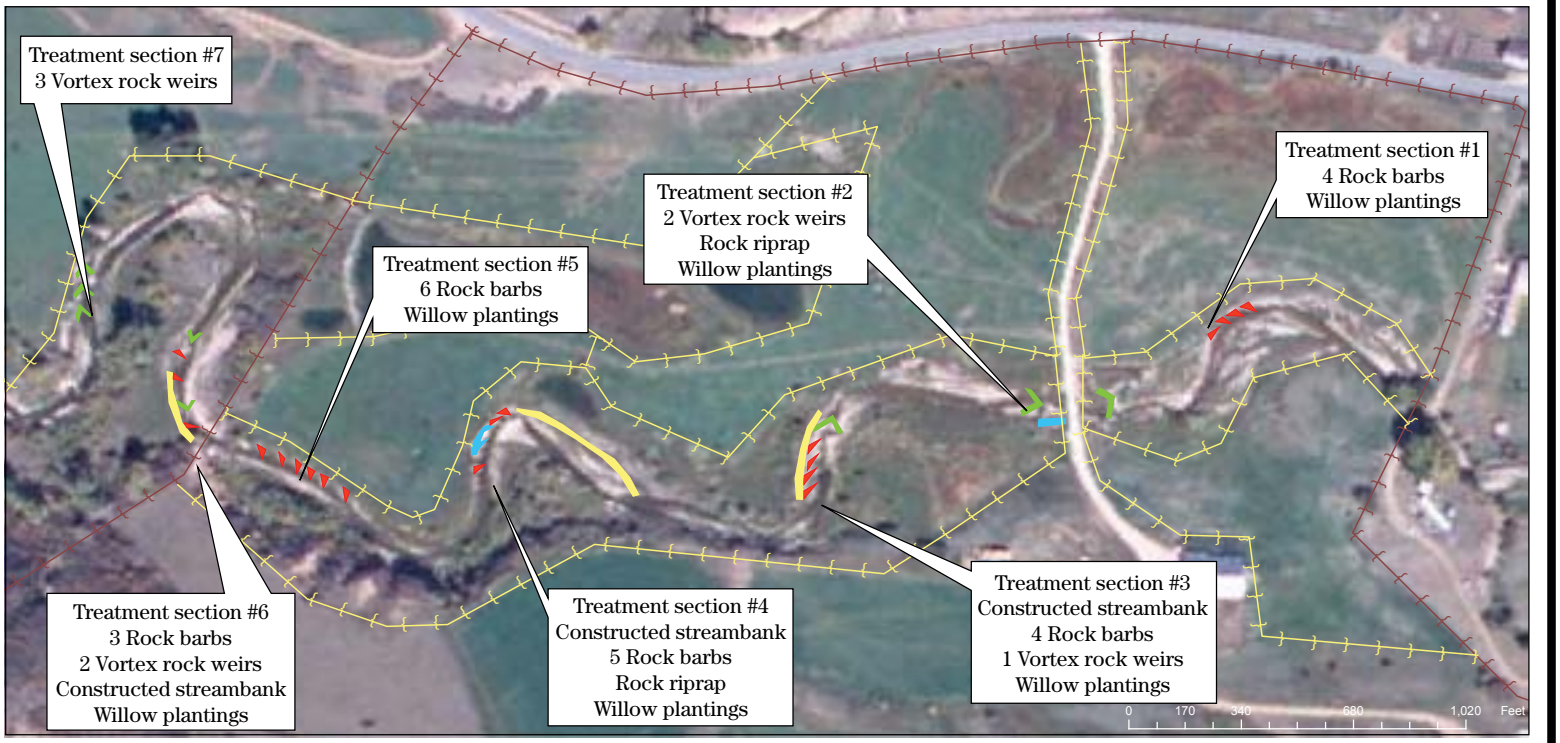
(CS1–6c). Note the function of the rock barbs as the current is focused away from the bank downstream of each barb.

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

Issued August 2007

Cover photo: Panoramic view of the completed Cottonwood Creek, Hutchins, Texas, project

Advisory Note

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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

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

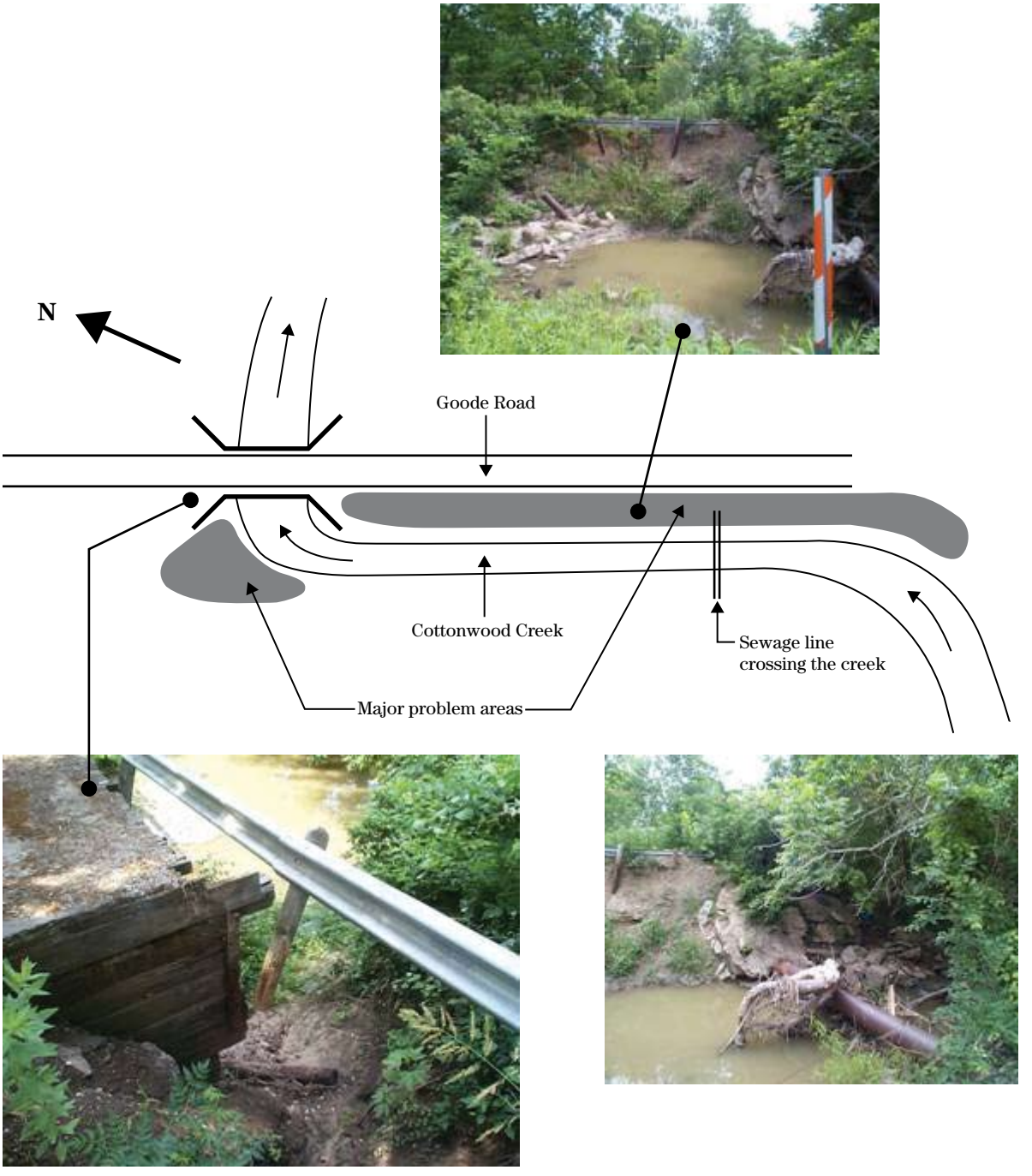
At the early design stage, TxDOT conducted a drainage study to determine the new bridge elevation. The proposed bridge was designed according to the following criteria:

- 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 $kN/m^3 \times 5.3\ m \times 0.003 = 156$ 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.

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

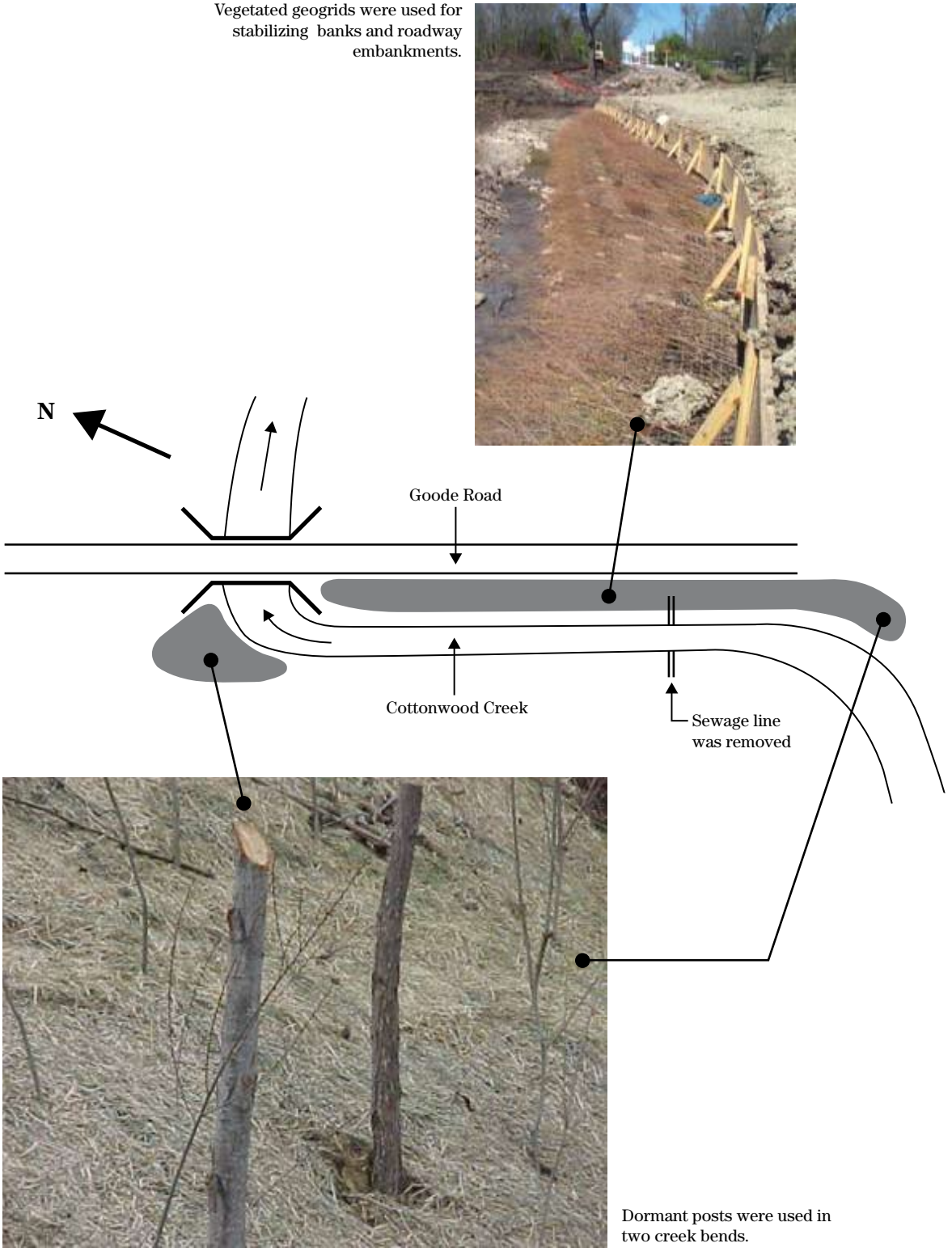
Vegetated geogrids (U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) 1996b) were the primary stabilization measure for the west-facing roadway embankment, which is also the creek bank (fig. CS2–4).

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

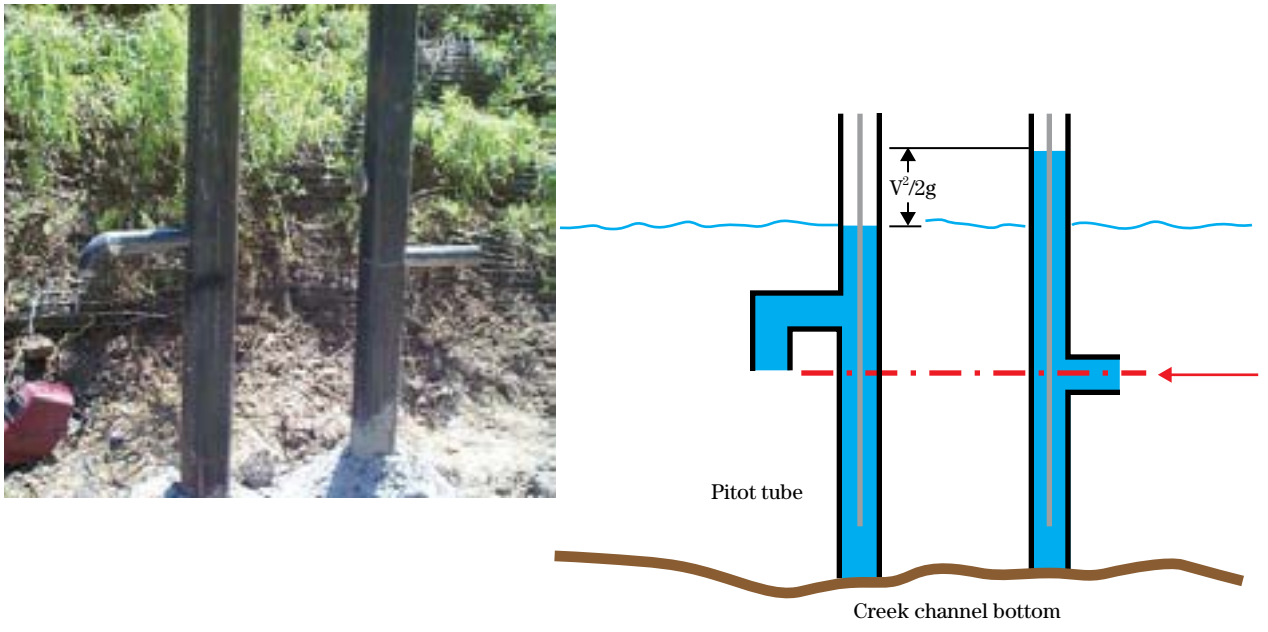
Dormant posts (USDA NRCS 1996b) of black willow (*Salix nigra*), 76 to 127 millimeters in diameter, were installed on two creek bend areas (fig. CS2–4). Boulders of 0.3 meters or larger were keyed into the toe of the bend areas to protect the streambank. In addition, turf reinforcement mats were installed along with the dormant posts to reduce the surface erosion.

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

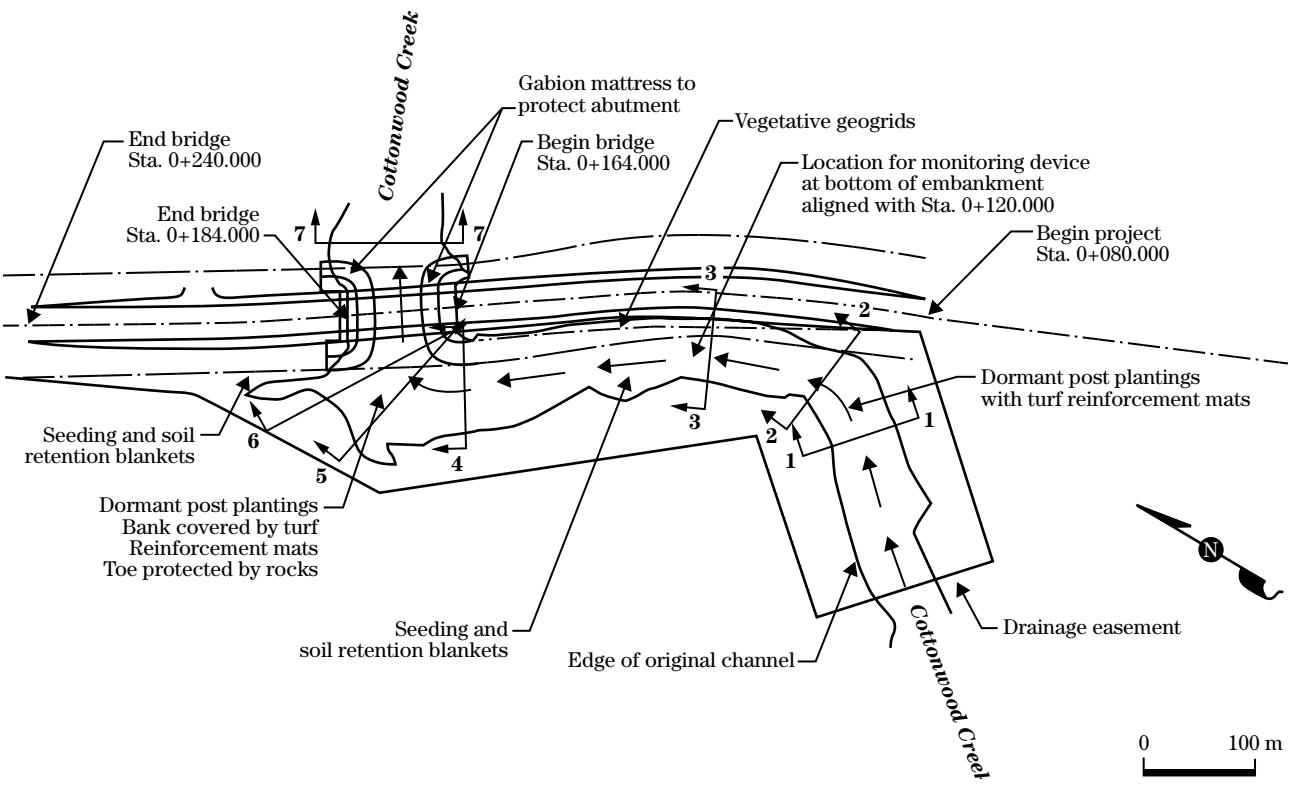


Figure CS2–7 (a) Mar. 2001—Goode Road embankment was stabilized with vegetated geogrids. The opposite bank on the left was protected with soil erosion blankets; (b) Apr. 2003—biomass increases with time; (c) May 2002—black willow covered the entire bank 1 year after construction.

(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)

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.

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

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.

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

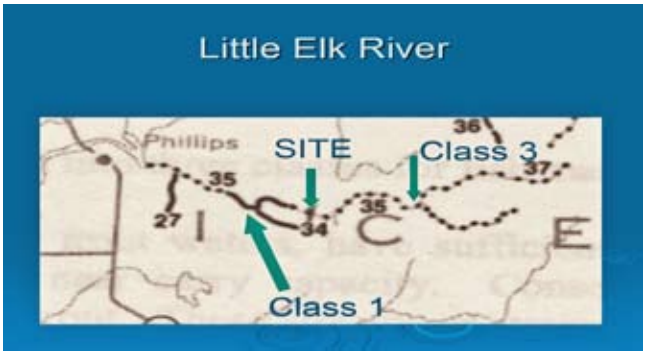
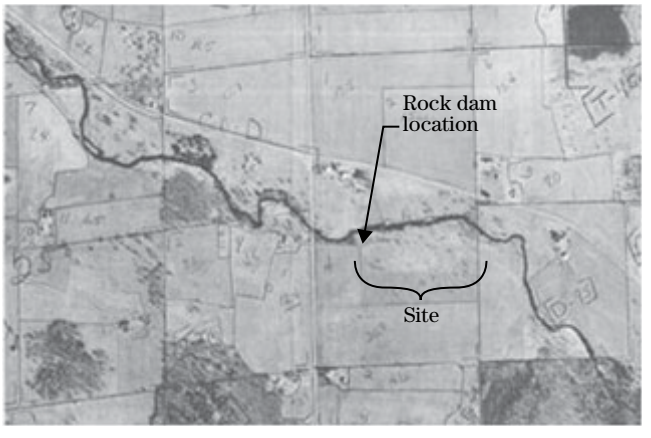


Figure CS3–2 1963 aerial photograph of Little Elk River



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

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

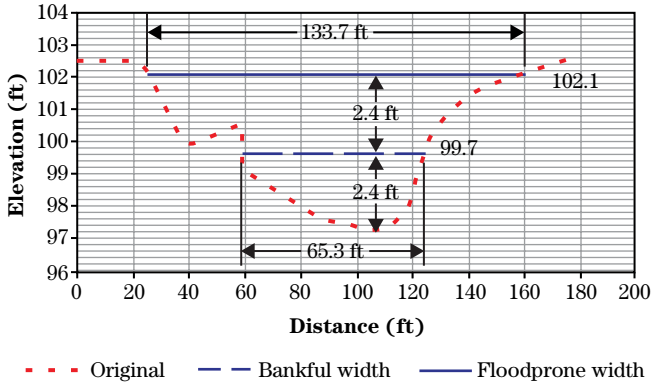


Figure CS3–7 Cross section of undisturbed site upstream

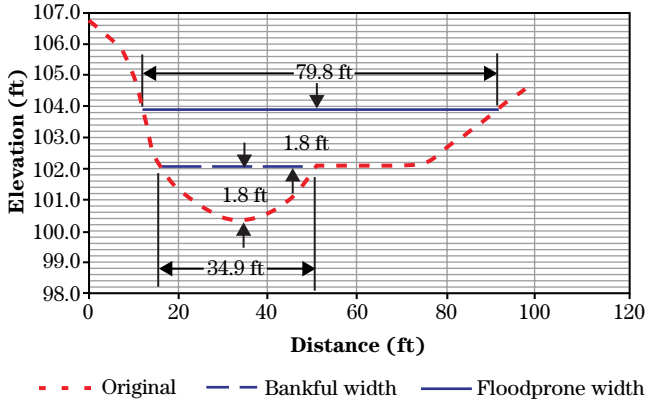


Figure CS3–5 Removal of rock dam crossing, Nov. 2001

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.

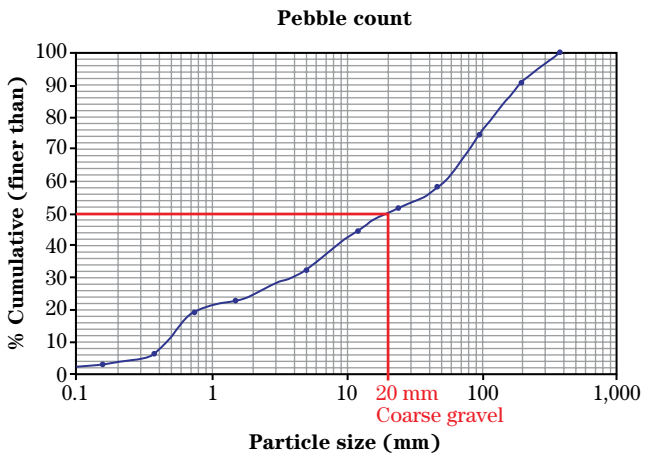
Applied River Morphology (Rosgen 1996) recommends suitable fish habitat improvement structures by stream classification. Channel type B4 is suitable for most structures. NRCS consulted the fish manager from WDNR to concur on the best structures for this site. Random boulder placement was chosen, as well 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

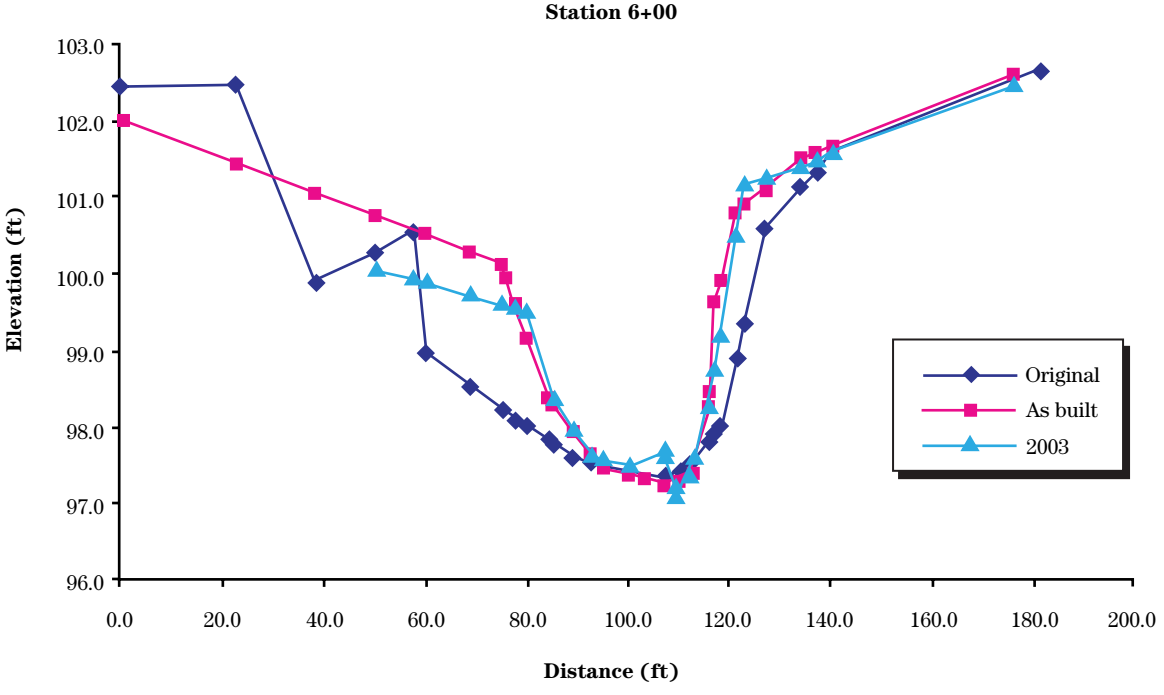
Figure CS3–11 Random boulder placement with DNR fish manager and NRCS engineer

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

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



Issued August 2007

Cover photo: Completed section of Silver Creek, Silver Creek, New York

Advisory Note

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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

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.

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.

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.

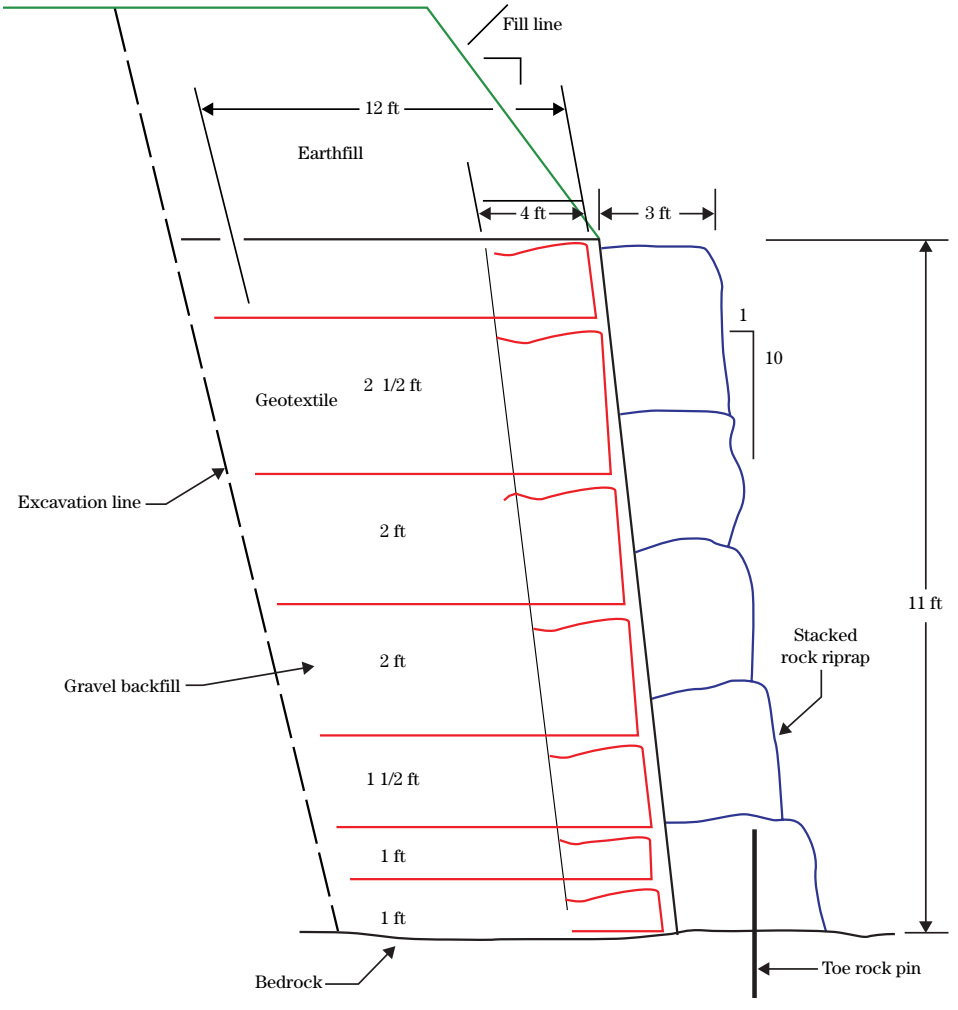
Figure CS4–1 Sheet pile wall

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.

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

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.

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

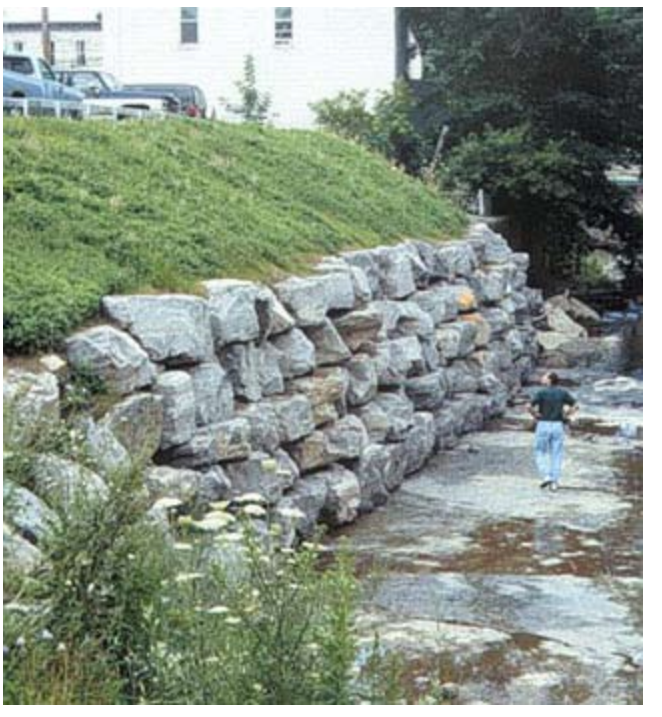
Figure CS4–9 Primary geotextile anchorage length

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.

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.

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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.

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.

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.

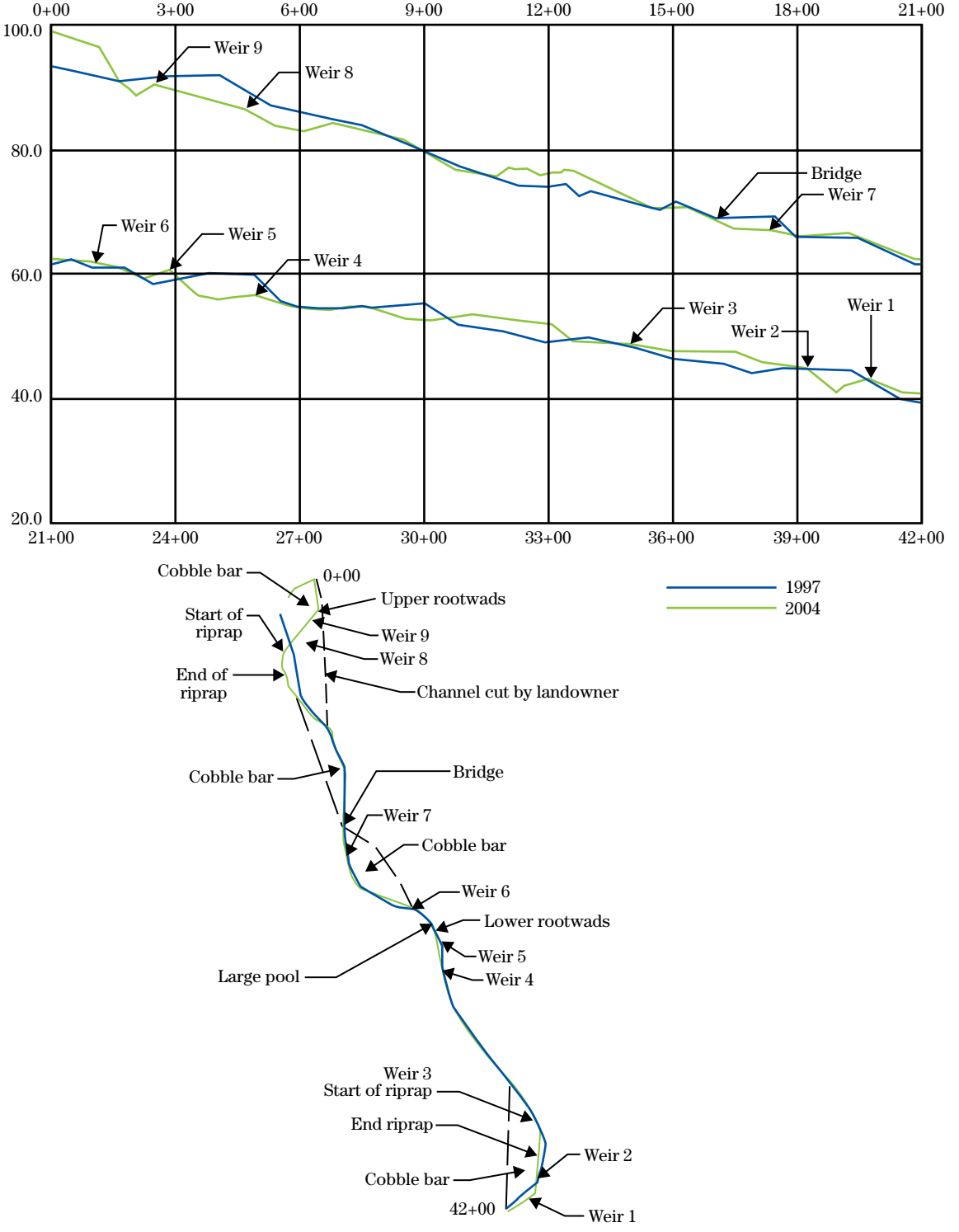
Figure CS5–1 shows complementary plan and profile survey information for Rose River for 1997 and 2004, after the severe storms.

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.

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.

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.

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Cover photo: Cross vane on Big Bear Creek in Pennsylvania

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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.

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

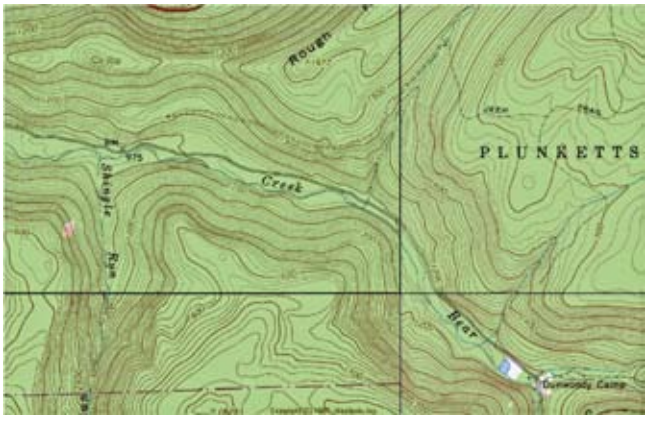
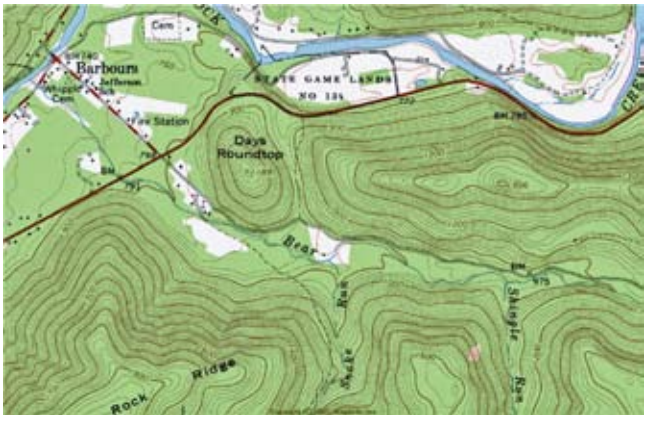


Figure CS6–2 Lower reach of Big Bear Creek restored in phases II and III



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

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.

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.

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.

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 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.

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.

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

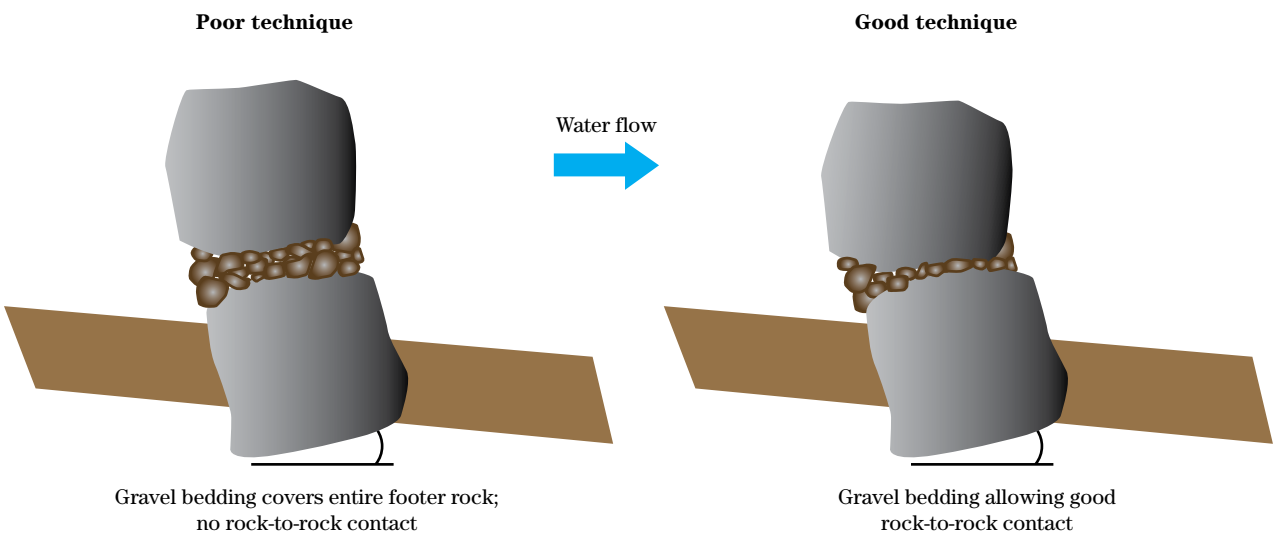
to the excavator using a rubber-tired front end loader (fig. CS6–10).

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

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

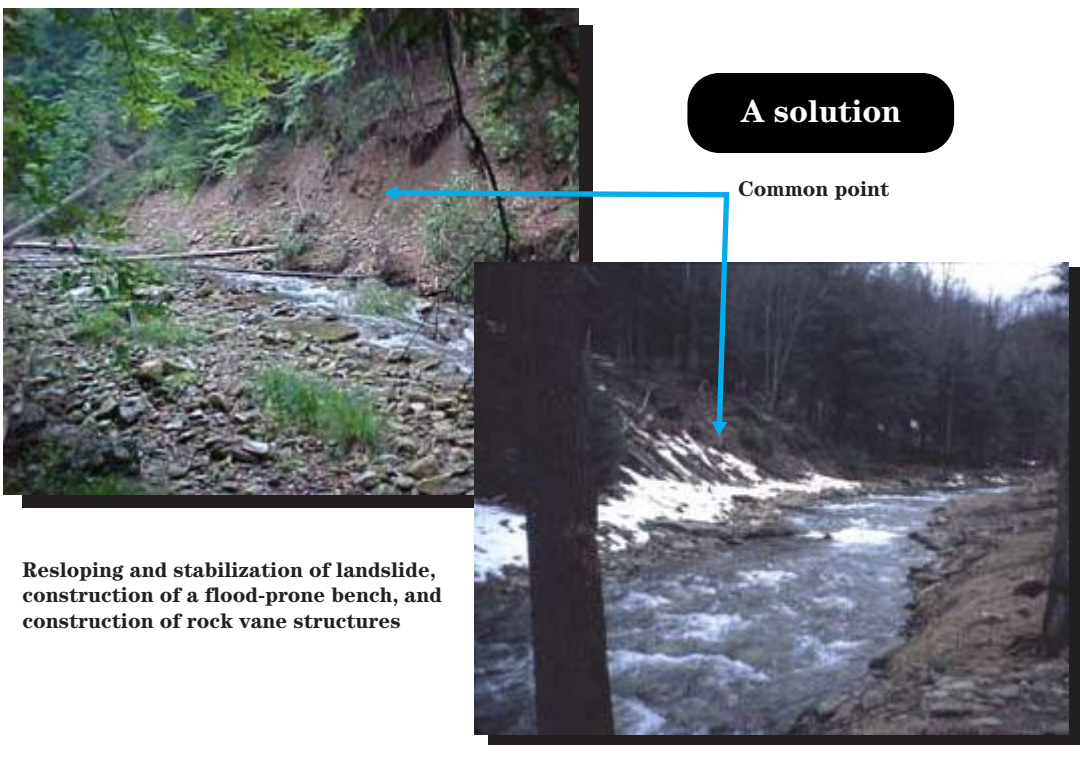


Figure CS6–21 Treatment of a channel avulsion with rock structures

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.

By David Walowsky, civil engineering technician, U.S. Department of Agriculture, Natural Resources Conservation Service, LaFayette, New York

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.

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

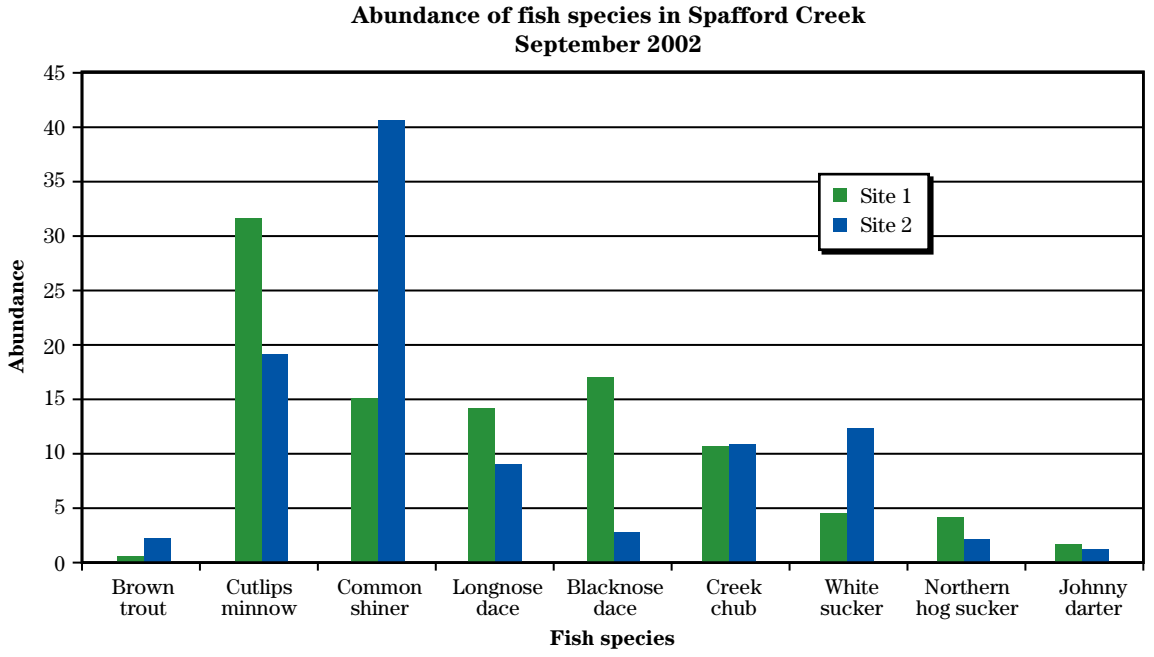
The design storm was the 10-year, 24-hour storm. This estimated discharge is 975 cubic feet per second. The hydrologic investigation of this reach included an in-depth analysis of the flow, the duration and frequency, and the sediment-carrying capacity. One of the most critical characteristics of a stream 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.

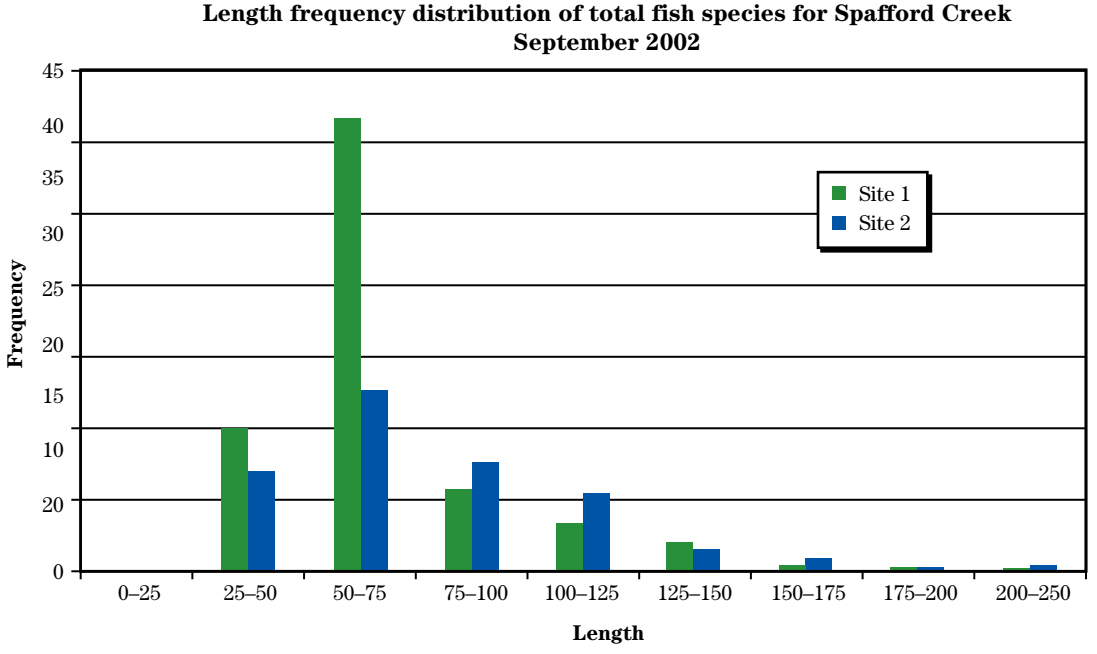
The analysis of the flow included the retrieval of stream gage data from the U.S. Geological Survey (USGS). The stream gage used for this project was located on Spafford Creek at the Sawmill Road Bridge. Baseflow velocity was measured at 1.9 feet per second. Bank pins were set to determine future erosion 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

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

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).

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

Lined waterway
Copper Mine Brook

Issued August 2007

Cover photo: Treated section of Copper Mine Brook, Burlington, Connecticut

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.

By Charles Galgowski, Design/Planning Engineer, P.E., U.S. Department of Agriculture, Natural Resources Conservation Service, Tolland, Connecticut

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.

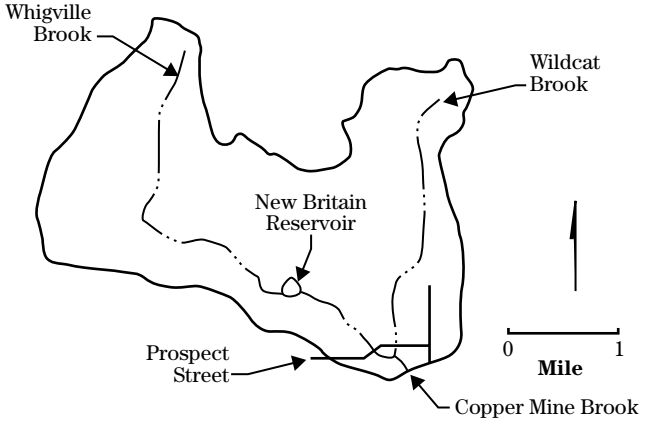
Stakeholders for stream rehabilitation projects in Connecticut desire to address increasing numbers of objectives for stream projects. The Copper Mine Brook EWP project is presented to illustrate what some of these objectives are, what the local landscape looks like, and what design constraints exist. 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.

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).

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.

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

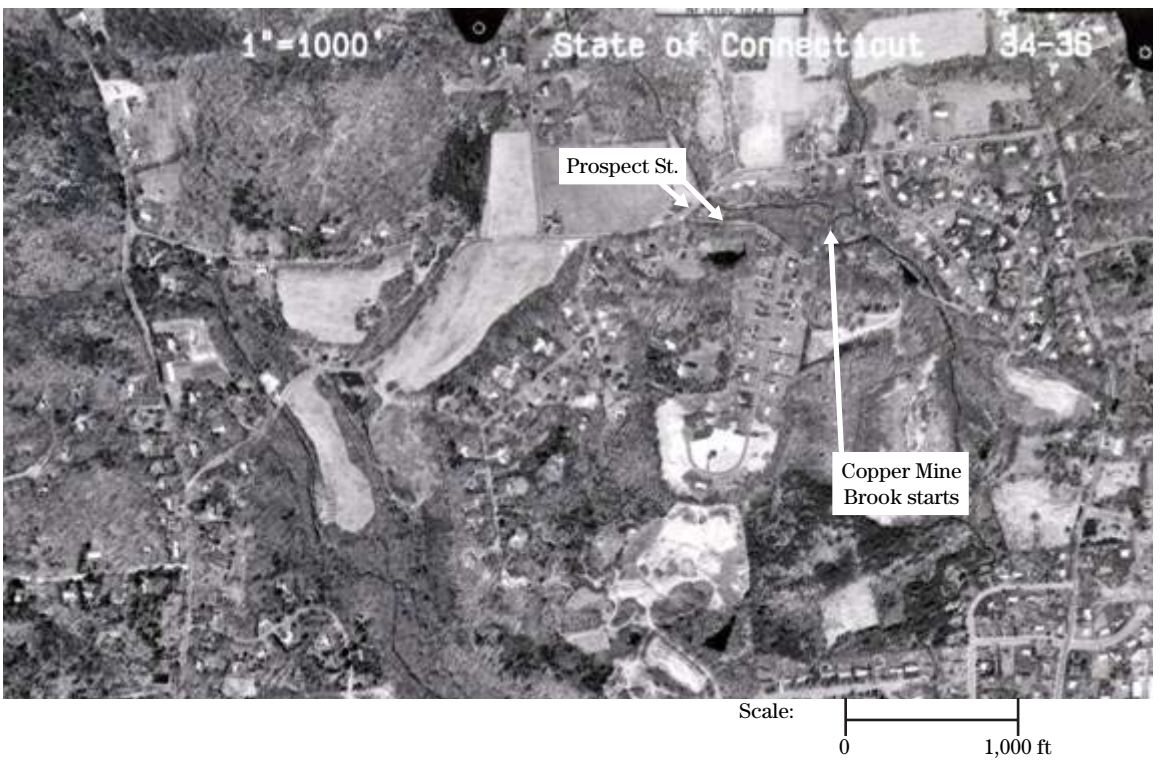


Figure CS8–3 Stream centerline stations, Copper Mine Brook EWP project

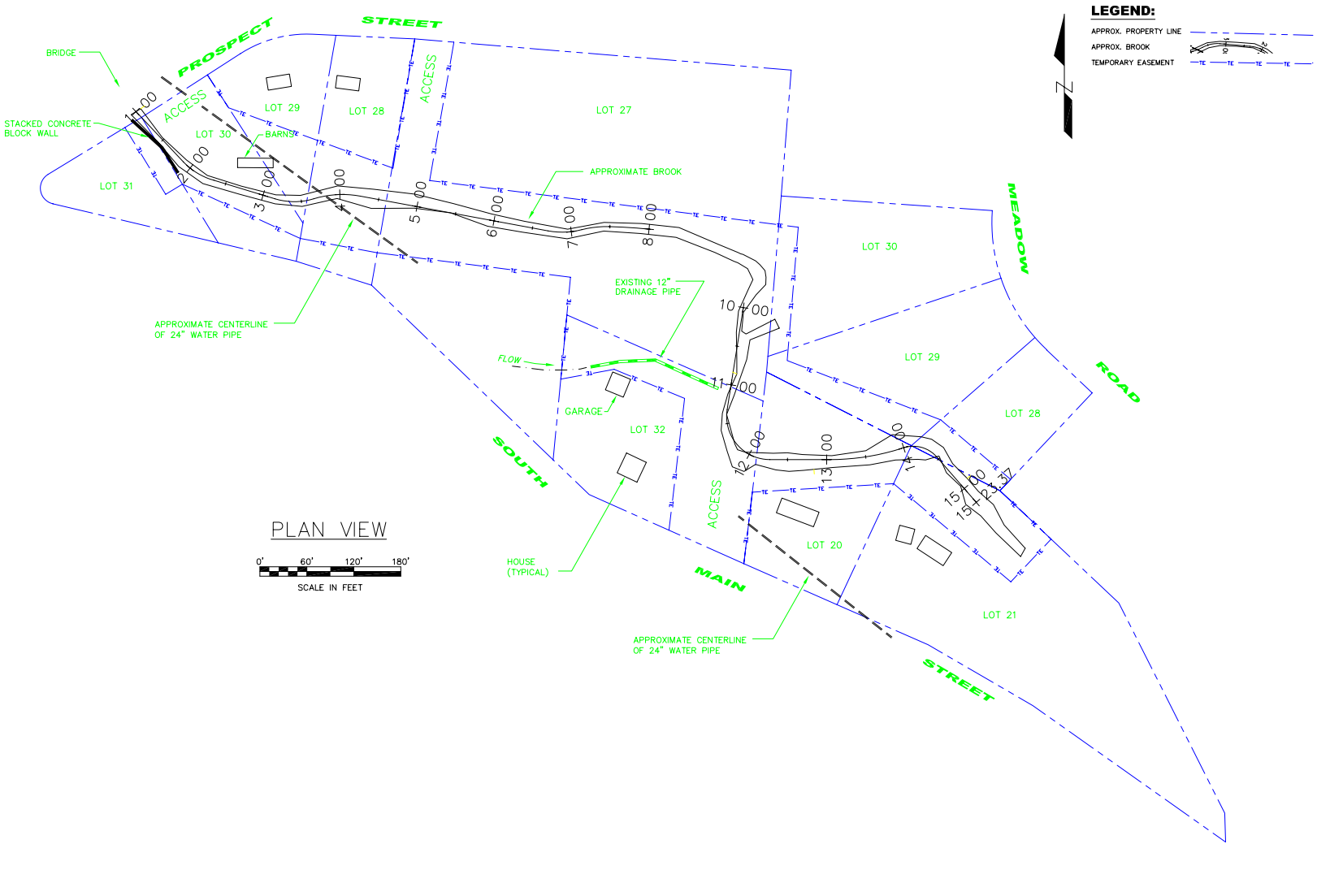


Table CS8–1 Site features on Copper Mine Brook EWP project

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.

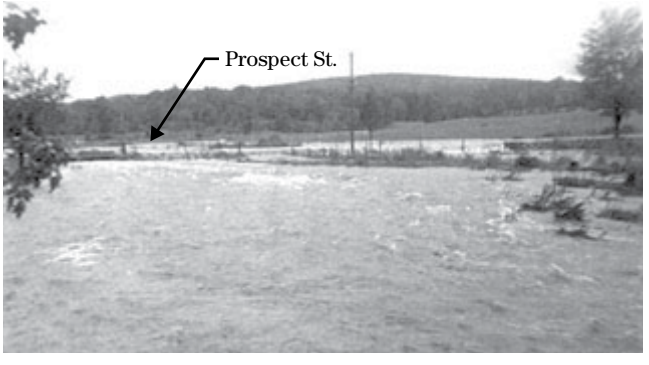
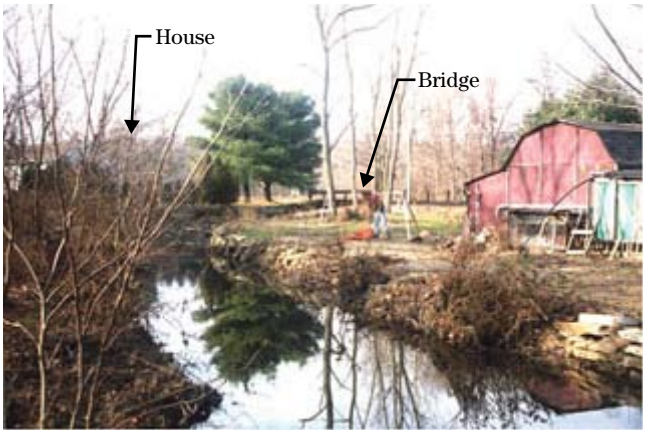


Figure CS8–5 Year 2000 looking upstream at Prospect St.



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.

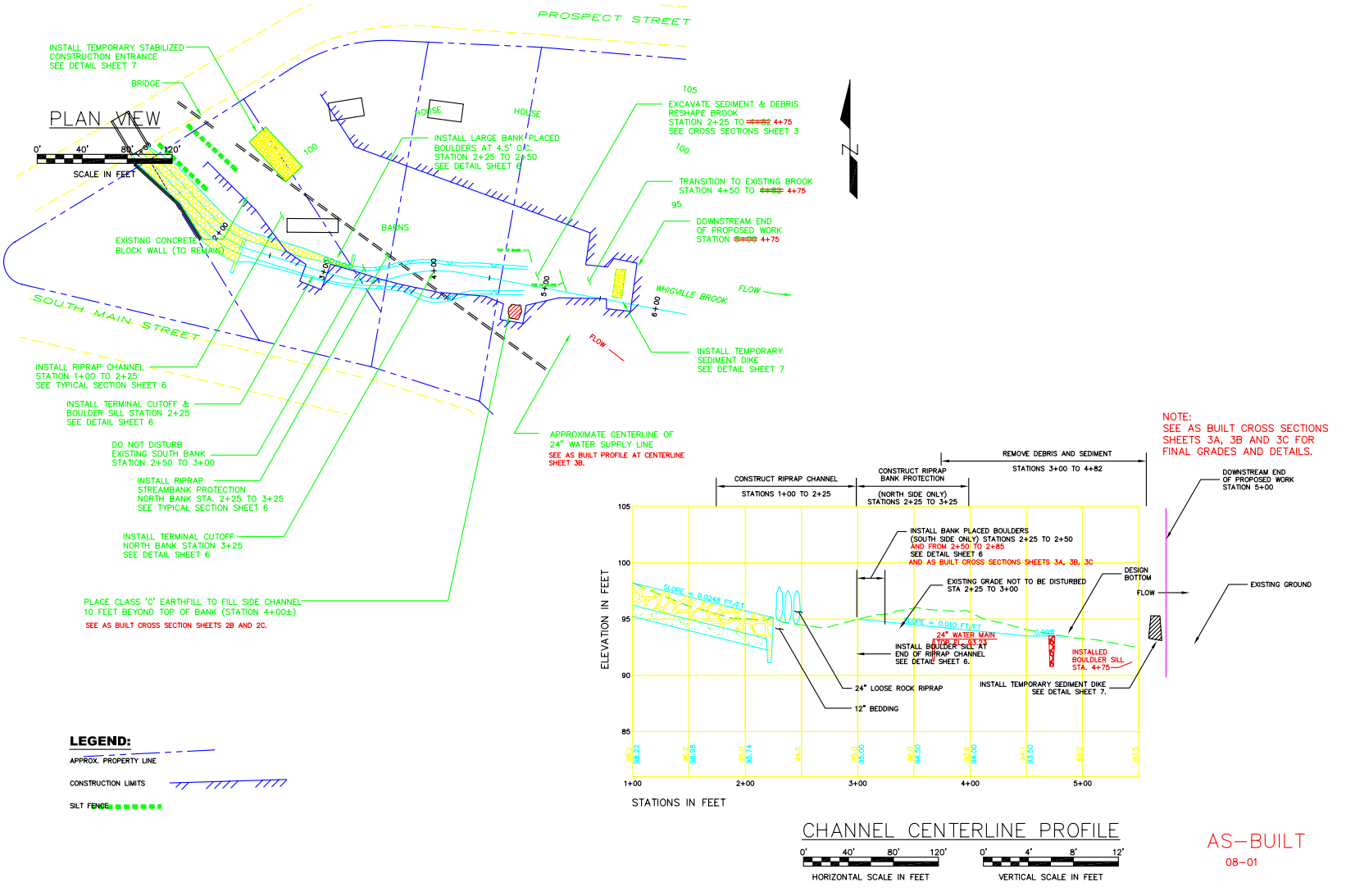


Figure CS8–7 As-built plan view starting 940 feet downstream of Prospect St.

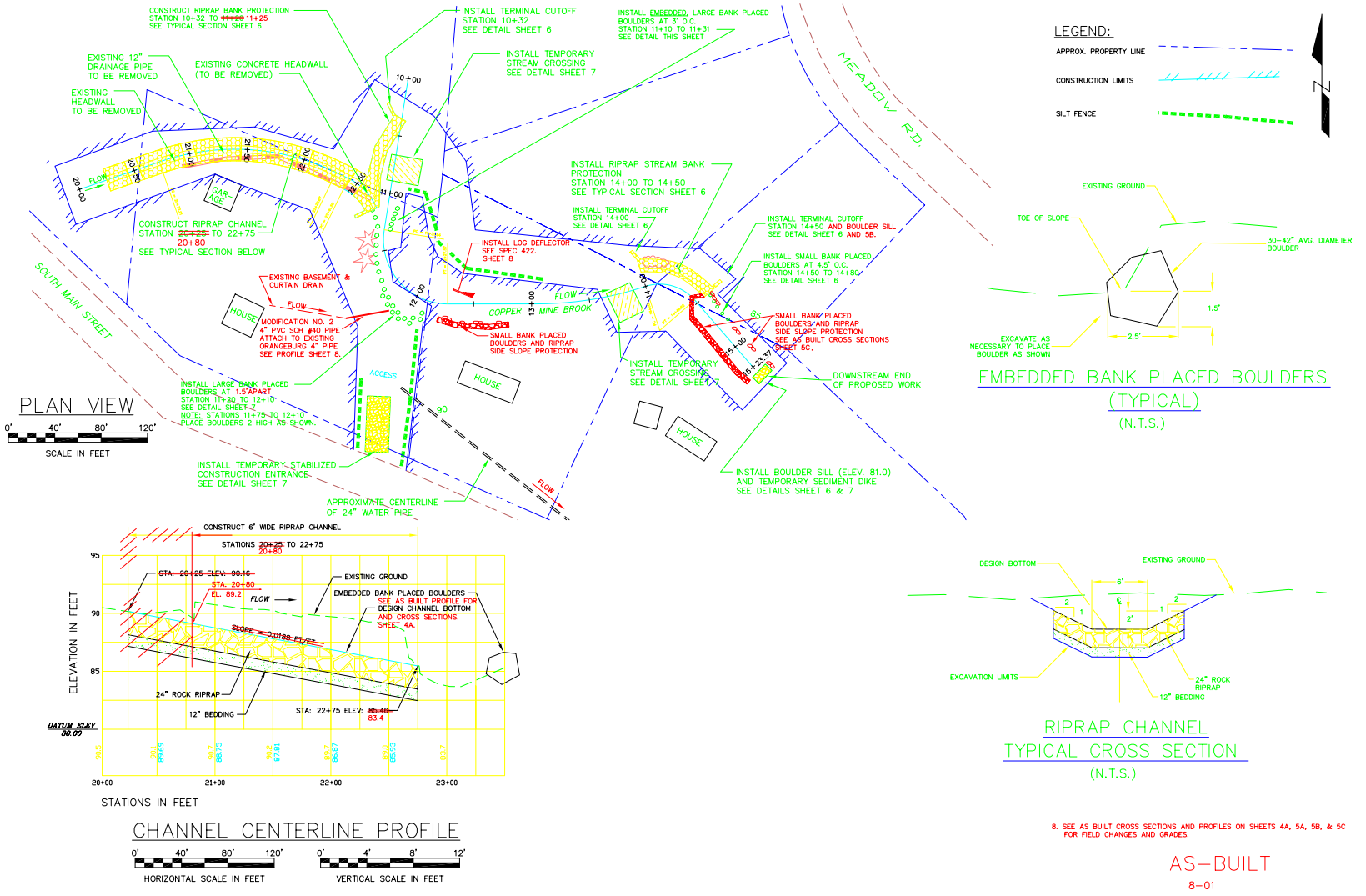


Figure CS8–8 Photo sites orientation aerial map

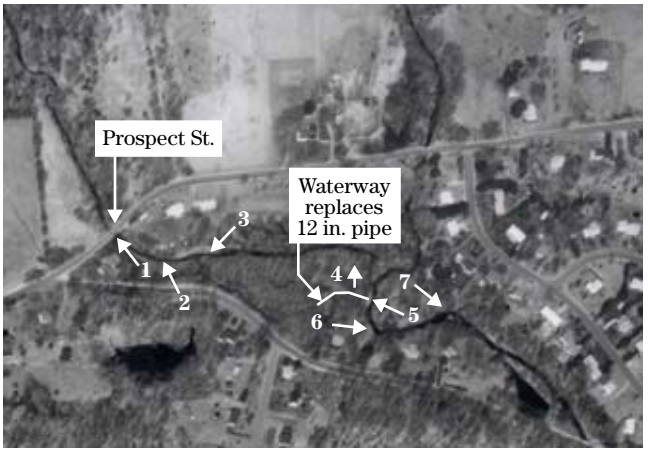


Figure CS8–10 Site 1 before project—looking upstream at Prospect Street Bridge

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.

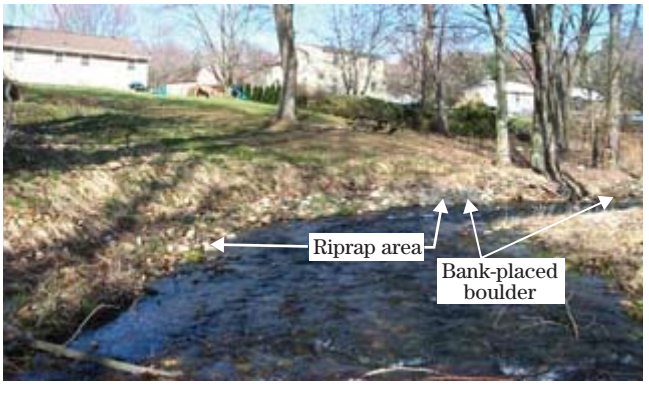
Figure CS8–17 Site 6 after construction—looking downstream at station 12+00

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

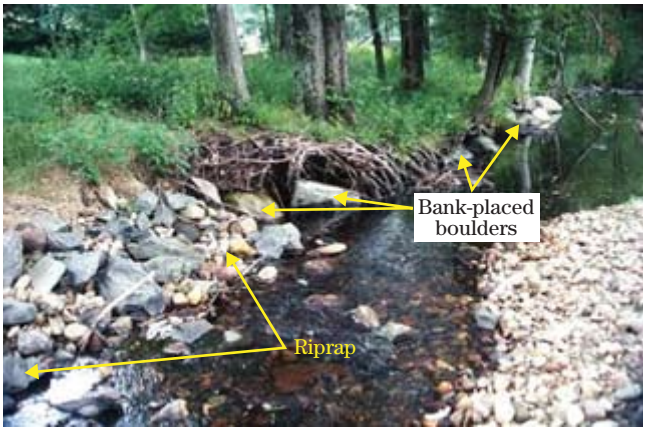
Survey tripod stands on the dewatering berm used during construction. Concerns here were that the brook was encroaching on the left flood plain, eroding into the lawn area and septic system and toppling trees. Area by the tree roots had deep pools and cover for fish, so saving the trees was desirable.

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.

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.

By Philip G. Balch, Stream Restoration Specialist, and Brock Emmert, Stream Specialist, The Watershed Institute, Topeka, Kansas

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 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.

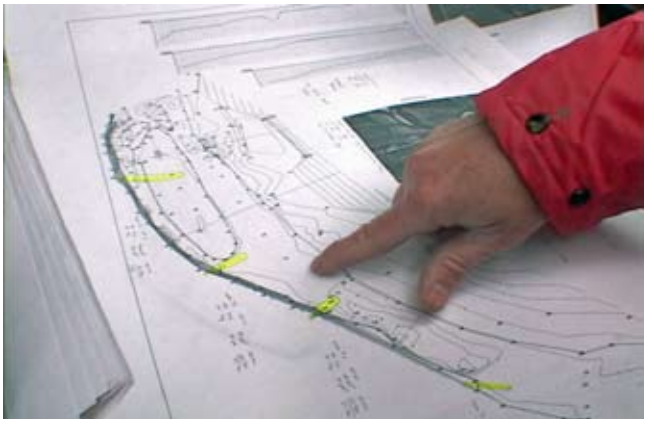
Each site was surveyed by the SCC and NRCS staff with a total station survey instrument. Data points were downloaded into computers, and topographic maps were produced for each site. The maps were then used for measurements and project stabilization design (fig. CS9–3).

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

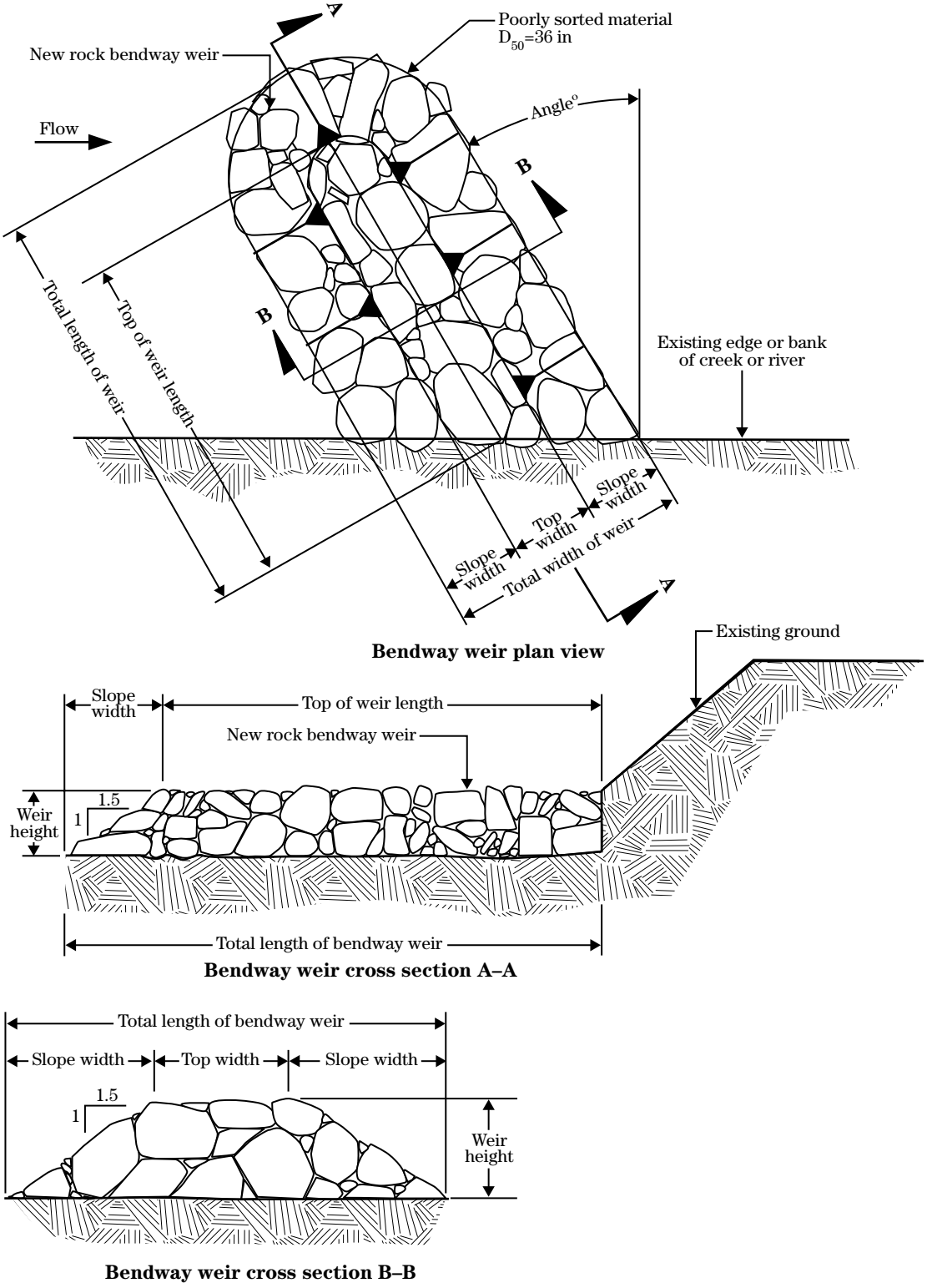


Figure CS9–5 Rock vane design detail

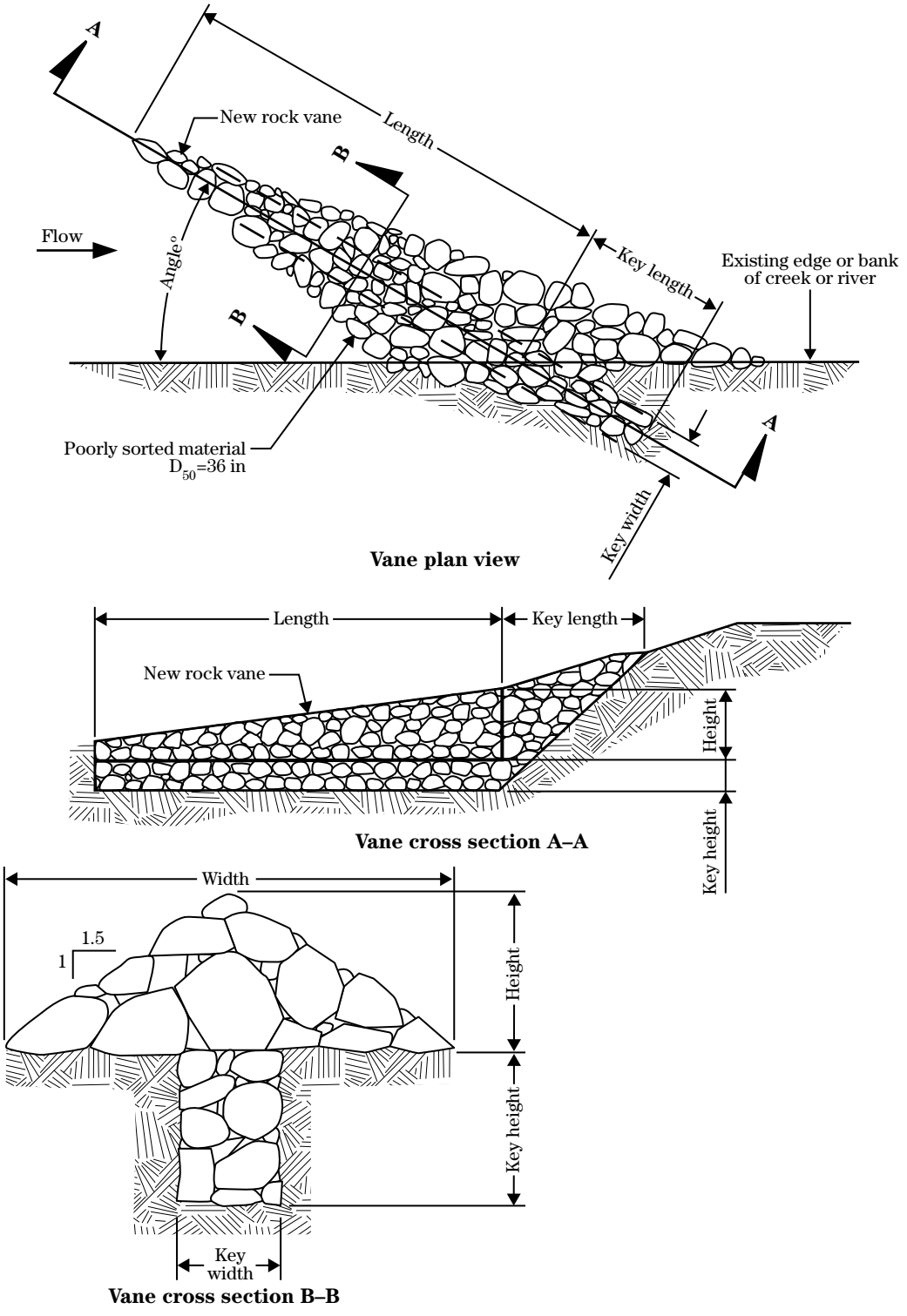


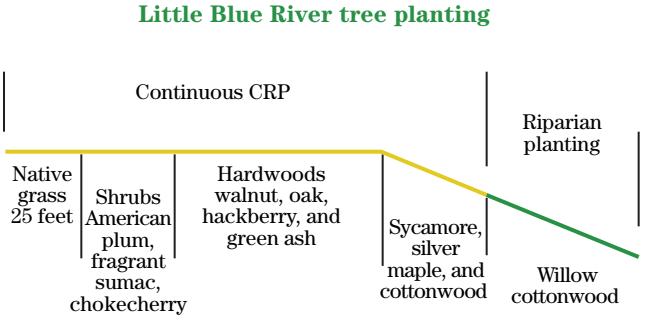
Figure CS9–6 Building weir on site 28

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.

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

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

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

Issued August 2007

Cover photo: Completed section of the Newaukum River, Lewis County, Washington

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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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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.

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.

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

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

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.

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

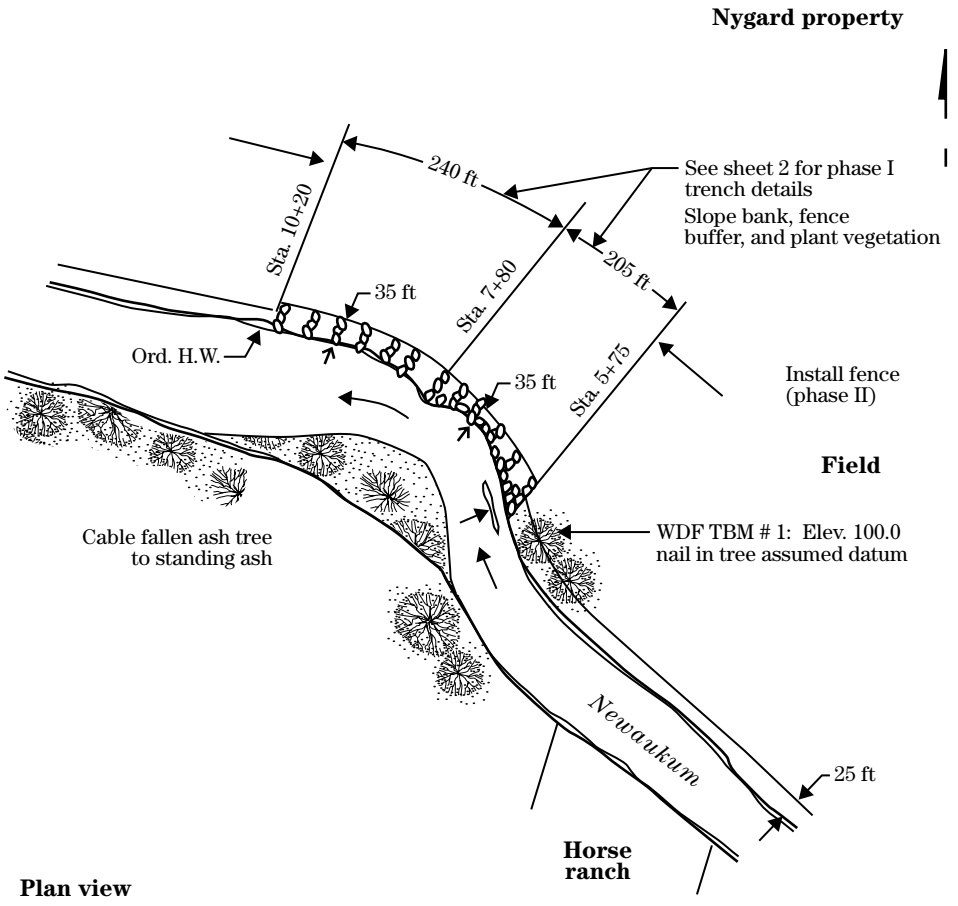

Figure CS10–3 Nygard site, RB detail—excerpt from original drawings

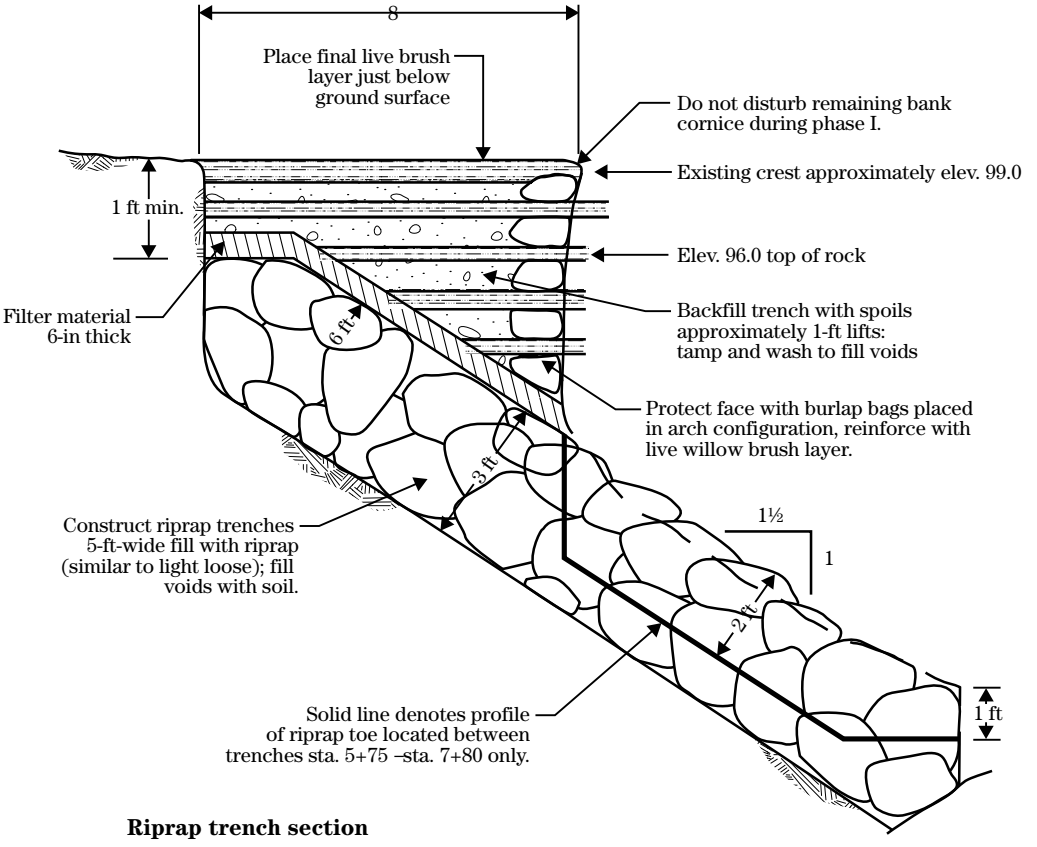

Figure CS10–4 Willow growth at RB structure

(a) Shading effect of willows

(b) Willow development above toe rock. There appears to be some loss of fine soils at the interface.

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.

Issued August 2007

Cover photo: Treated section of Cole Creek in the Red River Basin in North Dakota

Advisory Note

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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.

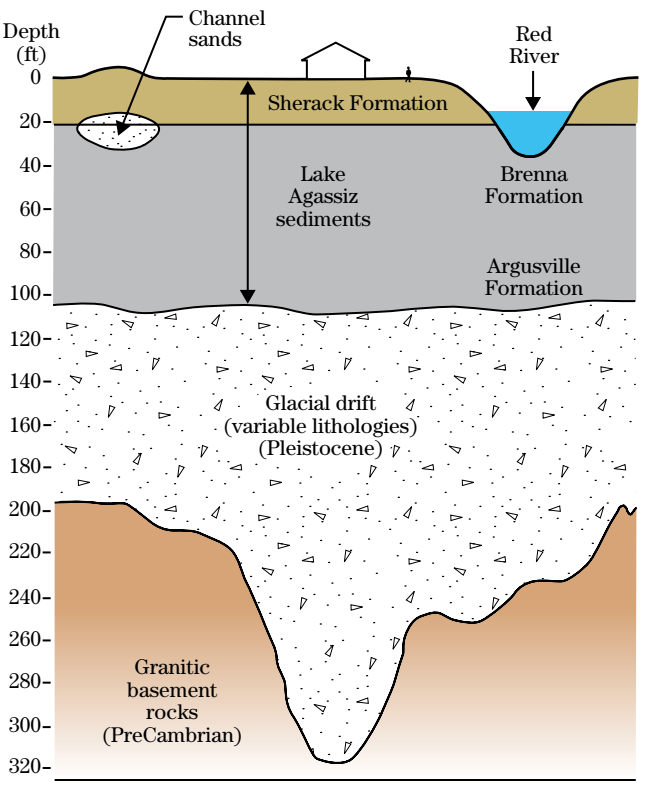
Figure CS11–1 Boundaries and shaded relief of the Red River Basin

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.

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



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

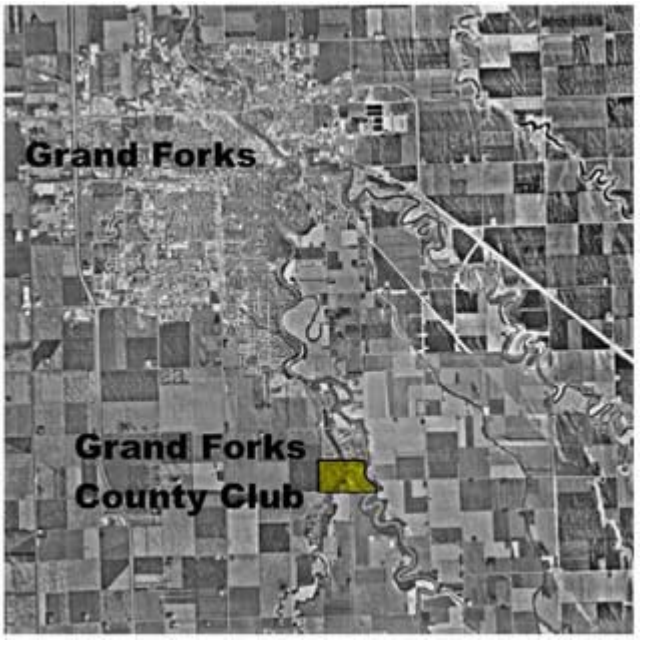
and flooding, especially during frost-free months, has drastically increased ground water levels and soil moisture. These changes in discharge and flooding have caused river channels to downcut and widen as they adjust to new flow regimes. The combined effect of the factors described above has been a rapid increase in the number of slope failures across the Red River Valley.

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.

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.

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

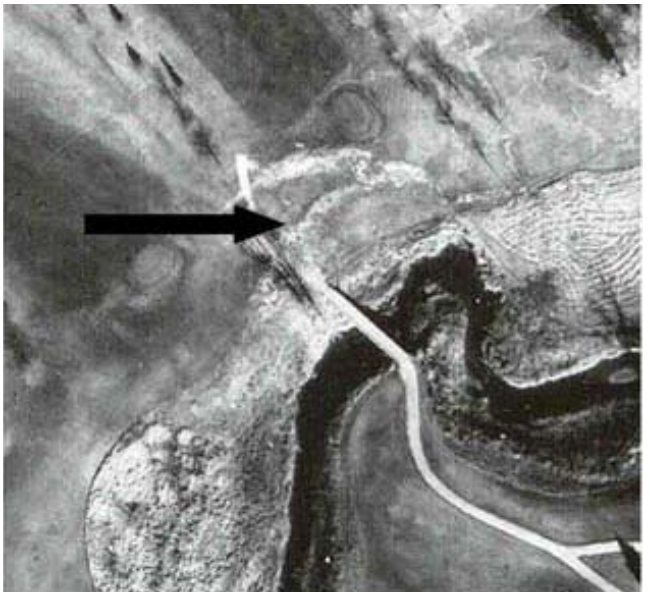
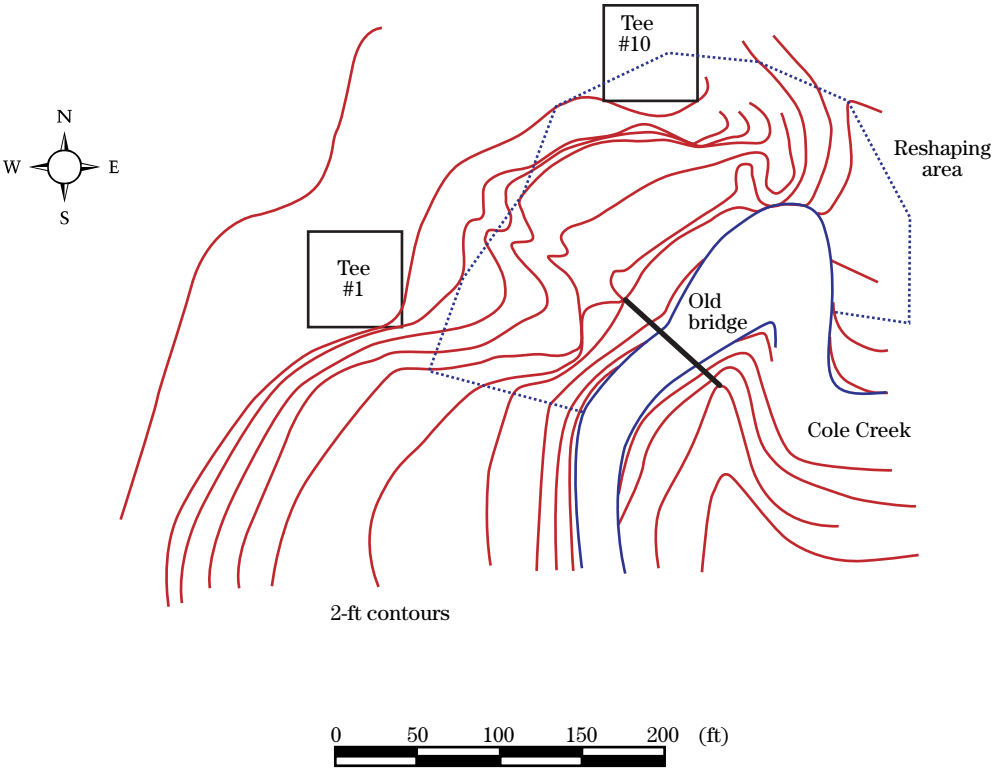


Figure CS11–7 Looking upstream on Cole Creek at the GFCC Cart Bridge (old)

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



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

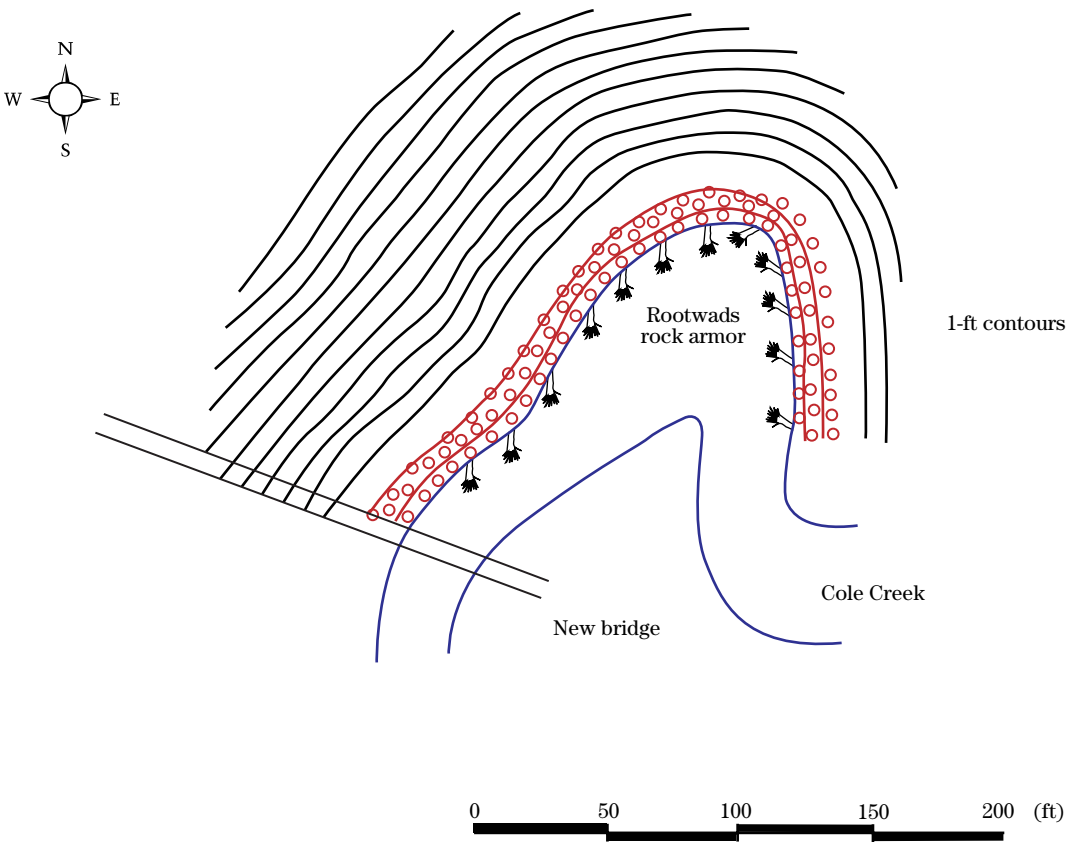
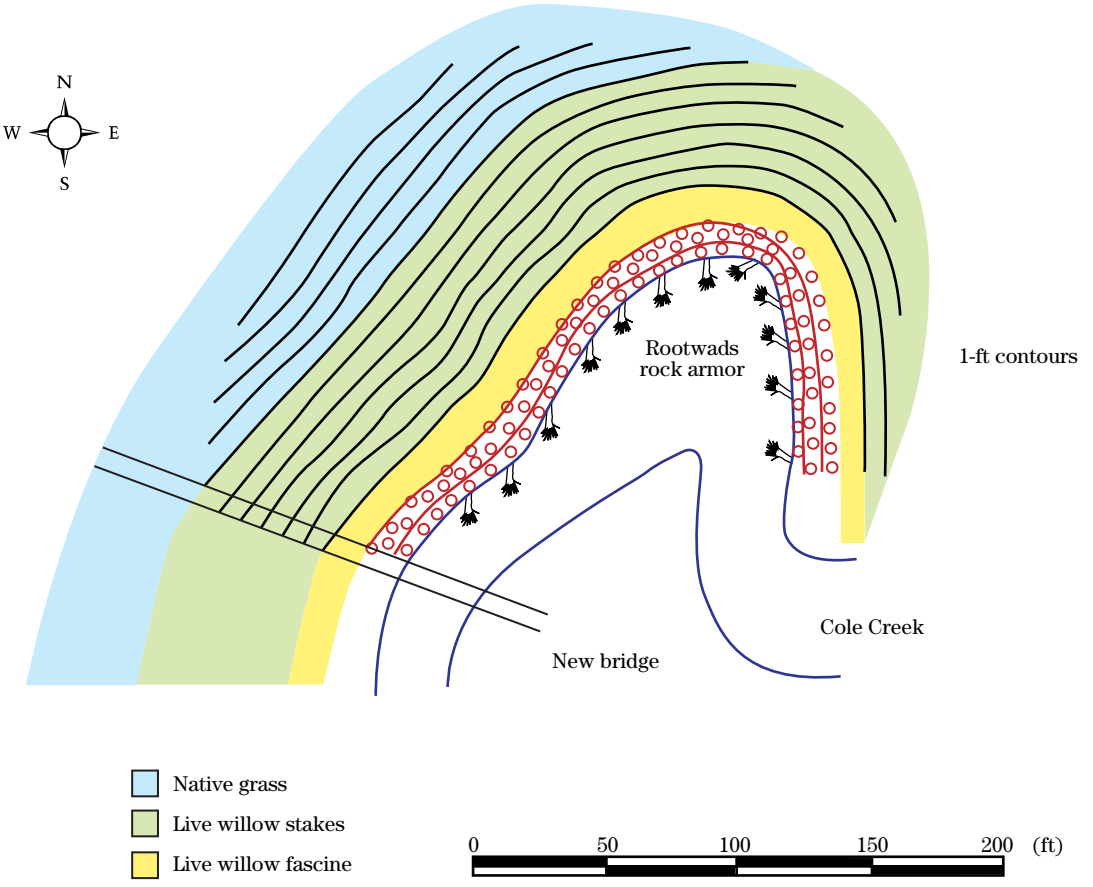


Figure CS11–11 Soil bioengineering and planting plan



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.

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.

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

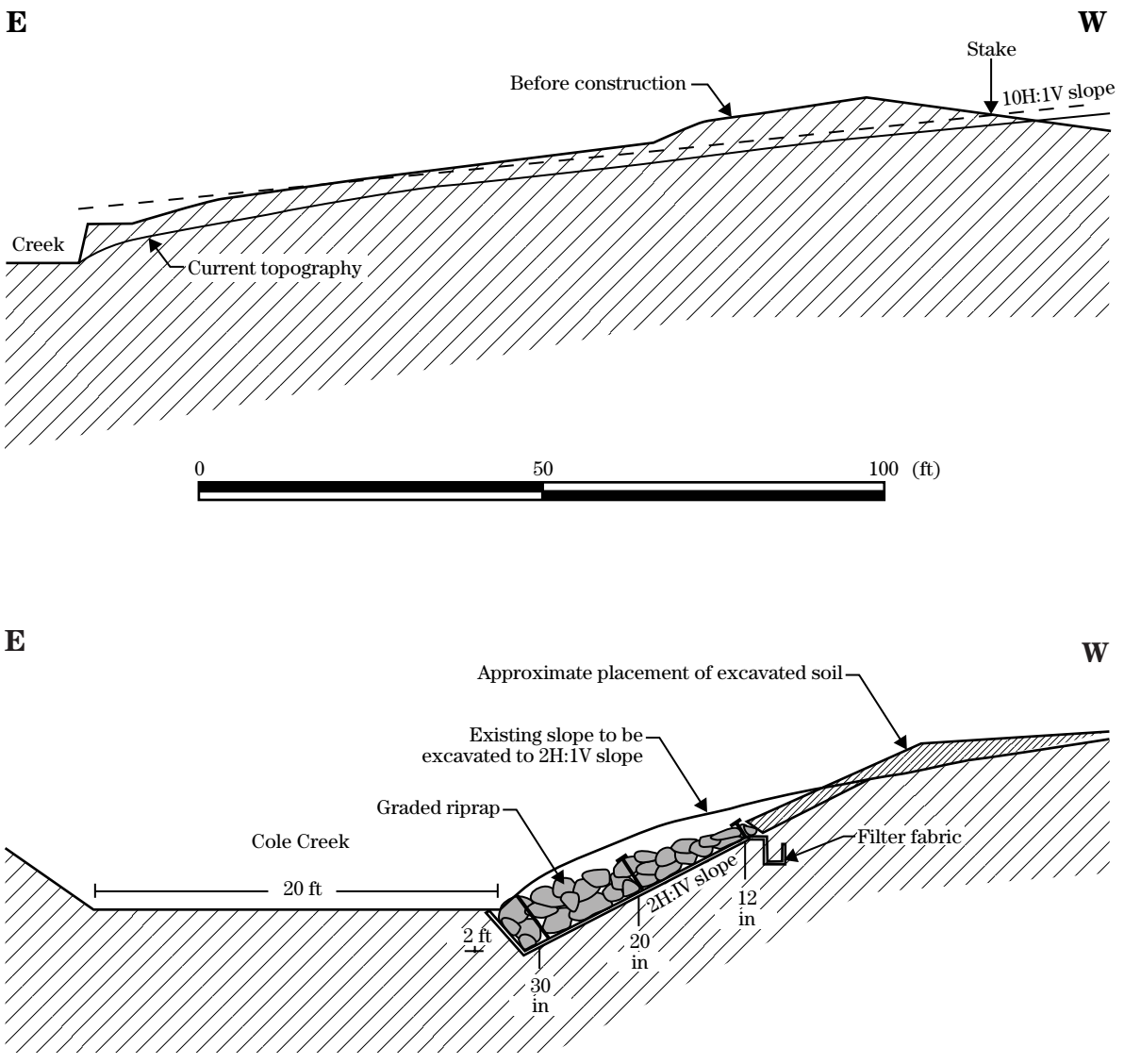
(a) Arrows indicate the same rocks in the stream

(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

July 2004

Issued August 2007

Cover photo: Grade control structure on David's Creek in western Iowa

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.

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.

- 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.

- 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.

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.

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.

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.

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.

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.

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.

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.

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

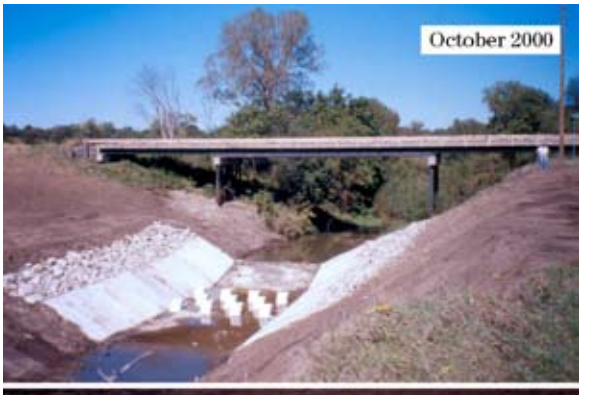
Data collected: Topographic survey

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.



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.

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)

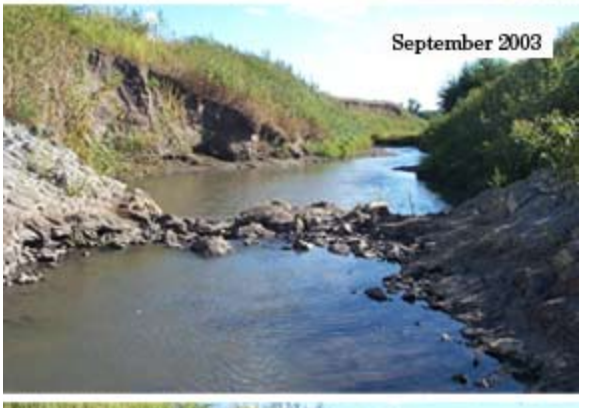
Data collected: Topographic survey

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.



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.

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.

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.

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.

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

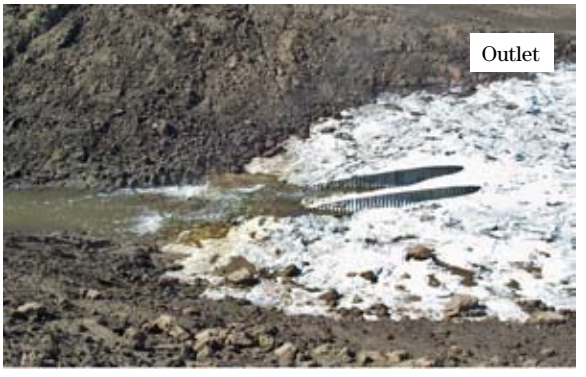
Goals met: All goals met

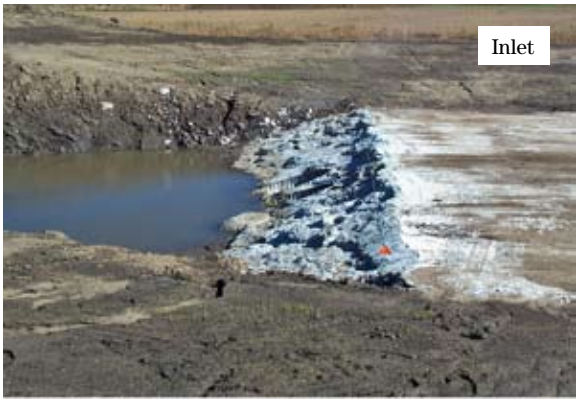
Project performance: Performing well and has been overtopped several times by high flows

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

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.

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.

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.

Issued August 2007

Cover photo: Treated section of Owl Creek on Owl Creek Farms, Ohio

Advisory Note

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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.

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.

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.

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.

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

Issued August 2007

Cover photo: Completed section of Merrimack River in New Hampshire

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By Ghislan Brunet, Maccaferri, Bioengineering and Ecological Systems, Maryland

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.

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.

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.

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.

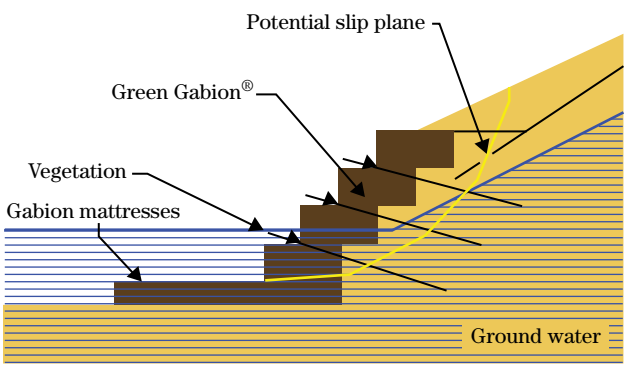
The slope stability analysis, done immediately after installation, resulted in a safety factor of 1.55 for global stability and 4.02 for internal stability (fig. CS14–1). The impact of the vegetation on slope stability was determined based on measured shear resistance of willow roots. The shear resistance 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

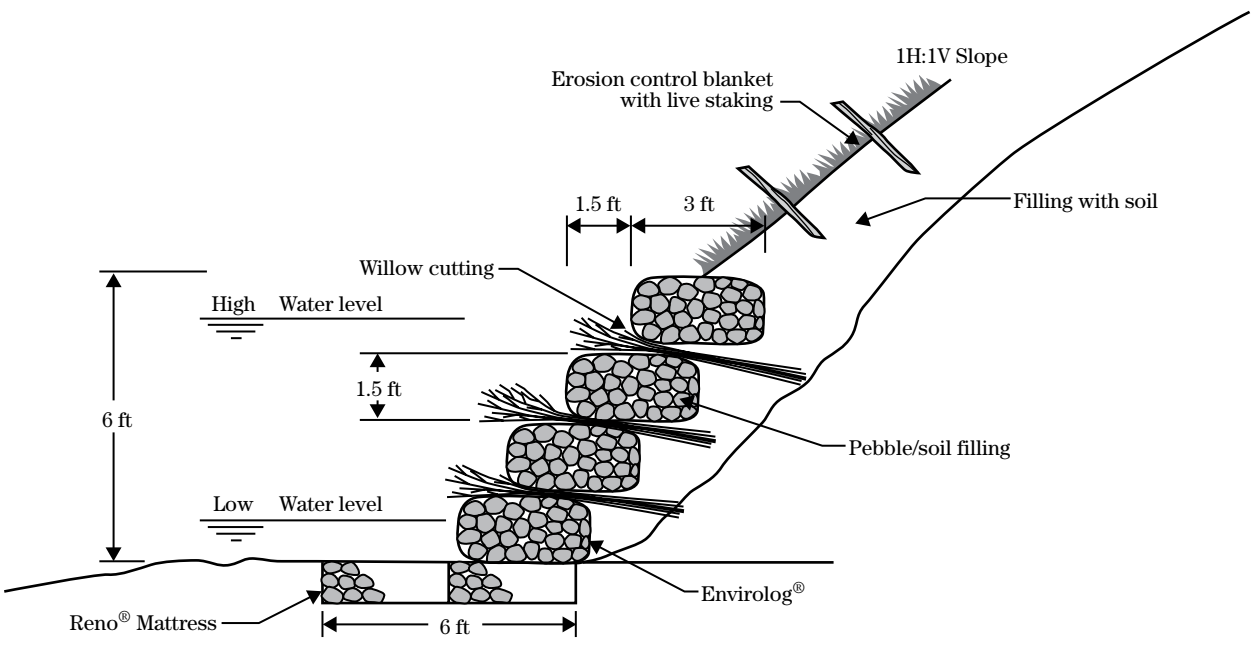


(3 ft), which is less than the maximum wave action allowed for stability.

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.

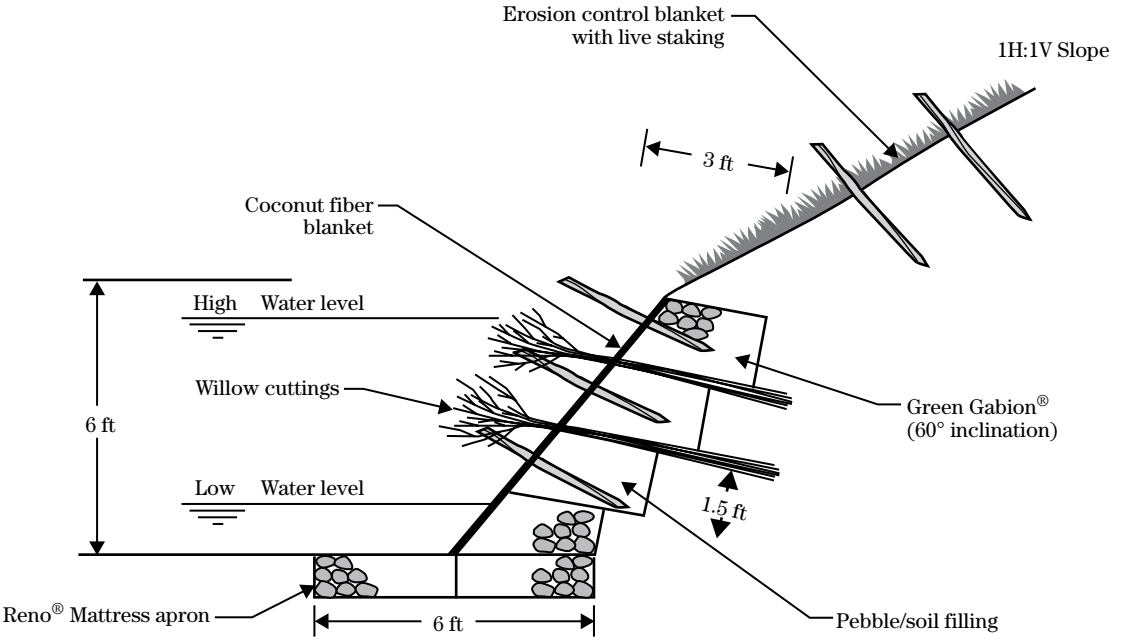
To prevent any potential scour under the structure, a Reno® Mattress of 30 centimeters (12 in) by 1.9 meters (6 ft) was installed at the base of the units, with an extension of 1 meter (3 ft) streamward to stop any erosion (figs. CS14–2 and CS14–3).

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



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.

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

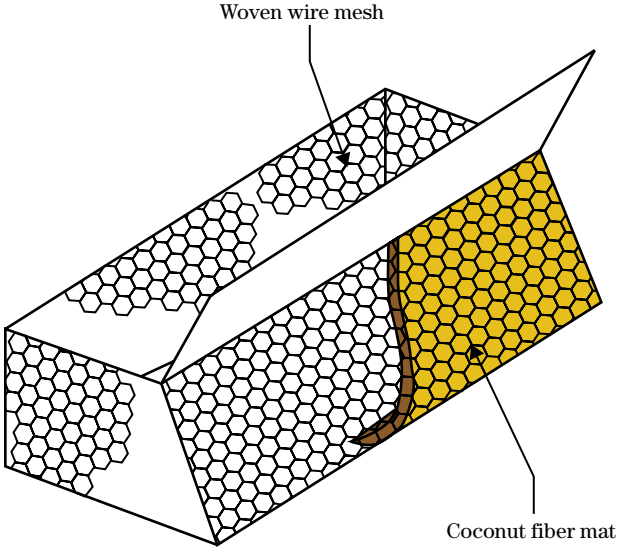
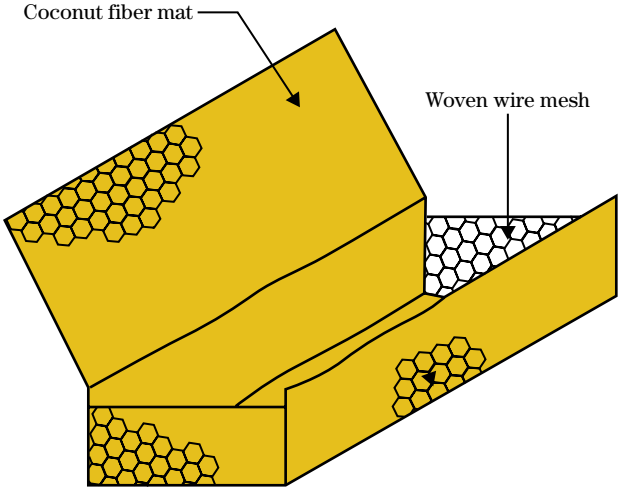


Figure CS14–5 Coconut fiber was used to line gabion baskets to retain soil for vegetation growth.



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

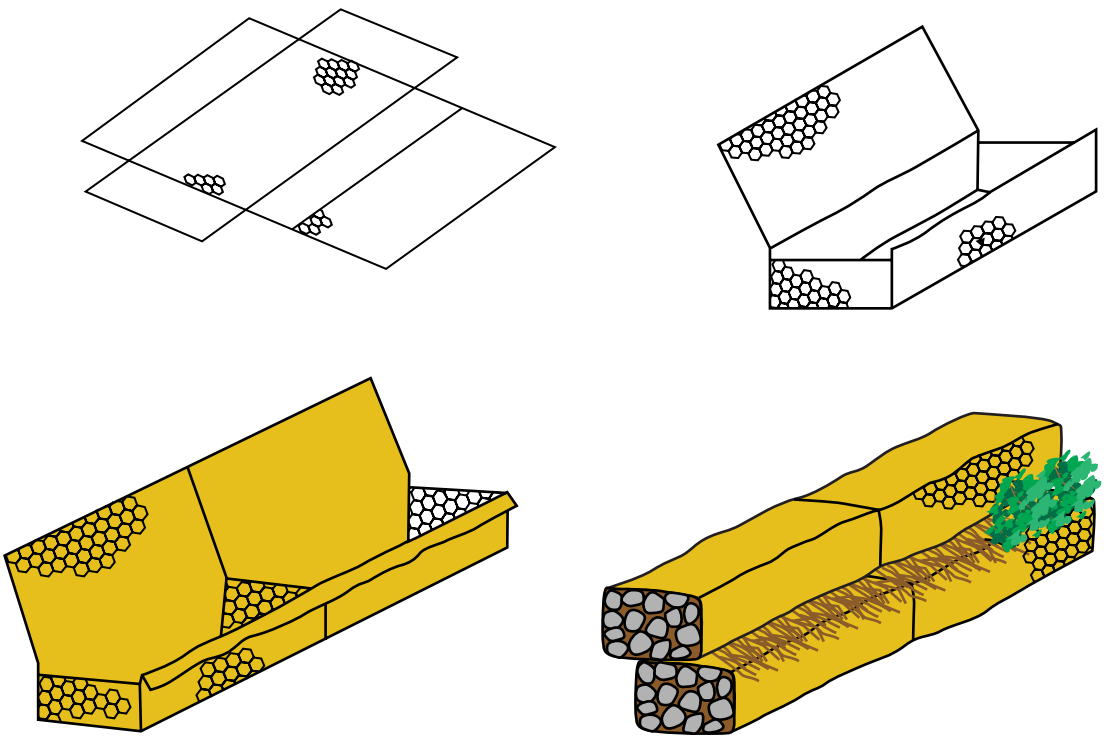


Figure CS14–7 Green Gabions® preassembled with coir fiber mat

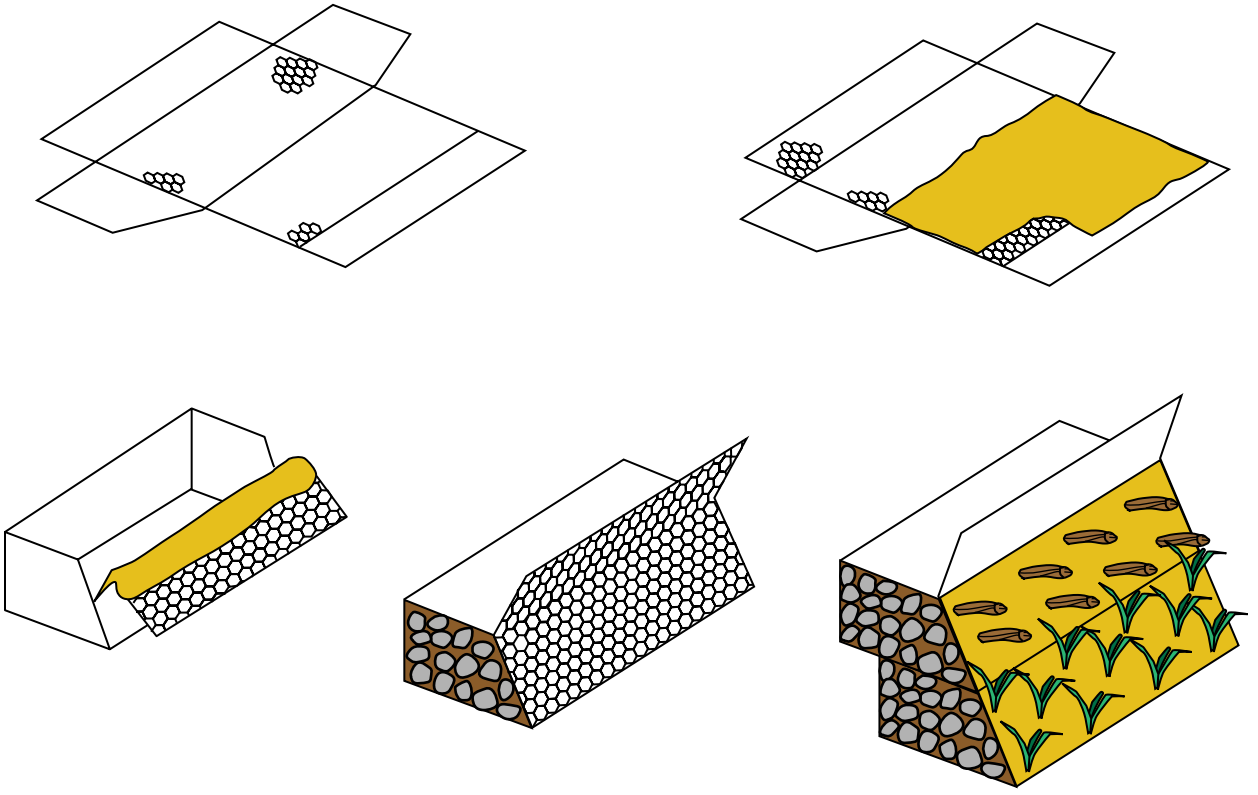


Figure CS14–8 Installed gabion baskets lined with coconut fiber mat and filled with stone and topsoil

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

Issued August 2007

Cover photo: Treated section of the Guadalupe River in Santa Clara County, California

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.

By John McCullah, Watershed Geologist, CPESC, and Kaila Dettman, Watershed Hydrologist of Salix Applied Earthcare

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.

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.

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.

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

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 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.

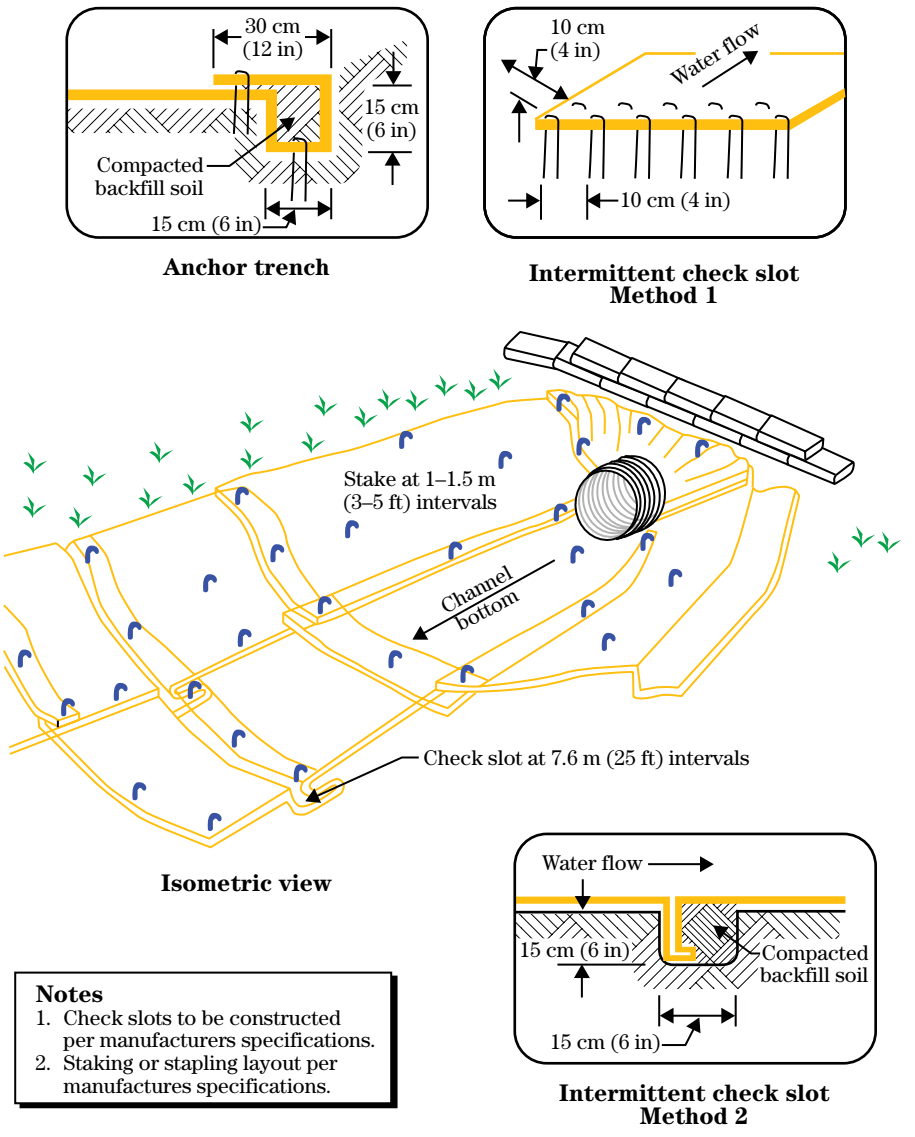
The use of stone was prohibited due to new policies that allowed only “green” solutions within the watershed. The recommendation that cobble be installed was met with resistance; however, regulatory personnel were reassured that cobble would provide natural habitat and aesthetic value while improving 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

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.

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).

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

Issued August 2007

Cover photo: Treated section of Coffee Creek in Edmond, Oklahoma

Advisory Note

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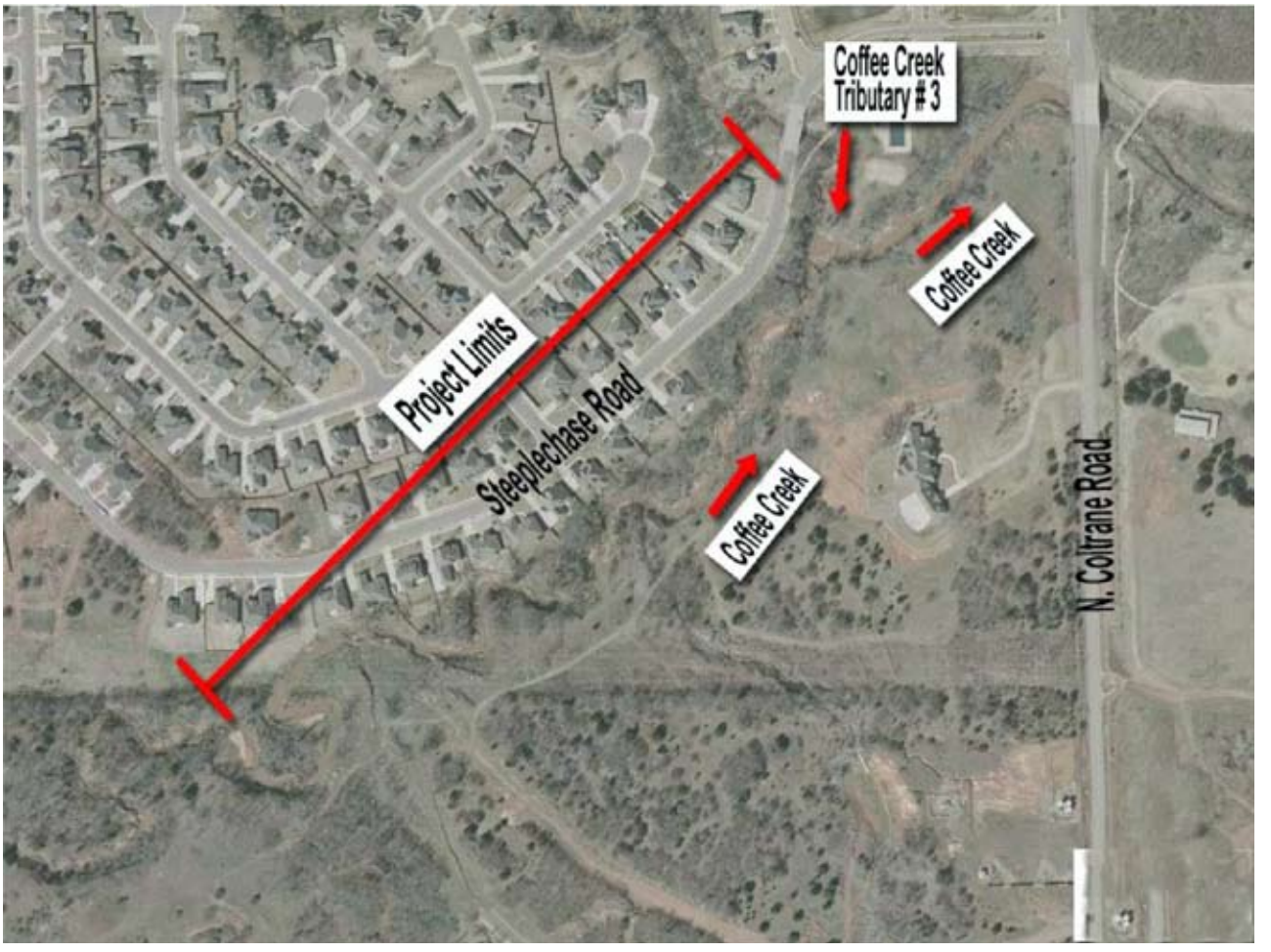
In October 2000, an extreme flood event caused severe erosion and lateral instability of a portion of Coffee Creek located along the southeastern edge of the Steeplechase subdivision in Edmond, Oklahoma (fig. CS16–1).

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



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.

Figure CS16–3 Streambank erosion prior to the project

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 CS16–5 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:

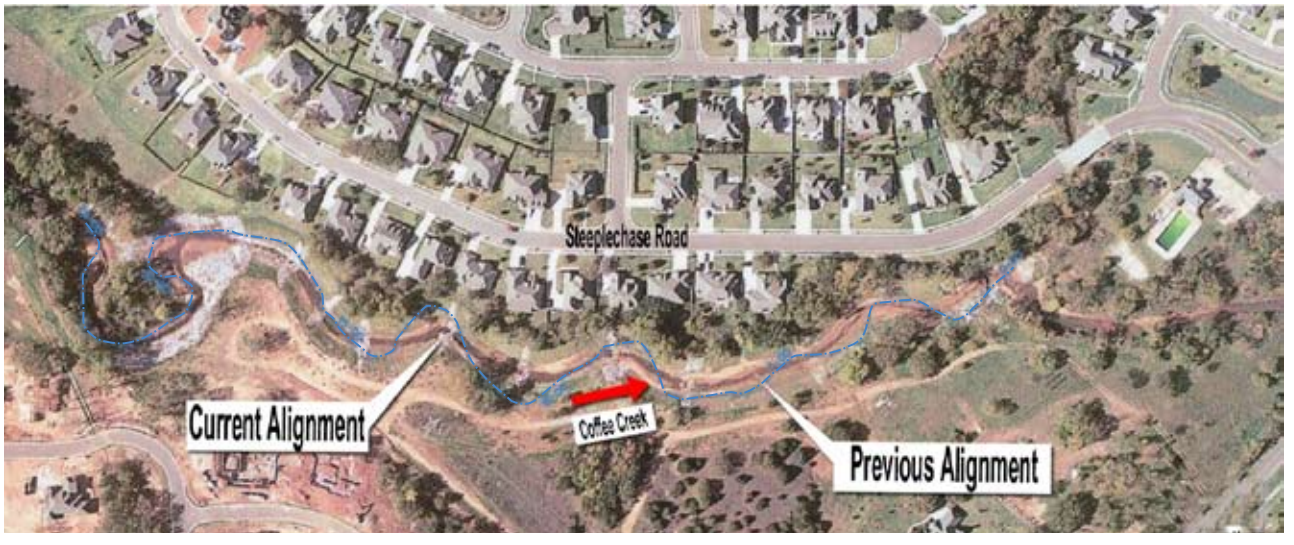
- using soil bioengineering techniques, while being confined to a less than desirable cross-sectional width
- protecting existing infrastructure and avoiding adverse impacts to other adjacent infrastructure

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.

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

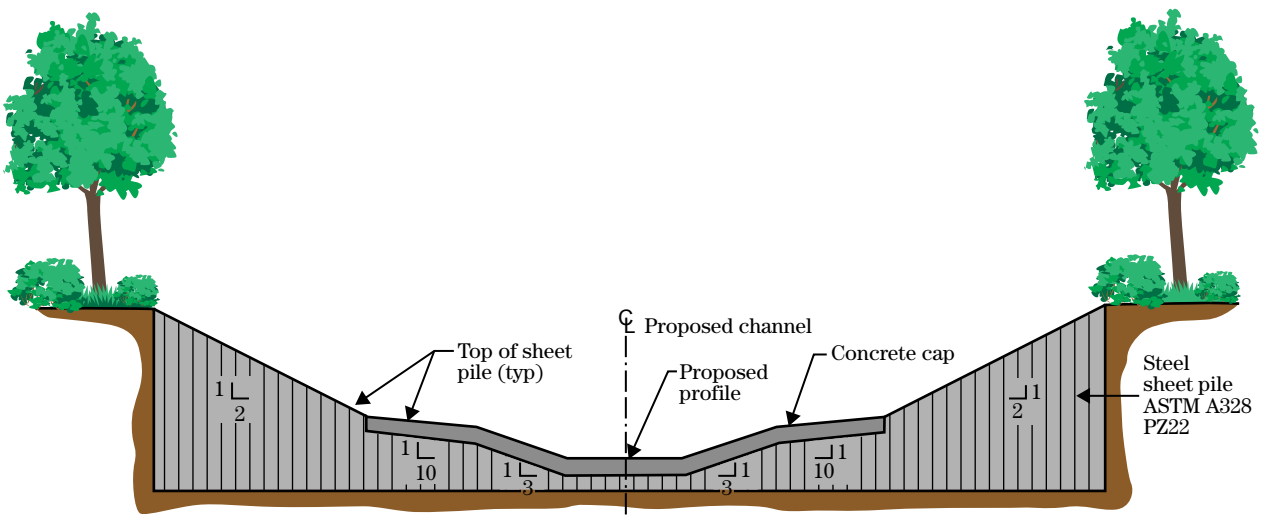
Figure CS16–6 Aerial view of project channel alignments



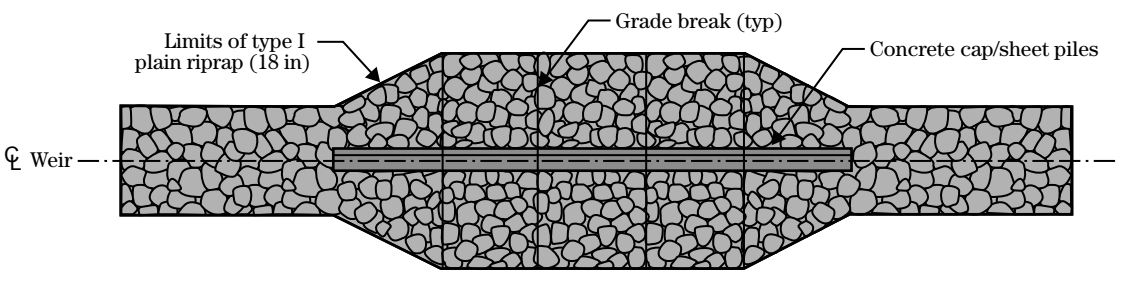
the alignment. A typical section and plan view of the weir structures is provided in figure CS16–7.

— 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



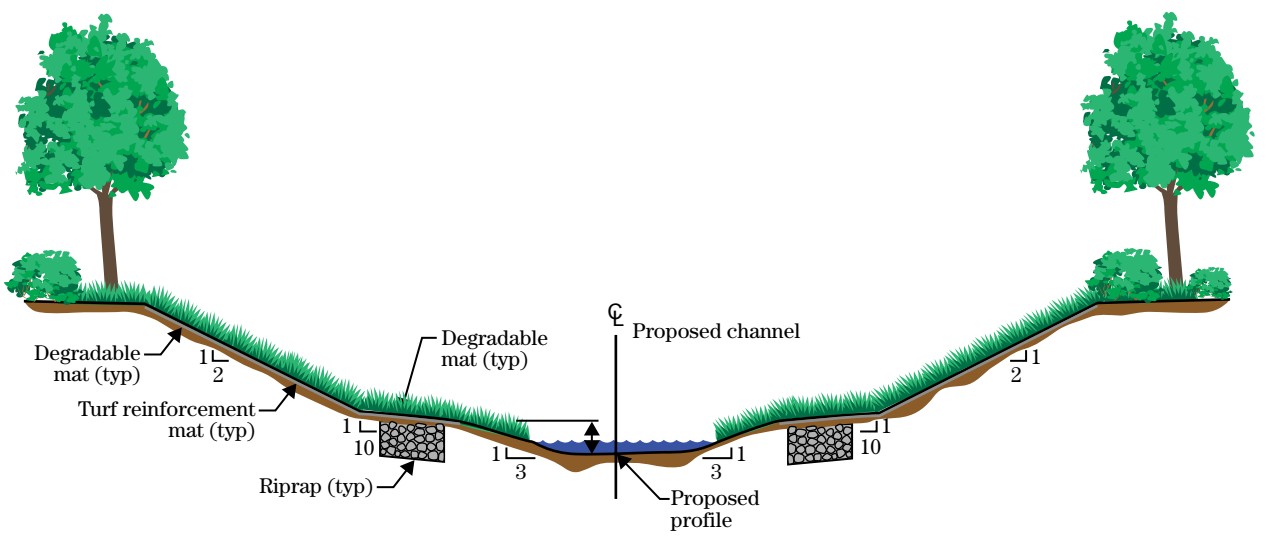
Channel weir plan view

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.

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

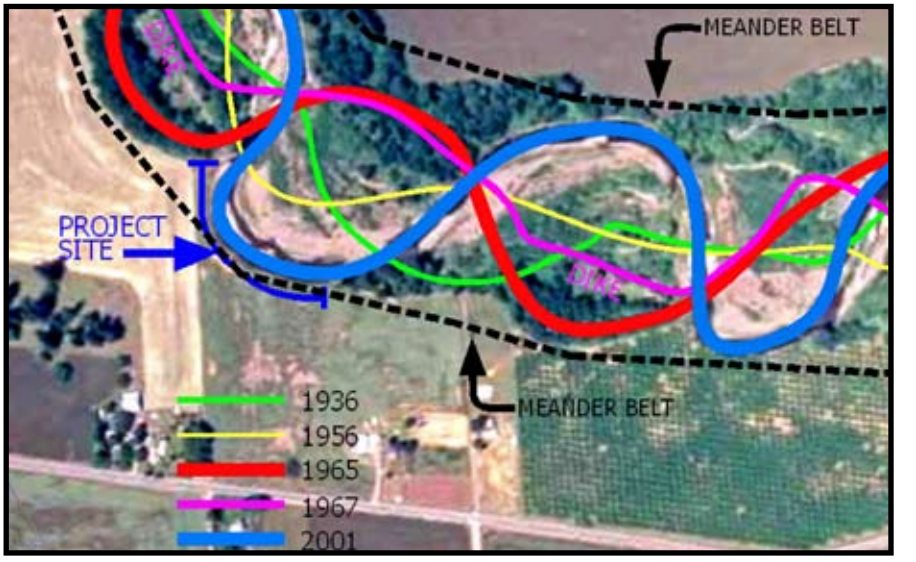


Channel typical section

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


MEANDER BELT
PROJECT SITE
DIKE
DIKE
MEANDER BELT
1936
1956
1965
1967
2001

Issued August 2007

Cover photo: Historic channel traces of the Calapooia River 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.

By Scott Wright, P.E., area engineer, U.S. Department of Agriculture, Natural Resources Conservation Service, Tangent, Oregon

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

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

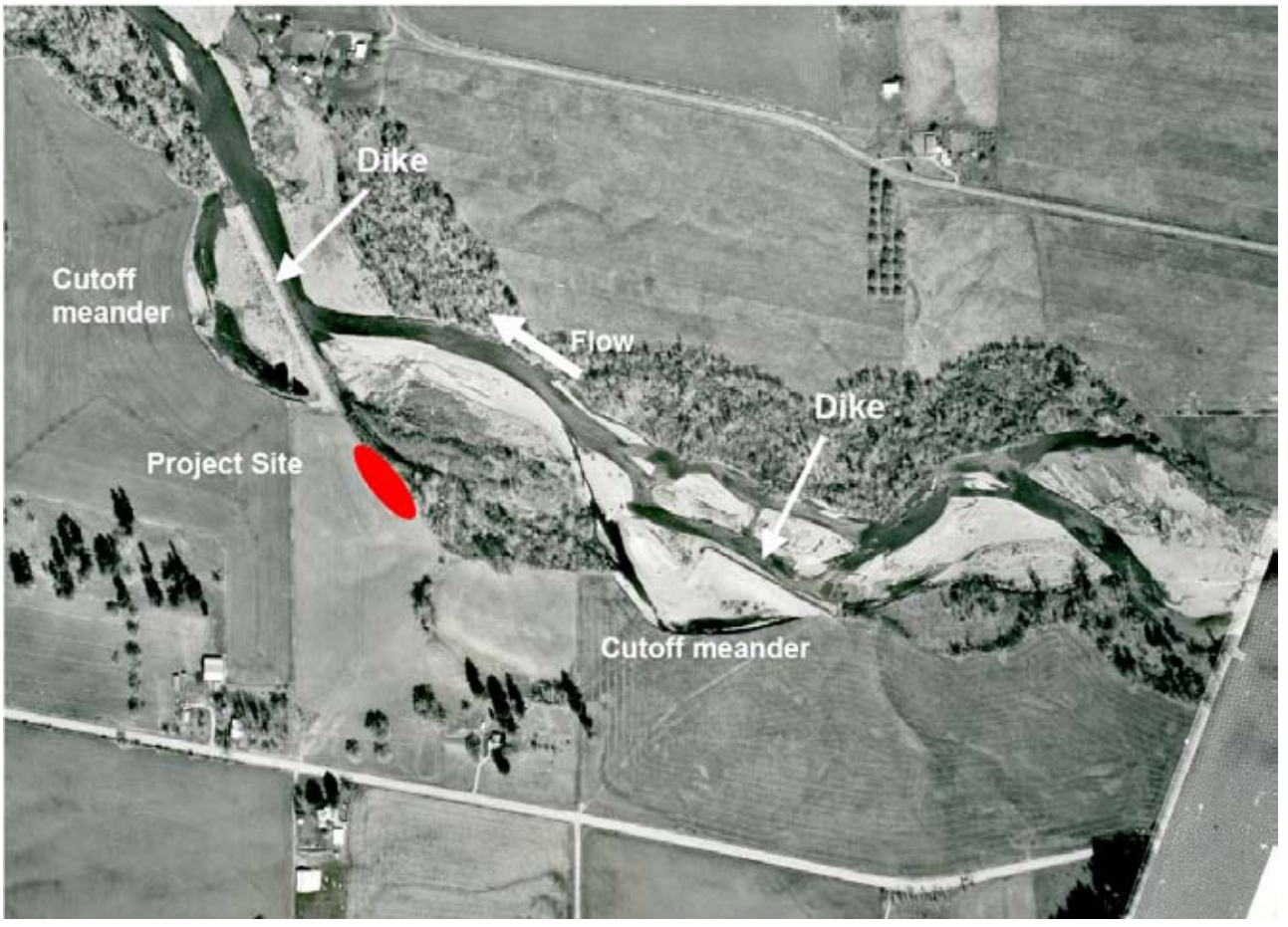
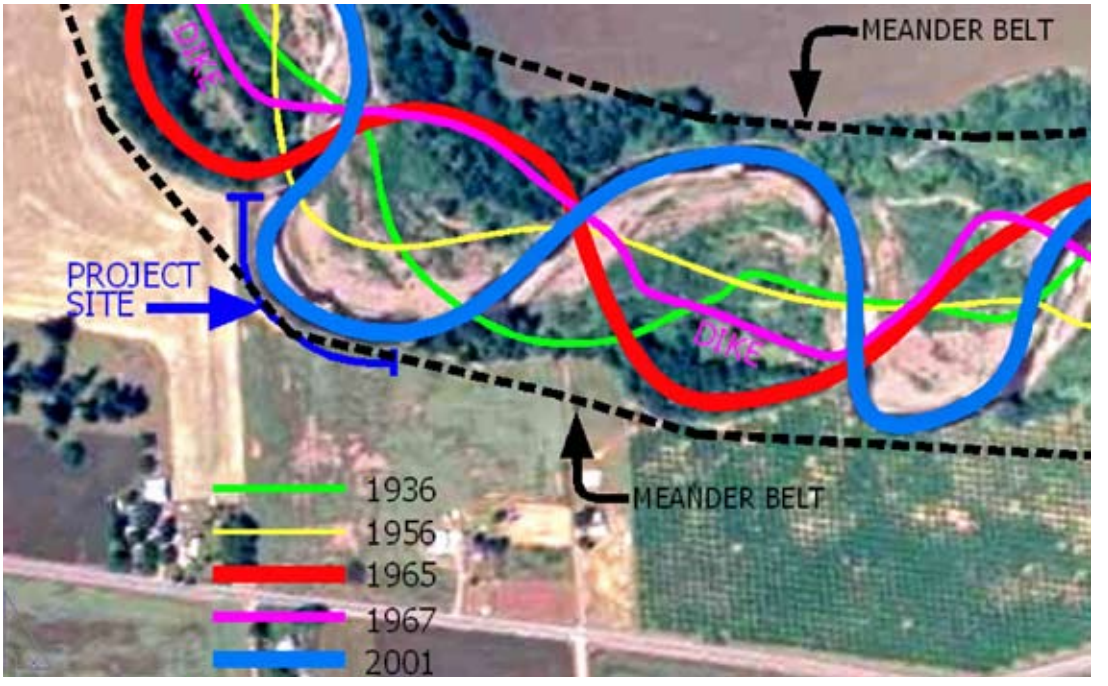


Figure CS17–4 Historic channel traces with corresponding year designated by color



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

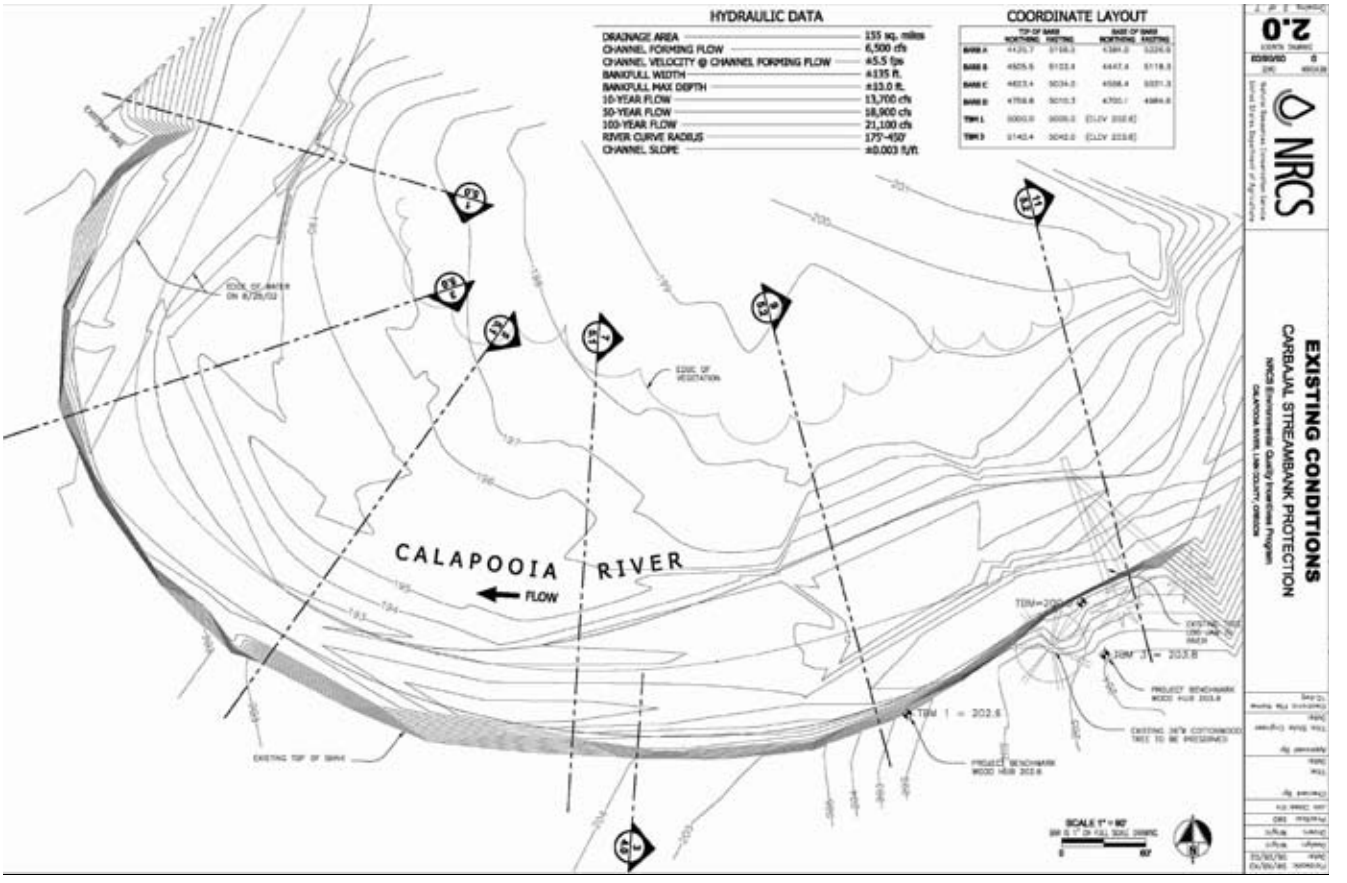


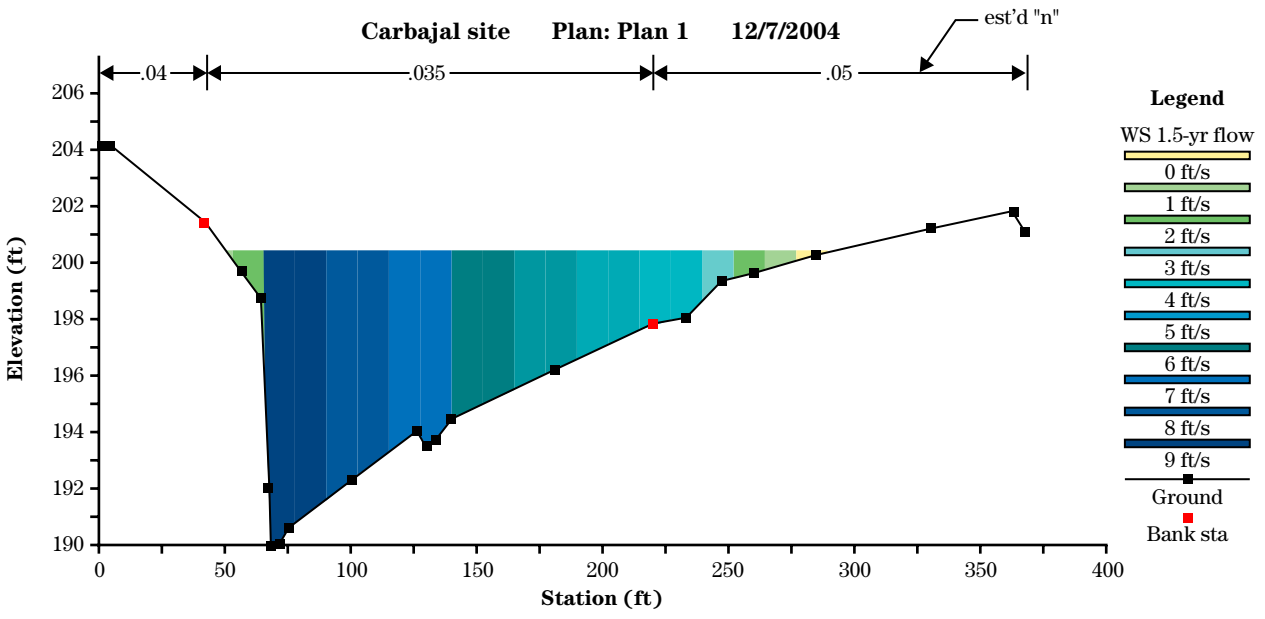
Table CS17–1 Project area characteristics

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



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 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*)

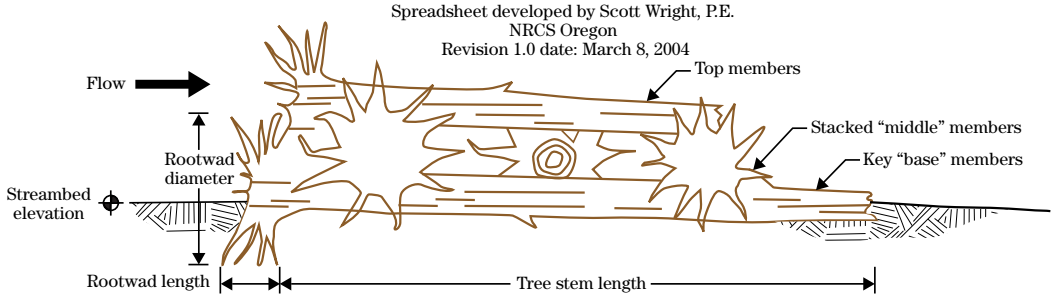
Table CS17–3 Summary of rock gradation used for barb construction

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.

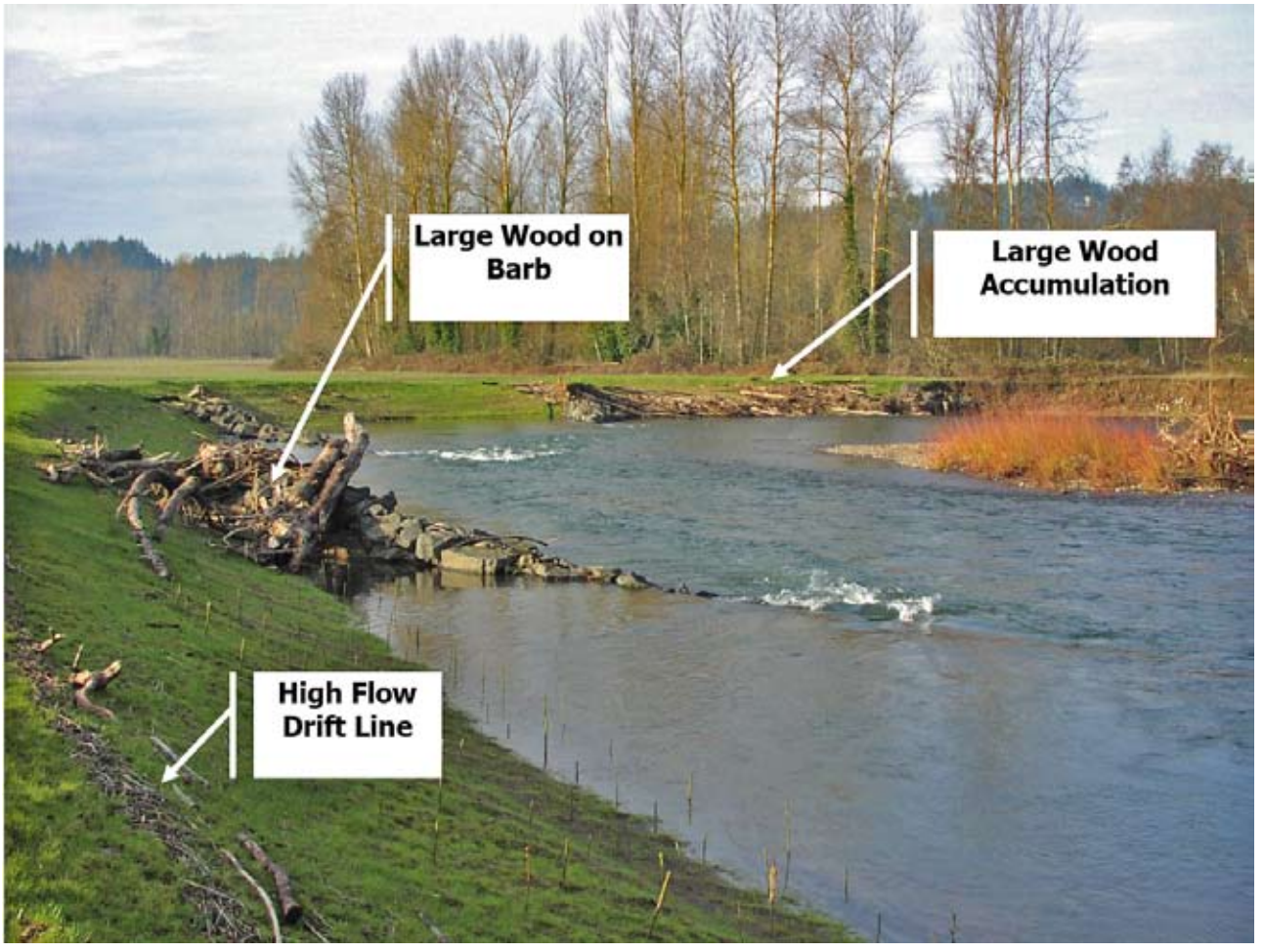
The existing bank consisted of noncohesive material and was geotechnically unstable. Therefore, the bank was shaped and excavated from the summer low-water elevation to the catch point of the existing ground at a 3H:1V slope. This slope creates a stable bank and provides an optimal surface to plant vegetation. 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.

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*)



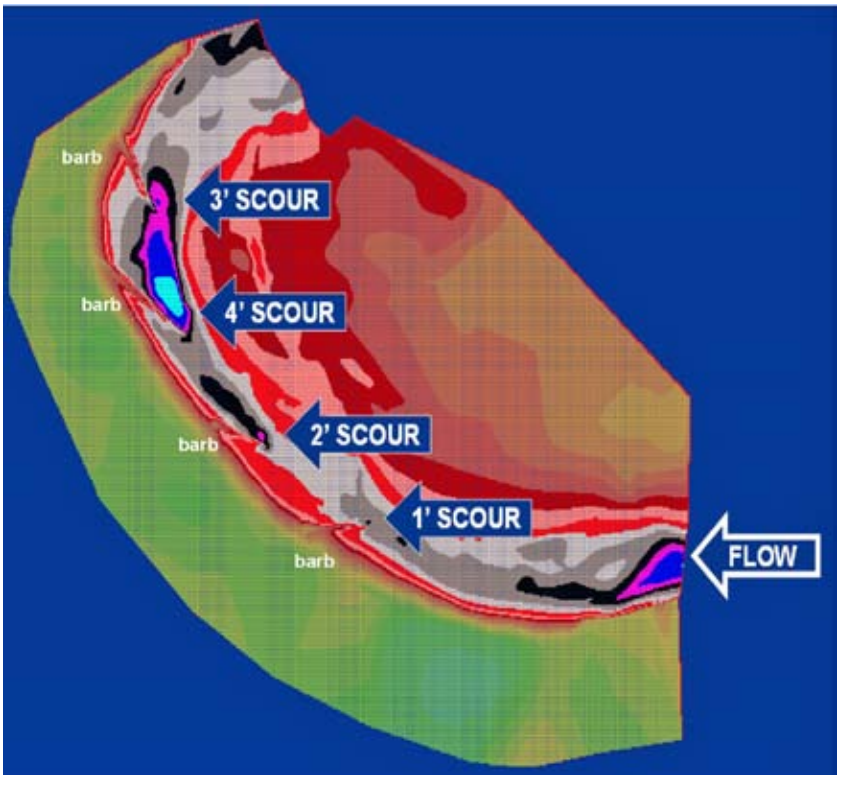
Detailed topographic information was collected pre- and postconstruction to identify actual geomorphic effects of the stream barbs and overall performance. Figure CS17–12 identifies the actual scour that occurred around each barb and the streambed.

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.

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.

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*)

Issued August 2007

Cover photo: Embankment treatment on Wiley Creek in Oregon

Advisory Note

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By Sean Welch, state hydraulic engineer, U.S. Department of Agriculture, Natural Resources Conservation Service, Portland, Oregon

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.

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

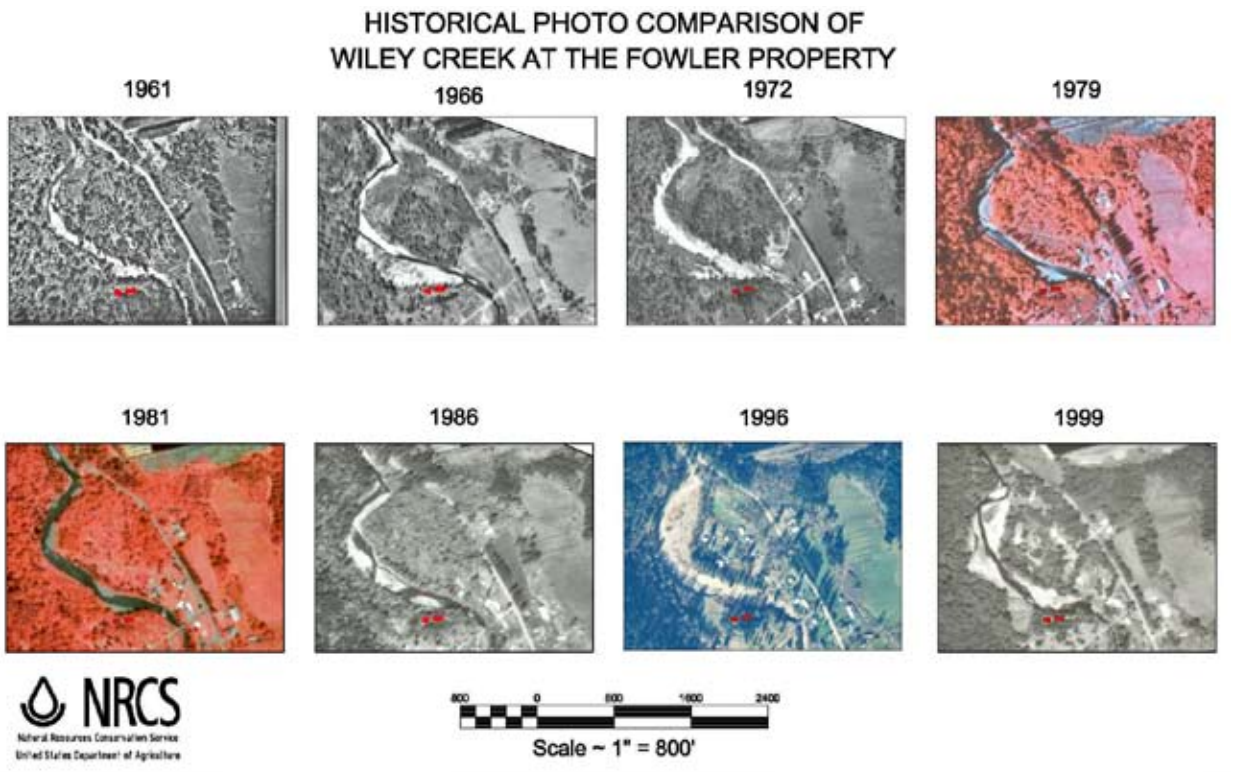
bank with two structures approximately 5 feet from the top edge of the bank (fig. CS18–1). Anecdotal information from the landowner and analysis of historical photographs indicated that the river's left bank had eroded more than 40 feet since the rain-on-snow flood event of 1996 (fig. CS18–3).

Design 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.

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

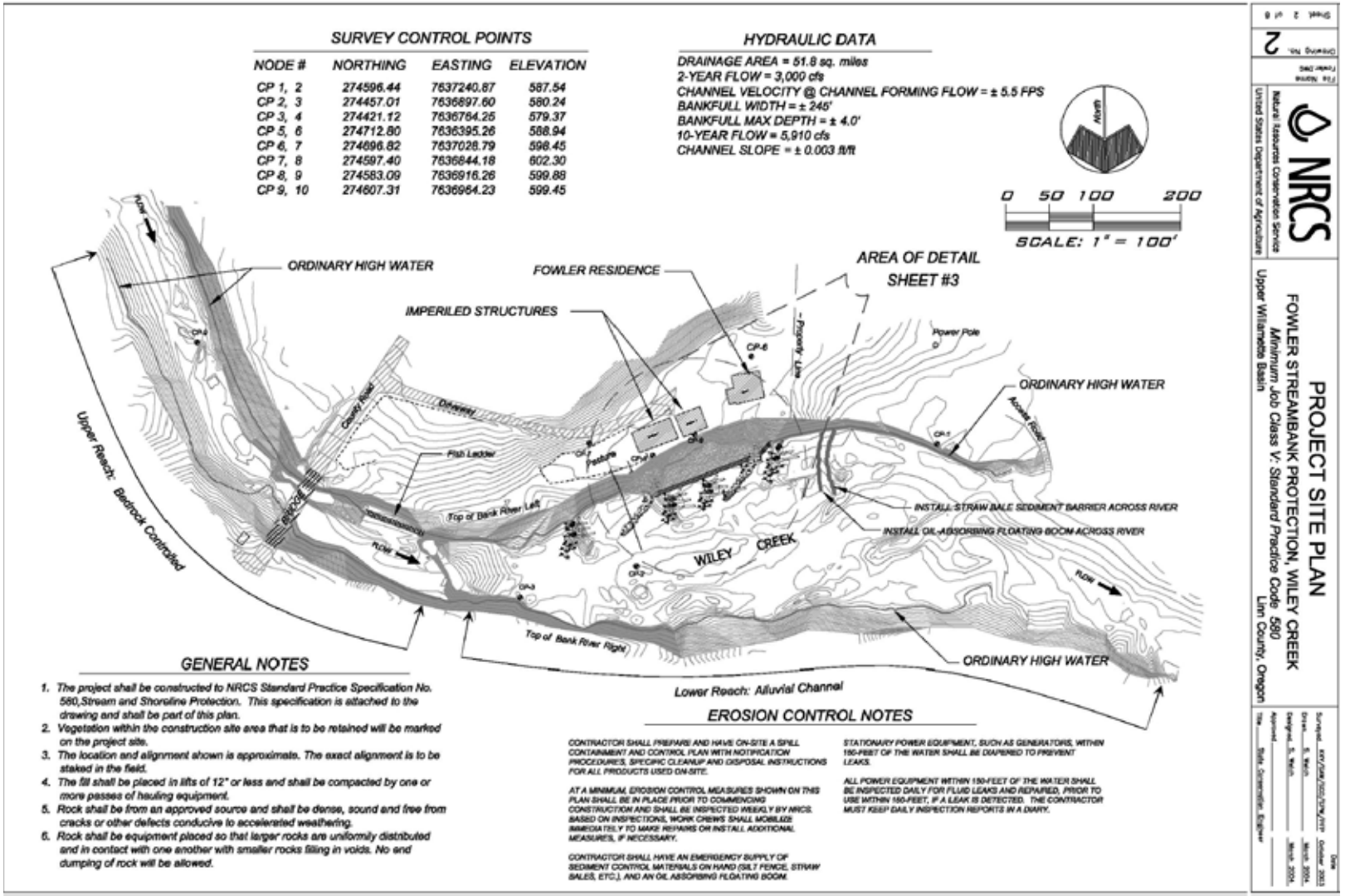
Figure CS18–4 (a) Looking upstream from the Wiley Creek Bridge at the B1c reach above the project; (b) looking upstream to the bedrock overfall ledge. The Wiley Creek Bridge can be seen in the background. This location marked the transition from the B1c to D4 reach.

(a)

(b)

Figure CS18–5 Looking upstream at the D4 reach

Figure CS18–6 Topographic site map of the Wiley Creek Project



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.

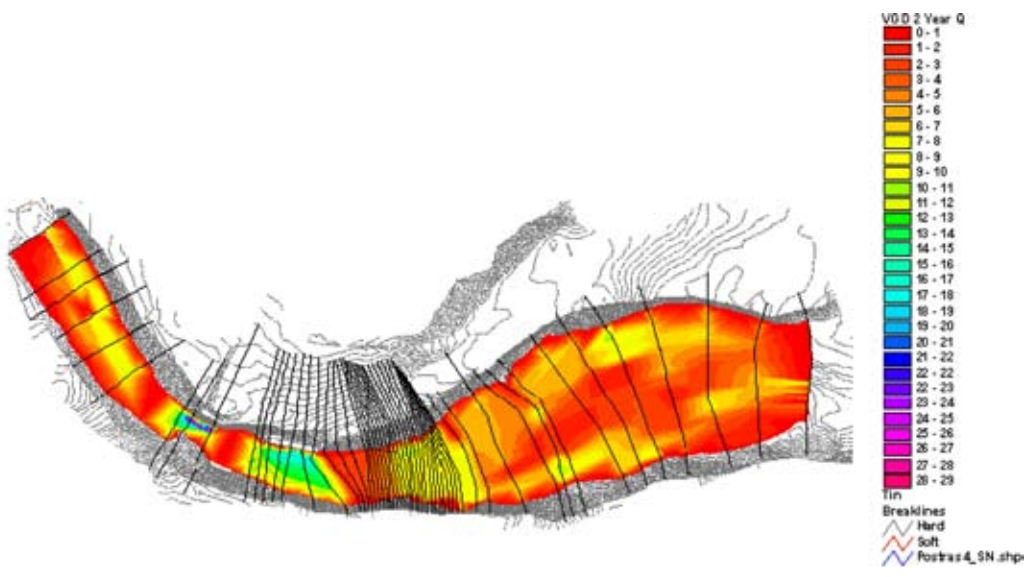
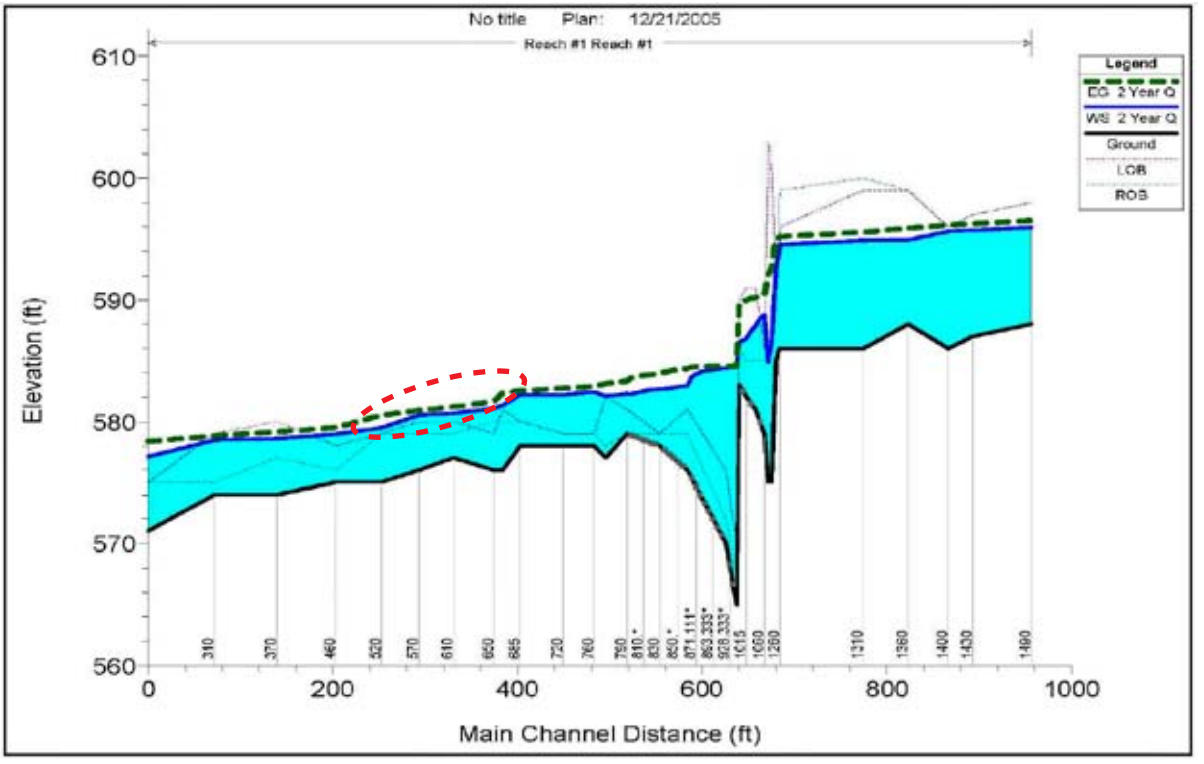


Figure CS18–8 Computed water surface profile and energy grade line for the 2-year flood. Note hydraulic jumps between sections 1260 and 928 across the bedrock overfall upstream of project reach. Project reach is defined by red oval.



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

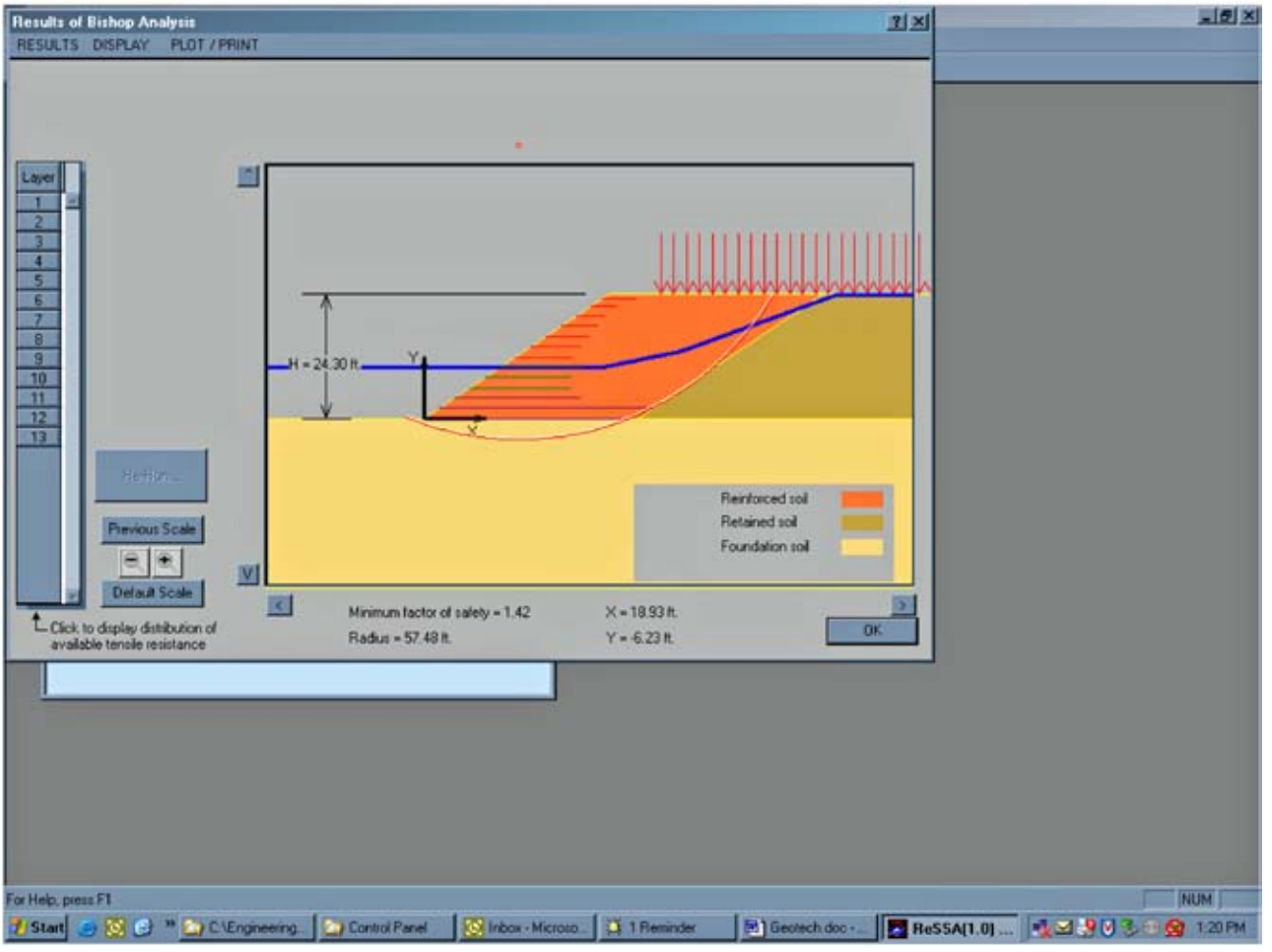
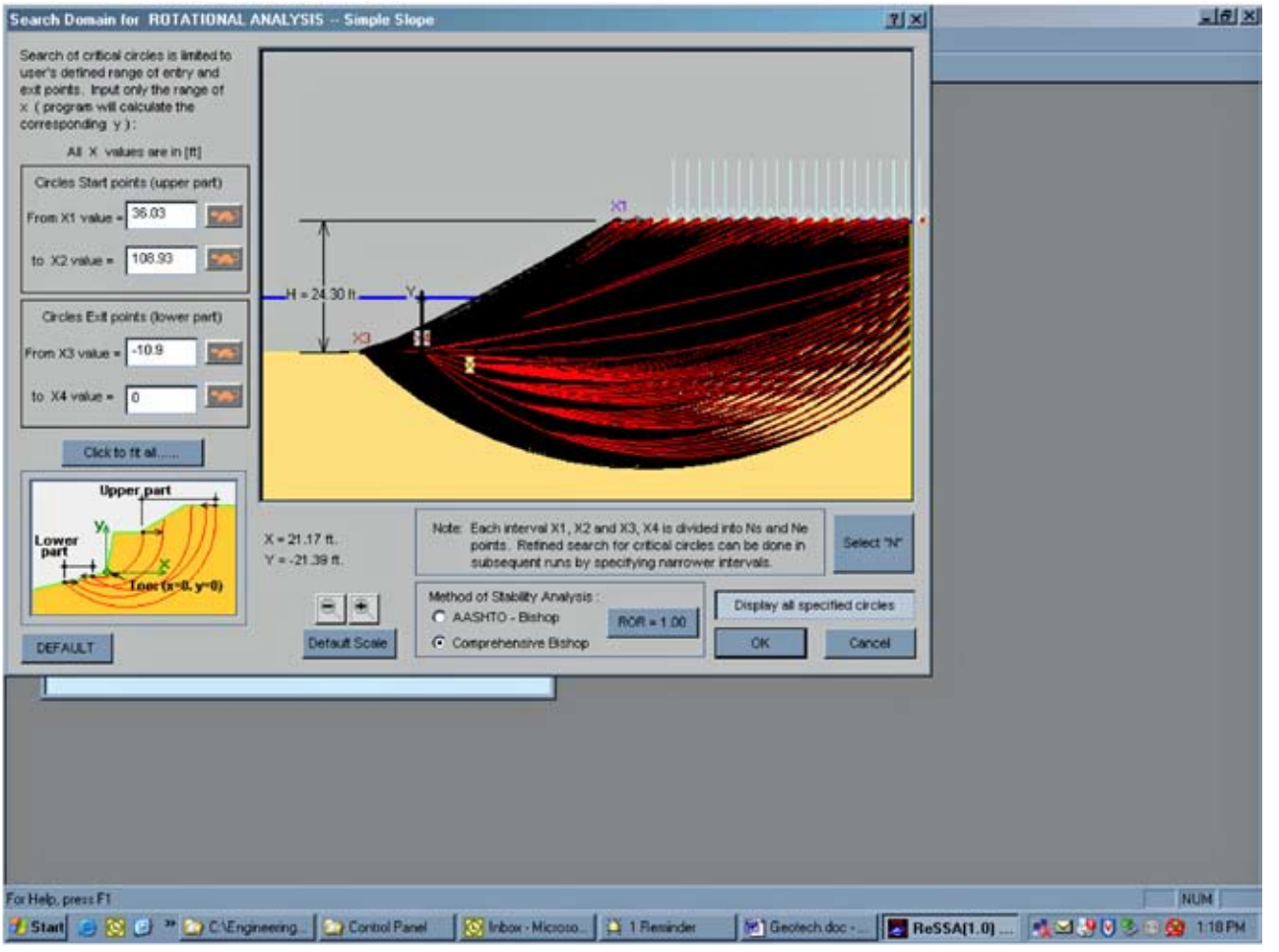


Figure CS18–12 Bishop slices showing optimization results for rotational bank failure



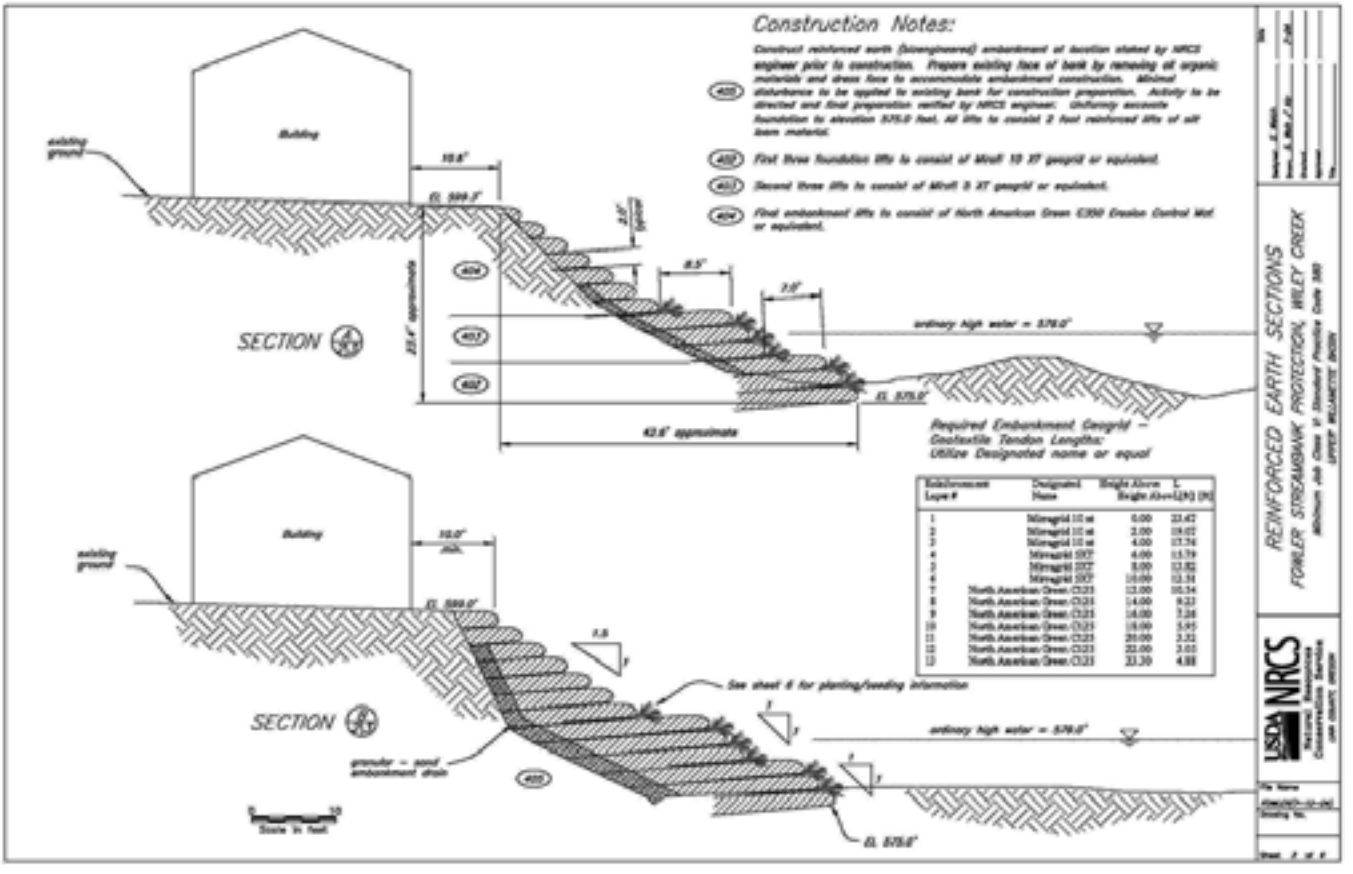
program optimizes on these two failure scenarios and computes required geogrid tendon lengths based on a user-provided elevation schedule.

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.

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



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.

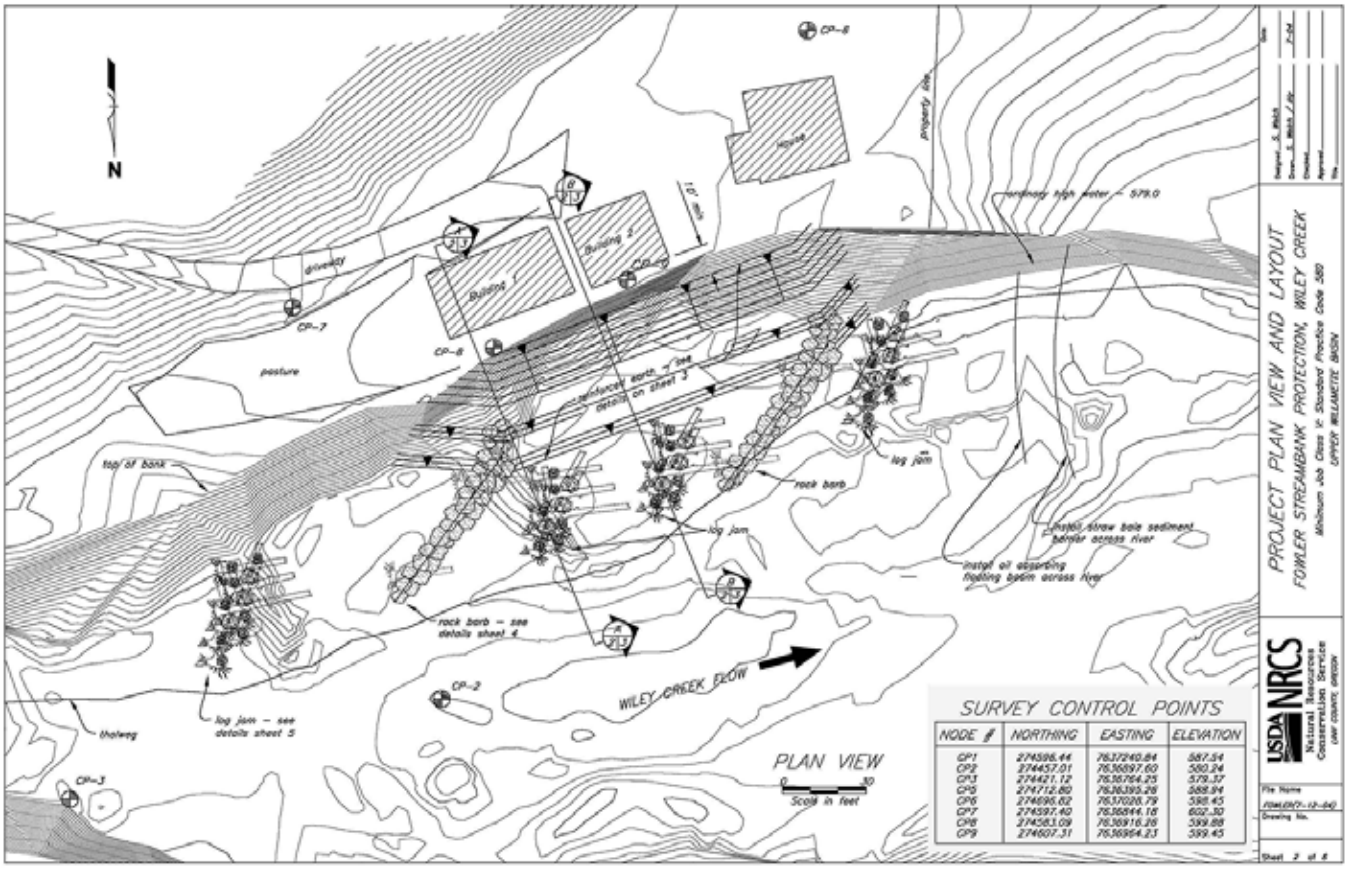
Two stream barbs incorporating large wood were used for hydraulic control at the upstream and downstream ends of the reinforced earth embankment. Barbs are a proven technology for near-bank velocity reduction and bank protection. NEH654 TS14H provides design guidance for these structures including geometric 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



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)

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)