Use of Historic Information 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.
Contents

Introduction TS2–1

Basic data sources and techniques TS2–1

Contemporary descriptions TS2–1

Weather events and climate TS2–1

Land use TS2–1

Land use changes................................................................................................... TS2–3

Land (cadastral) surveys................................................................................... TS2–3

Historic condition TS2–5

Field evidence .................................................................................................... TS2–5

Litter .................................................................................................................... TS2–6

Permanent landscape features........................................................................ TS2–6

Historic maps TS2–10

Instrumented topographic surveys................................................................. TS2–12

Aerial photographs TS2–12

Ground-based oblique photography................................................................. TS2–12

Photographs of site........................................................................................... TS2–14

Descriptive accounts and interviews with residents...................................... TS2–14

Summary and conclusions TS2–14

Tables

Table TS2–1 Sources of historic condition information TS2–5

Figures

Figure TS2–1 Precipitation time trends in the Driftless area of the Upper Midwest, 1867–1974 TS2–2

Figure TS2–2 Changes of land use, Coon Creek, WI, 1850–1975 TS2–3

Figure TS2–3 Sediment deposition rates based partially on topographic surveys at selected valley sites in Coon Creek Basin, WI, 1853–1977 TS2–4

(210–VI–NEH, August 2007) TS2–i
Figure TS2–4  Cross sections of Middle Oconee River at Highway 11 bridge, Jackson County, GA, showing stream aggradation between 1928 and 1969, based on bridge plans and resurvey  

Figure TS2–5  Headwater erosion rates based on sediment accumulation behind a small dam, Vernon County WI, 1936–1977  

Figure TS2–6  Sediment yields for the Tennessee River Basin c.1940–1975, based on TVA reservoir surveys  

Figure TS2–7  Original land survey plat of the confluence of Sandy Creek with the North Oconee River, dated 8 Jan. 1785  

Figure TS2–8  Air photos showing changes in land use in Vernon County, WI, between 1934 and 1967  

Figure TS2–9  Oblique ground photos showing improvement of tributary streams in Vernon County, WI, 1940–1974
Technical Supplement 2

Use of Historic Information for Design

Introduction

This technical supplement describes the use of historic information in the assessment of stream and watershed form and process. Form and process of effects in one place, however, may be the cause of form and/or process elsewhere. The historic assessment of climate and land use can also address causes. This information can be invaluable in the determination and assessment of goals and objectives.

Basic data sources and techniques

The value of the historical approach is shown not only by the number of practitioners who use an historical basis in determining project goals but also by the number of methodological papers on this topic which have appeared in the last quarter century. The reader is directed to Thornes and Brunsden (1977); Hooke and Kain (1982); Gregory and Walling (1979, 1987); Grove (1988); Cooke and Doornkamp (1990); Trimble and Cooke (1991); Trimble (1998); Collins and Montgomery (2001); Trimble (2001); Brown, Petit, and James (2003); Gurnell, Peiry, and Petts (2003). While it is possible to identify some general principles of using historical data, it is impossible to give clinical directions on their use because every application is different; the reader should refer to the works listed above. Instead, this chapter presents some basic approaches using historical data and analyses of the study of stream and watershed forms and processes.

Contemporary descriptions

Written accounts take two basic forms. The first is a description of past events or changes, while the second is the description of contemporary or baseline conditions useful for later comparisons. One must consider the scientific credentials of the observer and the stage of scientific development at time of the observation. Accounts from scientific observers tend to be dependable and are often extremely helpful, especially when they relate to direct observations like the clarity of streams. Observations from less-qualified people also can be helpful, but may need more qualification or interpretation. Newspapers, periodicals, books, government records, and unpublished manuscripts are somewhat less valuable sources.

Weather events and climate

Recorded climatic records, especially regular records kept by governmental agencies, are often of exceptional value to stream studies. Figure TS2–1 shows an example of available precipitation data for the Driftless Area in the Upper Midwest, along with some analyses (U.S. Weather Bureau data; Trimble and Lund 1982). Official climate records extend to the mid-19th century in the United States, but scattered records were collected earlier. Particular storms may have significant geomorphic effects locally or over larger areas, but little daily data are available. Cuts and fills in streambanks over time can also reflect the geomorphic history of a stream, where these data are available.

Stream and sediment discharge records—Stream discharge data for the United States extend to about 1850, are quite plentiful for this century, and most are easily available on the Internet. Suspended sediment data are available for certain streams, starting as early as about 1905. Web sites for various agencies with data on water quality and quantity are found in Ward and Trimble (2004) and described in NEH654.05.

Land use

Land use has been increasingly recognized as a major causal factor in stream change. Reconstructions of land use need to be as precise as possible because increasingly sophisticated information about the hydrologic and geomorphic effects of different land uses and treatments are available. Historical land use may be correlated with contemporaneous geomorphic phenomena in model building.

Agricultural census data may be complex and confusing because categories and definitions often change from one census to the next. For the United States census, county enumerations are for “land in farms” only and sometimes cover only fractional parts of counties. Where available, the census manuscripts,
Figure TS2–1  Precipitation time trends in the Driftless Area of the Upper Midwest, 1867–1974. The sigma notation denotes 1 standard deviation from the mean. (a) Number of weather stations; (b) Average annual precipitation and time trends, (c) Relation of annual precipitation and storms exceeding 2.5 inches in 24 hours; (d) 3-yr moving average annual precipitation.
rather than the published reports, give far more detailed information. Another major problem is that areas of enumeration units change with time, so that the boundaries and areas must also be reconstructed (Trimble 1974). Unfortunately, as yet there are no guides to the use of census data in reconstructing historic land use, and a great need clearly exists.

**Land use changes**

Although not specific to a stream’s local geomorphic condition, land use change information should be compiled. This can assist the stream restoration designer in ensuring that a proper analogue is selected for design work. If land use in a watershed has changed substantially, historic and even local geographic analogues may be inappropriate templates for design work. Changes in land use may be gradual, abrupt, or seemingly episodic over time, depending on climate fluctuations, population changes, and shifts in agricultural commodity markets, as illustrated in the example data set shown in figure TS2–2 (U.S. Census of Agriculture data, Trimble and Lund 1982).

**Land (cadstral) surveys**

While only planimetric land surveys can often supply important information to the fluvial geomorphologist. The original United States land surveys have been used to establish pre-agricultural flood plain conditions, upland vegetation, and stream widths. In other areas and in more recent periods, ongoing land resurveys sometimes give useful descriptions of geomorphological interest. Figure TS2–3 illustrates stream cross-sectional surveys and trends in watershed sediment delivery rates for the Coon Creek Basin in Wisconsin (Trimble and Lund 1982).

**Figure TS2–2** Changes of land use, Coon Creek, WI, 1850–1975

![Figure TS2–2](image-url)
Figure TS2–3  Sediment deposition rates based partially on topographic surveys at selected valley sites in Coon Creek Basin, WI, 1853–1977
Historic condition

A stream’s historic condition may be a useful target condition for physical restoration work if the causes of the degraded condition are local. The historic condition integrates many natural and human variables that controlled stream character at that past time. The historic condition can be an appropriate template for restoration design if these variables are relatively unchanged. Sources of information on a stream’s historic condition include photographs and maps, written references and reports, people with long-time knowledge of the area, highway and railroad bridge data, and field evidence (table TS2–1). The more accurate information that can be obtained about the historic stream planform pattern, longitudinal profile, and cross-sectional dimension, the greater the information can contribute to development of a template for restoration work. The information about the historic condition, in most cases, is of limited accuracy will probably contribute only a part of the information necessary to develop a restoration design.

Field evidence

Where a channel has been substantially modified from its historic condition (for example, channelized) or relocated by people, some field evidence of the historic stream condition may be preserved in the form of natural surface topographic features, soil patterns, and vegetation patterns. Surface topographic features may help to characterize the historic stream and include relict channel sections, cut banks, levees, stream terraces, and flood plain wetlands. Flood plain wetlands, or wetland soils indicative of the former presence of wetlands, may also help to determine the stream’s historic position and pattern. However, recent flood plain fill, alterations, or deposits may obscure remains of natural features. Riparian zone vegetation may obscure natural features, but leaf-off times would yield better results. Aerial photographs should be obtained to aid in locating these natural features if the stream reach to be restored is substantial in length.

Cultural features with documented locations may provide field evidence of the former position of the stream channel and bank prior to any changes in the stream’s depth, cross section, or location. Bridge support and streambank stabilization structures that were constructed in or immediately adjacent to the stream may provide particularly good information. In urbanized areas, gravity-fed sewer system pipes and manholes typically follow stream valleys. Exposure of subground portions of these features can provide dramatic evidence of stream position change, with exposure of subground instream cultural features being strong evidence of downcutting. Where numerous cultural features provide evidence of a stream’s former vertical and or horizontal position, the more likely that the watershed condition has changed substantially over time and the less likely that these features would be useful as historic analogues, since stream hydrologic condition may have changed substantially since their construction. Local cultural features may also have been the cause of current degraded conditions. If so, it is important to identify potential cultural features that could impact stream conditions and collect sufficient information to determine whether or not a stream is still responding to altered conditions caused by these constructed features.

<table>
<thead>
<tr>
<th>Table TS2–1</th>
<th>Sources of historic condition information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Source of historic condition information</td>
<td>Aerial photo</td>
</tr>
<tr>
<td>Planform pattern</td>
<td>✓</td>
</tr>
<tr>
<td>Channel dimension</td>
<td></td>
</tr>
<tr>
<td>Longitudinal profile</td>
<td></td>
</tr>
</tbody>
</table>

(210–VI–NEH, August 2007)
Litter

Litter includes mobile artifacts such as tools, vehicle parts, bottles, cans, package wrappers, and any other dateable artifacts. Although litter may be precisely dated in some cases, its location can only give an earliest possible date. For example, a can dateable to 1939 may have been dumped into a stream in 1950 where it was later buried in a point bar in 1952. The point bar may have eroded away in 1960 and the can buried 20 centimeters deep on a downstream flood plain. The only allowable conclusion is that there has been a minimum accretion of 20 centimeters at the final location since 1939. However, finding several items at similar levels with similar dates might allow a stronger inference. For example, an intact dump located in a flood plain with several items of similar dates could be valuable.

Permanent landscape features

Common, permanent landscape features including bridges, dams, mills, reservoirs, fords, fish traps, roads, canals, causeways, and buildings can sometimes act as gages to assist in measuring fluvial change.

**Bridges**—Inspection of older bridges by a practiced eye can often yield immediate information about stream processes. Reduced cross-sectional flow area under the bridge may indicate that the stream is aggrading. Burial of structural members (wingwalls) that are usually exposed to the stream may also indicate aggradation. Degrading streams, on the other hand, can often be diagnosed by old water lines left on structural members by exceptionally large openings and by the exposure of structural members (footings, pilings) which are usually placed beneath the water surface. Figure TS2–4 shows changes in stream morphology based on comparison of bridge plans with new survey data (Trimble 1970b).

---

**Figure TS2–4** Cross sections of Middle Oconee River at Highway 11 bridge, Jackson County, GA, showing stream aggradation between 1928 and 1969, based on bridge plans and resurvey

![Cross sections of Middle Oconee River at Highway 11 bridge](image-url)
Often of greater value are bridge plans. These usually include a stream and valley cross section surveyed before bridge construction that can be resurveyed for comparison. In comparing profiles, it must be ensured that the present bridge has not induced local scour or deposition, which would make comparison difficult or invalid. Often, plans include a detailed topographic map of the stream reach, and some plans may also include surveyed cross sections at some distance upstream or downstream. The latter surveys are particularly valuable because they are less affected by scour effects induced by some bridges.

Often, several bridges were built and rebuilt at the same site. Highway bridge plans in the United States often go back to the turn of the century and railroad plans to the mid- to late 19th century. Fortunately, the same vertical datum is normally used for successive bridges at a site, so that resurveys of century-old profiles are possible. Even when a datum changes, parts of the old bridge may be excavated to reestablish elevations.

Bridges are usually inspected periodically by government agencies. The resulting reports usually contain considerable description and measurements of site conditions and often include photographs that allow time-lapse photography.

*Dams, mills, reservoirs, fords, and fish traps*—Changes in streams often create severe problems in the operations of mills and reservoirs, and such problems may be documented. While nearly all water-powered mills had reservoirs, most of these were channel-type pools that had very low trap efficiency for sediment. More fortunately, some mills, and later hydroelectric and flood-control dams, have a large volumetric capacity in relation to their drainage area and, therefore, have a high trap efficiency so that sediment yield can be measured with some confidence. These data have the advantage of often being long term and include normally unmeasured sediment (bed load), but caution must be taken to consider the sediment from shoreline erosion. Many reservoirs are resurveyed periodically. These survey data can be obtained from government agencies or the people who surveyed the reservoir.

Figure TS2–5 shows a series of detailed surveys of sediment accumulated over time behind a gully-plug dam in Wisconsin (Trimble and Lund 1982).

Figure TS2–6 shows the variability in sediment production rates in watersheds in the Tennessee River Basin (Trimble and Carey 1992).

*Roads, canals, and causeways*—Roads (including railroads) and causeways serve as benchmarks to measure changes in stream morphology and process such as lateral movement of streams or gullies. When the location of a road or canal in relation to the stream can be determined from documentary evidence such as old maps or aerial photographs, its present location will allow an average rate of lateral migration to be calculated. Authorities will normally take whatever measures are necessary to protect a road or canal affected by a laterally migrating stream. Some sort of documentation is normally prepared, often with plans and maps. Structures put into place then act as benchmarks to measure future stream movement. Other road protection structures useful for future measurement are dikes, levees, and riprap. Additionally, roads are often raised by fill above normal flooding or aggrading flood plains, and the level of the old road beneath may be determined from construction plans, borings, or excavations.

*Buildings*—Except for occasional mills, buildings are rarely constructed on active flood plains and even when close to streams, they are usually sited on terraces. Thus, when a structure is affected by sediment, it indicates important changes in the stream or in sediment regime. Generally, when a building is significantly impacted by water and/or sediment, steps are taken to move or raise the building, if possible. If not, the building is usually dismantled, leaving only the foundation, which itself can serve as a benchmark. Once the foundation has been covered with sediment, however, the location must be established from old maps, land plats, eyewitness testimony, or even subsurface radar. Likewise, the chronology must be established from maps, land survey plats, tax records, or eyewitness accounts. Unlike roads and causeways, which may be located by borings, it is best to excavate around as much of the building as possible because it is necessary to see how the building’s occupants interfaced with the stream. For example, did an entrance face the stream? Artifacts between the building and the stream such as steps, walks, fences, and small outbuildings would imply that the area was frequented by people at one time, thus implying low frequency of flooding. Buildings also may be occasionally useful for measuring stream channel erosion.
Figure TS2–5  Headwater erosion rates based on sediment accumulation behind a small dam, Vernon County, WI, 1936–1977

Plan view

Explanation
- Old soil
- 1853–1936
- 1936–1977

Sediment accumulation 1936–1997
8527 m³@1440 mg/m³ = 12,325 mg

Profile views
- Profile number 1
- Profile number 2
- Profile number 3
- Profile number 4
- Station 2
- Station 3
- Station 4

Gully below dam
Borrow pit
Top of north bank
Spillway level
Approximate head of gully c. 1936

- Profile number 1
- Profile number 2
- Profile number 3
- Profile number 4

Old soil
1853–1936
1936–1977

- Station 2
- Station 3
- Station 4

0 5 10 15 20 25 feet
0 1 2 3 4 5 meters

A
A' B B'

0 100 200 300 400 500 feet
0 25 50 75 100 meters

1630 mg/km²/yr
1070 mg/km²/yr
132 mg/km²/yr
Figure TS2–6  Sediment yields for the Tennessee River Basin c.1940–1975, based on TVA reservoir surveys

Explanation
Values, in tons/mi²/yr

Churchill values
Brune values
Subbasin boundary
Basin boundary
Reservoir site

Reservoir capacity x10⁵ acre-ft

(210–VI–NEH, August 2007)
Historic maps

Historical maps can be an invaluable source of site-specific information to characterize past stream conditions. Historical maps are housed in many different collections, including libraries and historical societies, as well as local, state, and Federal government agency offices. For many areas of the United States, U.S. Geological Survey (USGS) maps are the oldest accurate maps available. USGS maps date from 1879 when systematic mapping of the country was begun in the West. Web sites of the USGS, National Archives, and Library of Congress are particularly valuable sources for locating older maps. The USGS Web site also contains information on mapping standards.

Information on past stream condition that can be derived from historic maps is limited by map scale, accuracy of original survey work, and climatic controls on stream character. In many areas, USGS topographic maps are often the largest scale historic maps available. Because USGS topographic maps are produced at infrequent intervals, only very long-term trends can be determined. This may prevent a detailed understanding of the effects of short-term physical processes and stream morphological responses.

Before the advent of aerial photography, streams were sketched in the field. The USGS generally mapped streams when water levels were at normal stage. The sketching of shorelines of broad rivers, however, was a perplexing problem due to periodic fluctuations in width. Riparian vegetation and other features also reduced map accuracy, depending on the date of observation. USGS map accuracy improved with the use of aerial photography beginning in the 1930s. Map accuracy further improved following establishment of national accuracy standards in the 1940s and the establishment of better horizontal control features in the 1950s. Since the 1940s, national map accuracy standard requires that 90 percent of defined test points are within 40 feet of their true horizontal position (large scale USGS maps, 1:24,000 scale). Older USGS maps were not subject to this standard. Currently proposed standards require that definite streams be depicted within 0.02 map inches (40 ft at 1:24,000 scale) of their horizontal position.

The mapped accuracy of stream width by the USGS also depends on mapping conventions related to stream width. Before 1954, USGS maps depict streams as double lined only when actual width could be displayed without exaggeration. In 1954, USGS adopted a standard whereby the minimum stream width required for depiction using double lines on a map was 40 feet for 7.5-minute maps and 80 feet for 15-minute maps. These criteria, however, had probably been widely used for several years beforehand. In 1993, the minimum width requirement for a stream to be depicted as a double line on 7.5-minute maps was increased to 50 feet. Streams narrower than this width criterion are depicted as single lines. On USGS maps from the 1800s through the 1950s, streams mapped as single line were depicted as tapering to become narrower towards the headwaters, and small side tributaries were depicted using a smaller weight single line than the mainstream. However, this was done to connote that the stream width decreases proceeding towards the headwaters, rather than to map specific stream widths. From the 1950s onward, all streams too narrow to meet the double-line width criterion are depicted as single bluelines of the same width, regardless of their actual width. Therefore, it is not possible to accurately determine the width of streams mapped as single lines on historic or current USGS maps.

In the vicinity of an engineered feature, such as a road or railroad, mapmakers often displace natural features slightly to allow for depiction of both engineered and natural features in a space on the map otherwise too small to permit both to be mapped to scale. Thus, this displacement possibility should be considered when using maps as sources of information about historic stream planform and width in vicinity of engineered features.

Positions of streams in arid regions are often relatively indefinite, as a consequence of infrequent flow conditions. In these regions, it would be inappropriate to scrutinize historic maps for specific channel location information.

Historic USGS maps can provide only limited information on elevational and longitudinal profile changes. National map accuracy standards established in the 1940s require that 90 percent of tested elevations on all USGS contour maps on all publication scales lie within a half the mapped contour interval. Currently proposed standards require that definite streams be depicted within a half contour interval of their verti-
cal position on USGS maps. In mountainous regions, the wide footage between contour intervals generally limits the ability to detect stream elevation and longitudinal profile changes, where substantial change has occurred. In flat areas, the great lateral distance between contour intervals typically limits the ability to accurately determine elevation changes over time. The range in potential change, however, is inherently much less.

Historic survey maps and associated notes can also be valuable sources of information. Surveys provide far greater detail than do regional maps. Streams were important resources to property owners, and creeks and rivers often form property boundaries, increasing the likelihood that valuable information on stream condition may be recorded in property surveys. Local governments, libraries, and local historical societies may also have historic property survey information. Surveys conducted in association with construction of structures over and along a stream by private individuals, commercial enterprises, and government agencies are of particular value. Government agencies involved in streamside projects potentially include highway departments, water supply and sanitary sewer agencies, the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS), U.S. Department of Interior (DOI) Bureau of Land Management (BLM), and USACE. The agencies involved in these past activities can be contacted for potential historic survey information that they have on file. Figure TS2–7 illustrates how the historical condition can be compared to the current condition, based on old land survey maps and information (Trimble 1970b). Because of stream aggradation, the area noted as good land in 1785 was swamp by the 20th century.

Figure TS2–7  Original land survey plat of the confluence of Sandy Creek with the North Oconee River, dated Jan. 8, 1785
**Instrumented topographic surveys**

While not available for many locations, topographic surveys may be the best quality data available. In this category are general topographic maps, which have existed in the United States for more than 100 years, created mostly by the USGS; precise river surveys, usually by the USACE; detailed maps of coastal areas including stream and estuaries with bathymetry, usually by the U.S. Coast and Geodetic Survey; stream stage-discharge studies by USGS and other agencies; and flood studies by various agencies.

**Aerial photographs**

The first aerial photographs available date from the 1800s when photos were taken from balloons and kites. Geographic coverage by these photos, however, is very limited. Aerial photography became widely practiced in the 1930s. The Library of Congress maintains a collection of aerial photographs taken between the early 1900s through the 1940s, and information on how to obtain these can be obtained at the Library of Congress Web site. Photographs taken by Federal agencies from the 1930s through 1940s for mapping purposes that cover approximately 80 percent of the area of the lower 48 states are available through the National Archives. Information on how to obtain copies of these photos is on the National Archives Web site. The USGS maintains a collection of aerial photographs taken since the 1940s. Information on obtaining copies of these photos can be obtained by visiting the USGS Web site. Streams located within cropland regions would also likely be included on aerial photography conducted by the USDA. Aerial photography dating from the 1950s is available from the USDA Aerial Photography Field Office. High quality aerial photographs are taken of nearly the entire country every 5 to 7 years by the National Aerial Photography Program. These photos are available from 1987 onward through the USGS.

Figure TS2–8 shows dramatic watershed land use changes and changes in streams and drainage patterns between 1934 and 1967 (Trimble and Lund 1982). Note the rectangular fields in the old system and the contour strip farming in the new. Note also the great decrease of drainage density (gullies) resulting from better land use and decreased overland flow.

Because of the great number of factors affecting the resolution of aerial photographs, there is no rule of thumb guiding the minimum linear dimension that can be resolved on historic aerial photographs. Major factors influencing aerial photograph resolution include atmospheric conditions, ground conditions, aircraft movement, lens character, film character, camera height (flying height), camera position with respect to the Earth’s surface, camera quality, and whether the film was black and white or color. For the USGS, camera height was determined primarily by the desire to compile contours accurately for mapmaking. If the height was appropriate for contour mapping, it was generally good enough to map planimetric features including streams. Historically, color film was much grainier than black and white, further limiting the resolution of historic color aerial photographs. Also, because of potential variation in scale across an aerial photograph due to distortion, information on historic stream condition is of greatest value when taken from a relatively small area of any given photograph.

Aerial photographs, dating from as early as 1917, have been used to demonstrate and date fluvial changes, along with the land use changes which were responsible for those stream changes. The general coverage of stereographic aerial photography in the United States dates from 1937 to 1938, but limited coverage exists from circa 1925. The value of aerial photography is a function of scale, photographic quality, and availability of stereographic coverage. Stream and valley aggradation is difficult to detect on air photos, but some attendant effects, such as the creation of backswamps and vegetational changes, can be seen and measured. On upland areas, air photos can be used to quantify land use and consequent erosion.

**Ground-based oblique photography**

Ground-based oblique photography has been used to date geomorphological processes dating back well into the 19th century. Figure TS2–9 shows an example of the use of such pictures taken at different dates. The 1940 photograph in figure TS2–9 shows a typical tributary in 1940. Note the eroded, shallow channel composed of gravel and cobbles, with coarse sediment deposited by overflows on the flood plain. Such tributaries were de-
Air photos showing changes in land use in Vernon County, WI, between 1934 and 1967.
scribed as resembling gravel roads. The 1974 photograph in figure TS2–9 is a remake of the 1940 photograph. The stream channel is narrower, smaller, and more stable. The coarse sediment has been covered with fine material, and the flood plain is vegetated to the edge of the stream. This condition has continued and improved over the past 30 years (Trimble and Crosson 2000). All data are from the U.S. Weather Bureau (Trimble and Lund 1982). Although not as systematically available as aerial photography, ground-based photography has existed longer and generally offers better scale and resolution for time-lapse comparisons. Many major repositories of such photographs exist, but queries should always be made to museums, libraries, and individuals. In some cases, photogrammetric techniques can also be used with oblique photography, making it possible to make precise measurements.

Photographs of site

Stream sites of interest by government or commercial interests following the advent of photography in the mid-1800s may have been captured in historic photos. Local residents may also have photographed the stream. These photos offer the advantage of potentially being relatively large in scale in comparison with maps and historic aerial photographs and can provide detailed local information to aid in interpreting changes in fluvial geomorphology over time. Historic photos may show local changes in depositional and erosional features in stream reaches and provide information on stream corridor character and human activities and land use. Unfortunately, locating these photographs can be difficult. Government agencies, local historical societies, and long-time area residents can aid in locating historic photographs. The Library of Congress maintains a national digital library from more than 100 historical collections that might also be worth reviewing.

Descriptive accounts and interviews with residents

Streams in long-settled areas or areas with major historic flooding events or other natural disasters impacting people may have been described in historic accounts. Local libraries can be good sources of this information. This information can be of particular use in cases where a stream condition is still evolving in response to a past disturbance, for which no obvious evidence is readily apparent. People familiar with the stream in the past, particularly long-term area residents that have a mental record of stream evolution, can often provide qualitative information on change in stream character.

Summary and conclusions

Although not always precise, historical data and techniques can provide powerful tools for establishing watershed conditions and stream forms and processes over the past few centuries, particularly during the last few decades. Comparative information can be used to determine the character of watershed and stream changes and may sometimes provide important quantitative measurements to support stream restoration designs.
Technical Supplement 3A

Stream Corridor Inventory and Assessment Techniques

(210–VI–NEH, August 2007)
Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.
# Technical Supplement 3A

## Stream Corridor Inventory and Assessment Techniques

<table>
<thead>
<tr>
<th>Introduction</th>
<th>TS3A–1</th>
</tr>
</thead>
</table>

<table>
<thead>
<tr>
<th>Tables</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Table TS3A–1</strong></td>
<td>Attributes of stream corridor assessment techniques</td>
<td>TS3A–2</td>
</tr>
<tr>
<td><strong>Table TS3A–2</strong></td>
<td>Summaries of site assessment and investigation techniques</td>
<td>TS3A–6</td>
</tr>
</tbody>
</table>
Technical Supplement 3A
Stream Corridor Inventory and Assessment Techniques

Introduction

This technical supplement contains an inventory of assessment techniques. This material was developed by an interdisciplinary team composed primarily of U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) employees (USDA NRCS 2001b). This guide provides the titles, reference citations, descriptive summary, and attributes of a number of stream corridor inventory and assessment techniques that are suitable for local conservation programs. Such programs are typically pursued at the site or project level, with increasing attention given to the landscape scale to optimize future treatments, management, and monitoring. The purpose of this guide is to aid in the selection of the appropriate inventory and assessment techniques to determine the conditions of their stream corridor. It is intended that this material possibly be used to supplement the information provided in this handbook.

The methods contained in this technical supplement are listed in table TS3A–1 along with their attributes. The table provides a description of principal features and a comparison of the techniques. Techniques are grouped by the primary stream corridor setting to which they pertain and are arranged in alphabetical order. Standard dictionary definitions for terms are assumed unless otherwise noted. Explanations of attribute ratings (columns 1–6 of table TS3A–1) are:

- **Primary Setting** that the particular technique addresses (many techniques are used for additional primary or secondary settings):
  - Channel—flood plain
  - Riparian area
  - Water quality (properties, contaminants)
  - Aquatic habitat

- **Sampling Intensity**:
  - Cursory (preliminary: observations and estimates of conditions and attributes are made usually without the need for specific measurements or quantification)
  - Detailed (comprehensive: conditions and attributes are itemized and specifically measured)

- The required **Skill Level, Training**, and **Time** to properly carry out the technique are rated as High (Skill level: specialists with considerable specialized expertise; Training: 3 to 5 days; Time: generally 4 or more hours per site), Medium (Skill level: specialists with basic specialized expertise; Training: 1 to 3 days; Time: generally 1 to 3 hours per site), or Low (Skill level: professionals or technicians trained in the technique; Training: 1 day or less; Time: usually less than 1 hour per site).

- The techniques are classification by **Kind** (Inventory—a collection of data or Assessment—a collection of data and value judgment as to condition), **Measure Type** (Qualitative—using charts, tables, attribute groupings or illustrations to classify or rate, or Quantitative—measurements, dimensions, quantities) and **Proximity** (Onsite—observers or data collectors physically at the site, or Remote—observers or data collectors can use satellite imagery or aerial photos).

- The need for a **Reference Site** (Yes, No, or Optional)—a reference site is a representative segment or reach of a stream corridor system in dynamic equilibrium with a relatively undisturbed watershed.

- The technique’s **Suitability for Monitoring** (High—suited for statistical analysis with consistent results between different collectors at the same site and accurate detection of change or trend over time, Medium—reproducible or repeatable results, but generally not suited for statistical analysis, or Low—not intended for monitoring purposes).

The ratings for the attributes in table TS3A–2 were developed by a team of interdisciplinary specialists with experience in stream corridor inventories and assessments. For each technique, a full citation, source address, and a brief summary are provided. Readers are encouraged to obtain and test the techniques that appear promising for their settings and requirements.
<table>
<thead>
<tr>
<th>Technique (to obtain a technique’s citation and summary, turn to the page number listed in parentheses)</th>
<th>Primary setting (listed first)</th>
<th>Sampling intensity</th>
<th>Skill level, training time</th>
<th>Kind, measure, proximity</th>
<th>Reference site needed</th>
<th>Suitability for monitoring</th>
</tr>
</thead>
</table>
Table TS3A–1  Attributes of stream corridor assessment techniques—Continued

<table>
<thead>
<tr>
<th>Technique (to obtain a technique's citation and summary, turn to the page number listed in parentheses)</th>
<th>Primary setting</th>
<th>Sampling intensity</th>
<th>Skill level, training</th>
<th>Kind, measure type, proximity</th>
<th>Reference site needed</th>
<th>Suitability for monitoring</th>
</tr>
</thead>
</table>
### Table TS3A–1  Attributes of stream corridor assessment techniques—Continued

<table>
<thead>
<tr>
<th>Primary setting—Riparian area—Continued</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
</table>

### Primary setting—Water quality

### Table TS3A–1  Attributes of stream corridor assessment techniques—Continued

<table>
<thead>
<tr>
<th>Primary setting—Water quality—Continued</th>
</tr>
</thead>
<tbody>
<tr>
<td>Technique (to obtain a technique's citation and summary, turn to the page number listed in parentheses)</td>
</tr>
</tbody>
</table>

Column notes:

1. Primary Setting (listed first): Channel flood plain, Riparian area, Water quality, Aquatic
2. Sampling intensity: Cursory, Detailed
3. Skill level, training, time (each rated as): High, Medium, Low
4. Kind: Inventory, Assessment, Measure type: Qualitative, Quantitative; Proximity: Onsite, Remote
5. Reference site required: Yes, No, Optional
6. Suitability for monitoring: High, Medium, Low
Table TS3A–2  Summaries of site assessment and investigation techniques


*Summary:* The survey's purpose is to help local stream teams determine vital signs of a river or stream, report immediate problems to proper authorities, and prioritize both short-term and long-range work. The water course is divided into reasonably sized segments that can be walked or canoed. Field data sheets include measurement of instream conditions, stream vegetation, streambank and corridor conditions, and presence of observable fish and wildlife species. Other data sheets include a summary sheet for a segment or reach survey, pipe survey, bridge survey, and wetlands survey.

**Agricultural Water Quality Index.** Robert B. Annis Water Resources Institute. Grand Valley State University, J. Cooper et al. 1998. WRI Publication #MR–98–1, One Campus Drive, Allendale, MI 49401. 75 p.

*Summary:* The Agricultural Water Quality Index (AWQI) is an assessment protocol that is specifically designed to evaluate the relationship between agricultural operations and water quality in agroecosystems. The AWQI is based on a series of assessments that can be examined separately and accumulated into a total score. Individual assessments include riparian zone metrics (width, completeness, vegetation types, summary), stream channel metrics (flow status, flow stability, channel sinuosity, channel structure, summary), and, optionally, a benthic macroinvertebrates metric (population diversity including indicator types). Specific recommendations for land and water management are associated with the ranked levels of individual metrics. Worksheets and scoring tables are provided.


*Summary:* The guide book includes fundamental principles of river behavior, a hierarchical stream inventory, a classification of natural rivers with illustrations, and data summaries and photographs depicting major stream types. The book contains field techniques and forms for:

- Stream classification of a reference reach
- Bank erosion prediction
- Fish habitat structure evaluation
- Sediment relations
- Hydraulics
- Channel stability evaluations


*Summary:* A classification of channel-reach morphology in mountain drainage basins synthesizes stream morphologies into seven distinct reach types: colluvial, bedrock, and five alluvial channel types (cascade, step pool, plane bed, pool riffle, and dune ripple). Coupling reach-level channel processes with the spatial arrangement of reach morphologies, their links to hillslope processes, and external forcing by confinement, riparian vegetation, and woody material defines a process-based framework within which to assess channel condition and response potential in mountain drainage basins. The classification is broadly applicable with its primary advantage of addressing the role of large woody material.
Table TS3A–2  Summaries of site assessment and investigation techniques—Continued

Summary: The handbook describes the standard inventory procedures for collecting fish habitat and salmonid fish species data for streams managed by the Northern Region (R1) and Intermountain Region (R4) of the Forest Service. The inventory defines the structure (pool/riffle and forming features), pattern (sequence and spacing) and dimensions (length, width, depth, area, and volume) of fish habitat; describes species composition, distribution, and relative abundance of salmonid species; and facilitates the calculation of summary statistics for habitat descriptors. The handbook is illustrated in color and includes data collection forms.

Summary: The guidebook provides the basis (or template) for applying the hydrogeomorphic (HGM) approach for specific physiographic regions for wetland functional assessment of riverine wetlands in context with the Clean Water Act Section 404 Regulatory Program. The concept of a reference standard is used, for example, conditions exhibited by a group of reference wetlands in a physiographic region that correspond to the highest level of functioning. Fifteen functions are identified for the riverine wetland class and are evaluated by an index computed using equations of selected variables from a group of 44 variables. Generic equations, detailed information, and field tally sheets are provided to document functions and develop models for a specific regional riverine subclass.

Summary: The original basis of the document was a report on the geomorphic characteristics of channelized streams in northern Mississippi to determine if their future behavior could be predicted. The publication contains a literature review on incised channels, historical information on subject channels, and discussion of geomorphic evolution of incised channels. The concept of entrenched streams is introduced in chapter 5 of the document including the hypothetical sequence of arroyo evolution. A summary of incised channels is listed in chapter 7, including a description of a possible evolutionary sequence.

Summary: The guide provides an integrated approach for: stratifying and classifying riparian areas according to their natural inherent characteristics, and their respective existing conditions; data collection; evaluation of riparian areas; future development and linkage of a riparian data base; preparation of a written narrative to interpret the data and suggest management applications; providing a process to prioritize or rank riparian areas based on management objectives; strengthening the riparian management implications of the Forest Land Management Plan. The approach is split into threes levels: level I is an office procedure, level II is a field procedure, and level III is a more quantitative, site-specific field data collection. Levels are progressive and should be completed in order. The guide includes data collection forms.

Summary: The report compiles a comprehensive set of methods for resource specialists to use in managing, evaluating, and monitoring riparian conditions adjacent to streams, lakes, ponds, and reservoirs with an emphasis on streams. Issues of sampling kind and intensity, accuracy, and precision are described. Detailed procedures are given for measuring vegetation, classifying riparian communities and soils, using remote sensing, measuring water column attributes, detecting streambank morphology and alteration, mapping woody material, using benthic macroinvertebrates, and evaluating historic riparian habitats. Emphasis is on procedural details, rather than reliance on predefined data collection forms.


Summary: The document describes a monitoring system to assess grazing impacts on water quality in streams of the western United States. Methods described are reportedly easy to use and cost-effective (reduced sampling frequency, limited need for specialized equipment, and limited laboratory analyses). The protocols focus on attributes of the stream channel, streambank, and streamside vegetation (characteristics are sampled during low-flow summer conditions). Methodology requires an interdisciplinary team. Explanatory illustrations and various field data collection forms are included.


Summary: The manual and handbook contain detailed procedures for completing vegetation field forms and ecological sites. The National Forestry Manual is applicable to stream riparian areas that are currently forested or have a potential for a plant community dominated by woody plants (trees) with a height potential of at least 4 meters. The National Range and Pasture Handbook is applicable to stream riparian areas that are currently in herbaceous or shrub vegetation or have a potential for a plant community dominated by herbaceous or shrub species. Detailed instructions, coding conventions, and data collection forms are provided in both the manual and handbook. Collected field data and information may be entered into a national database maintained and supported by the NRCS.


Summary: This technique is a single page form that permits the user to record major attributes of a representative segment of a stream reach. It was developed for use with private landowners to focus attention on the existing conditions of their streams. Basic stream attributes (stream order, depth, width, gradient, entrenchment), soil conditions (bank erosion frequency, bed load fine sediments, upper bank compaction), water conditions (turbidity, presence of algae, color, temperature), plants (potential native vegetation, present vegetation, dominant terrestrial plants, aquatic species), air condition, animals (fish species, aquatic macroinvertebrates, land species), and human use attributes are collected.

Summary: The assessment procedure can be used on a broad reach or site-specific scale. Values that are entered on the data sheet can be estimated or measured. The intended use is for planning, baseline data, monitoring, and evaluating restoration alternatives. The procedure is not intended to replace intensive surveys conducted by professional biologists. Users of the procedure are encouraged to complete the watershed overview sheet before the habitat data sheet. The data sheet accommodates entries to identify the site, substrate composition, and bank vegetation. A series of criteria tables are used to assess and score stream habitat condition.


Summary: The document defines protocols and procedures for evaluating streamside vegetation and streambank stability for Idaho’s small (usually less than 30 feet wide) rangeland streams. It also provides protocols for monitoring stream canopy cover, streambank stability, solar input, and establishing permanent photo points associated with livestock grazing and other activities that affect streamside vegetation and beneficial uses of water. The protocols are directed at three important pollutant sources affecting the biological integrity of streams and lakes that may result from livestock grazing: streambank erosion, water temperature, and vegetation.

Qualitative Habitat Evaluation Index [QHEI]: Rationale, Methods, and Application. State of Ohio Environmental Protection Agency. 1989. Edward T. Rankin, Ecological Assessment Section, P.O. Box 1049, 1800 WaterMark Dr., Columbus, OH 43266–0149. 51p.

Summary: The index is designed to provide a measure of habitat generally corresponding to those physical factors that affect fish communities and which are generally important to other aquatic life, such as invertebrates. The field sheet for the QHEI consists of qualitative descriptors that are checked as appropriate. Highest scores are assigned to the habitat parameters that have been shown to be correlated with streams having high biological diversity and integrity, with progressively lower scores assigned to less desirable habitat features. Individual scores are provided for the habitat components of substrate, instream cover, riparian zone and bank erosion, pool/glide quality, riffle/run quality, and gradient. A total score of 100 is possible.


Summary: The assessment was developed to have a tool more applicable to streams in Arizona than those currently being used throughout the West. The technique addresses riparian area classification, channel geomorphology, riparian functional analysis procedure, and riparian monitoring with photography. The objective of the developers was to collect quantitative field data to document and defend functional interpretations. The Tonto National Forest approach (Tonto Riparian Inventory and Monitoring Methods or TRIMM) was the working model for developing the assessment. The Arizona Game and Fish Department can be contacted for the final report and assessment procedure.
Table TS3A–2  Summaries of site assessment and investigation techniques—Continued


Summary: The document provides states with a practical technical reference for conducting cost-effective biological assessments of lotic systems. The protocols were designed as inexpensive screening tools to determine if a stream is supporting or not supporting a designated aquatic life use. They may also be appropriate for priority setting, point and nonpoint-source evaluations, use attainability analyses and trend monitoring. Worksheets are included. The protocols must be locally adapted and scaled.


Summary: The protocol is a synthesis of several techniques with applicability to nonlimestone Piedmont streams with drainage areas less than 150 square miles. RSAT employs both a reference stream and an integrated numerical scoring and verbal ranking approach. Evaluation categories include: channel stability, channel scouring/sediment deposition, physical instream habitat, water quality, riparian habitat conditions, and biological indicators (macroinvertebrates). Parameters are measured at approximately 400-foot intervals along the stream. Data is first recorded via field survey sheets and later transferred into a spreadsheet database.


Summary: The guide establishes a method for evaluating the condition of riparian-wetland lotic areas and classifying segments or reaches of streams into proper functioning condition (PFC), functional at risk, nonfunctional, and unknown categories. The qualitative, yet science-based process, considers both abiotic and biotic factors as they relate to physical function. A standard checklist of 17 key questions is provided and enables users to determine the functional condition of a stream reach or segment. PFC must be conducted by an interdisciplinary team trained and familiar with the local conditions being assessed. The supporting science and related quantitative methodologies for each of the 17 questions are provided.


Summary: The technical reference gives the detailed procedure for the greenline monitoring method. Greenline is a term used to essentially identify nearest-to-stream continuous riparian plant community types using a line intercept transect running parallel to the stream. It is a procedure that is both repeatable for monitoring purposes and a point of reference which minimizes problems associated with changing moisture gradient. Data collection forms are included. (Note: As of the date of this report, the Forest Service is in the process of updating the greenline methodology with plans to republish the technique as a Forest Service technical publication.)
Table TS3A–2  Summaries of site assessment and investigation techniques—Continued


Summary: The technical reference contains suggested techniques and procedures for performing an extensive inventory and, if warranted, an intensive inventory. Extensive components include drainage pattern, landform, soils information, channel form and condition, vegetation types and ecological sites, flood plain characteristics, and other attributes. Intensive components include detail soil characteristics and properties, channel parameters, vegetation identification and structure, woody species characteristics, and other attributes. A section on monitoring is integrated in the technical reference. Inventory forms are included.


Summary: The technical reference provides detailed field procedures for describing and documenting riparian-wetland ecological sites (potential vegetation) which are a function of and defined by the interaction of soils, climate, hydrology, and vegetation at riparian-wetland sites. The document contains a standard site field review checklist, site correlation checklist, standard site description, and a completed, sample standard site description. The technical reference is intended for use with the National Range and Pasture Handbook, the National Forestry Manual, and the National Soil Survey Handbook available from the USDA, Natural Resources Conservation Service, P.O. Box 2890, Washington, DC 20013.


Summary: The document provides a procedure for using aerial photography to answer proper functioning condition checklist items. It supplements TR1737–15, Riparian area management: A user guide to assessing proper functioning condition and the supporting science for lotic areas. The technical release gives the detailed procedure for gathering existing source material, analyzing equipment needs, defining reaches and areas, interpreting aerial photos, and verifying interpretations in the field. Also included are specific recommendations pertaining to needed aerial photo qualities, photo interpretation examples, and the results of large area case studies in Montana.


Summary: This supplement is part of the Federal guide developed to help resource managers implement direction in the record of decision (ROD) for amendments to Forest Service and Bureau of Land Management planning documents within the range of the Northern Spotted Owl. The ROD requires watershed analysis prior to the final delineation and management of the riparian reserve network in a watershed. The riparian analysis process is divided into two levels based on anticipated activities: level 1—geared toward small effects along intermittent streams, and level 2—addresses larger magnitude effects.
Table TS3A–2  
Summaries of site assessment and investigation techniques—Continued

Summary: The paper describes an approach to initial site selection in the San Luis Rey River watershed in southern California that uses watershed-level information on basin topography and land cover to rank the potential suitability of all sites within a watershed for either preservation or restoration. The approach requires the use of a geographic information system (GIS) to map relative wetness and land cover within a watershed. Relative potential wetness values were derived from USGS 30-meter digital elevation models; land cover was derived from a Landsat scene covering the 1,500 square kilometers study area. The paper is illustrated with color diagrams and pictures.

Summary: The assessment is a method for rapidly addressing a lotic site’s overall health or condition. It provides a site rating useful for setting management priorities and stratifying riparian sites for remedial action or more rigorous analytical attention. It is intended to serve as a first approximation, or coarse filter, by which to identify lotic wetlands in need of closer attention so that managers can more efficiently concentrate effort. The term riparian health is used to mean the ability of a riparian reach (including the riparian area and its channel) to perform certain functions. These functions include sediment trapping, bank building and maintenance, water storage, aquifer recharge, flow energy dissipation, maintenance of biotic diversity, and primary production. Stream Channel Reference Sites: An Illustrated Guide to Field Technique. USDA Forest Service. General Technical Report RM–245. C. Harrelson et al. 1994. Rocky Mountain Forest and Range Experiment Station, Fort Collins, CO. 61p.  
Summary: The guide helps users establish permanent reference sites. The minimum procedure consists of: select a site, map the site and location, measure the channel cross section, survey a longitudinal profile of the channel, measure stream flow, measure bed material, and permanently file the information with the Vigil Network. The document includes basic surveying techniques and provides guidelines for identifying bankfull indicators and measuring other important stream characteristics. The object is to establish the baseline of existing physical conditions for the stream channel. The guide is amply illustrated with diagrams and black and white pictures.

Stream Corridor Assessment Survey. Maryland Department of Natural Resources. 2000 (revised draft). K. Yetman, Watershed Restoration Division, Chesapeake and Coastal Watershed Services, Annapolis, MD 21401. 100+p.  
Summary: The survey protocols help users identify environmental problems and prioritize restoration opportunities that exist within Maryland watersheds. The assessment is designed to be done by small teams of well-trained volunteers who walk 2 or more stream miles per day. Potential environmental problems identified during a survey include channelized stream sections, streambank erosion, exposed pipes, inadequate stream buffers, fish blockages, trash dumping sites, near stream construction, pipe outfalls, and general conditions of instream and riparian habitat. In conjunction with the AmeriCorp program, more than 700 miles of Maryland streams have been surveyed using the assessment protocols. This has led to more than $1 million of restoration work to date. One Maryland county has included the assessment as part of the NPDES permit system for municipal stormwater discharges.
Table TS3A–2  Summaries of site assessment and investigation techniques—Continued


**Summary:** The handbook provides standards for a level I (office inventory) and level II (field inventory) of stream systems. The protocol identifies core attributes necessary to evaluate the condition of a stream. It contains instructions and data forms for stream habitat conditions (flow, water quality, historical land use, valley-channel parameters, streambed substrate, flood-prone dimensions, and riparian habitat dimensions). Other data forms are included for inventorying culverts, falls, chutes, dams, marshes, braids, and fish species.


**Summary:** The guide provides methods for obtaining a holistic picture of a stream's watershed, as well as collecting detailed information. The techniques presented in the guide are fairly simple, inexpensive, and can be accomplished with readily available equipment. Readers not only learn how to evaluate the physical and biological characteristics of streams using the latest quality control and quality assurance planning techniques but can also study a chapter devoted to presenting field data to a wide range of audiences. The section called Streamkeeper Tales includes inspirational examples of volunteers who have used their field data as the basis for protecting and restoring streams. The active voice of the text and the large number of humorous technical illustrations accompanied by poignant editorial cartoons make this book engaging to volunteers and scientists alike.


**Summary:** The document provides guidance to the user of the Stream Network Temperature Model (SNTEMP). Planning, executing, and using the results from a stream temperature modeling study are described. Details of field data gathering, instrumentation, and data collection priorities are given for the range of stream geometry, meteorology, and hydrology components necessary for the model's application. Each input variable is defined, and its relative data collection effort is approached from the perspective of sensitivity in predicting stream temperatures. Alternative public domain stream and reservoir temperature models and techniques are also described.


**Summary:** The assessment protocol provides a basic level of stream health valuation based primarily on physical conditions for a stream reach. It is intended to be conducted with the landowner and incorporates talking points for planners to use during an assessment. Assessment elements, which receive a numerical rating based on observations and some rapid measurements, include: channel condition, hydrologic alteration, riparian zone, bank stability, water appearance, nutrient enrichment, barriers to fish movement, instream fish cover, pools, invertebrate habitat, canopy cover, manure presence, salinity, riffle embeddedness, and macroinvertebrates observed. Rating criteria and worksheets are included. The protocol works best if locally modified.
Table TS3A–2  Summaries of site assessment and investigation techniques—Continued

Summary: The paper outlines the procedure for mapping riparian and other small areas which were traditionally identified by spot symbols on soil survey maps. Riparian areas are typically very linear and are more difficult to map and display than upland soil polygons. Certain soils, hydrology, and vegetation criteria must be met for an area to be identified and mapped as a riparian area. Cartographic procedures for delineating point and line features are included. Examples of soil map unit descriptions and a sample soils map are provided.

Summary: Underwater observation with snorkeling gear is a valuable tool for studying fish populations and assessing how fish use habitat in flowing waters. Precise estimates of fish abundance can be obtained using underwater counts. However, several factors, including the behavior of the target fish species and attributes of the physical habitat (stream size, water clarity, temperature, cover), can bias results. This report was developed to assist biologists in identifying and accounting for potential biases and to encourage a standardized procedure for the use of underwater techniques to survey salmonids in streams. The guide addresses the principal resident and anadromous salmonids found in the Intermountain West (Idaho, Montana, Nevada, Utah, and western Wyoming). Color illustrations and pen and ink drawings of target fish are included.

Summary: The guide examines five major sources of agriculturally related nonpoint source pollution: sediment, nutrients, animal waste, pesticides, and salts. Field sheets are provided to enable the user to observe and record surface water quality problems and to select appropriate remedial practices. Field sheets are arranged in matrix format with environmental indicators given for each of the five major pollutant types. Each indicator is divided into descriptions of the environment from excellent to poor with each description given a weighted numerical ranking. There are two types of field sheets: one for receiving waters and one for the lands that drain into receiving waters.

Summary: The publication consists of a worksheet and action plan developed for use by landowners having a stream or stream systems on their property. The worksheet’s 15 questions direct the user to all aspects of stream corridor condition. The action plan correlates individual answers from the worksheet to helpful notes and contact agencies and addresses for further investigation. The assessment system is voluntary, useful for a first approximation of stream corridor conditions, and alerts the landowner of possible concerns.
Using Aerial Videography and GIS for Stream Channel Stabilization in the Deep Loess Region of Western Iowa
Cover photo: Channel stabilization in western Iowa begins with an understanding of streambed gradients and their natural controls and causes of instability. Channel degradation or incision is widespread in this area. Soils in the deep loess region of the Mississippi River Basin are some of the most highly erosive in the country.

Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.
# Using Aerial Videography and GIS for Stream Channel Stabilization in the Deep Loess Region of Western Iowa

## Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abstract</td>
<td>TS3B–1</td>
</tr>
<tr>
<td>Introduction</td>
<td>TS3B–1</td>
</tr>
<tr>
<td>Previous research</td>
<td>TS3B–3</td>
</tr>
<tr>
<td>Methods</td>
<td>TS3B–3</td>
</tr>
<tr>
<td>Results</td>
<td>TS3B–5</td>
</tr>
<tr>
<td>Conclusion</td>
<td>TS3B–5</td>
</tr>
</tbody>
</table>

## Figures

<table>
<thead>
<tr>
<th>Figure TS3B–1</th>
<th>Straightened versus meandering stream (Walnut Creek, Pottawattamie County, IA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS3B–2</td>
<td>Bridge endangered by exhumation of pilings</td>
</tr>
<tr>
<td>Figure TS3B–3</td>
<td>Nickpoint eroding headward</td>
</tr>
<tr>
<td>Figure TS3B–4</td>
<td>NRCS bank stabilization site that did not include streambed stabilization (Page County, IA)</td>
</tr>
<tr>
<td>Figure TS3B–5</td>
<td>HCA steel sheet pile weir with riprap and cement grout that protects a large sewer main for the City of Denison, IA</td>
</tr>
<tr>
<td>Figure TS3B–6</td>
<td>Six-stage channel evolution model</td>
</tr>
<tr>
<td>Figure TS3B–7</td>
<td>Stream reaches flown and classified in 2002 and 2003, grade control structure locations, loess depth, and 22 western Iowa counties in the HCA</td>
</tr>
<tr>
<td>Figure TS3B–8</td>
<td>Comparison of two classified stream reaches on Graybill and Jordan Creeks in 1994 and 2002</td>
</tr>
</tbody>
</table>
Technical Supplement 3B

Using Aerial Videography and GIS for Stream Channel Stabilization in the Deep Loess Region of Western Iowa

Abstract

This technical supplement presents an example of using an aerial assessment and classification of streams in the deep loess region of western Iowa. This is a large-scale effort directed by the Hungry Canyons Alliance involving approximately 5,250 miles of stream in 22 western Iowa counties. This effort has determined areas of active stream erosion and the impact streambed stabilization structures have had on controlling stream degradation. The stream assessment consisted of flying along streams in a small helicopter videotaping the stream channel and recording positions with a global positioning system receiver. Streams were classified based on a six-stage channel evolution model to describe the dominant channel processes occurring along stream reaches. Streams across the region in 1993 and 1994, and in a smaller area in 2000, were similarly classified.

The comparison of the recent classification to those of the past has allowed the researchers in this study to describe how stream stabilization structures have impacted the streams and make predictions as to where future stream erosion will occur. The data has also been used to mathematically model channel evolution in the region.

The purpose of Hungry Canyons Alliance (HCA) is to focus attention on the problems and develop solutions related to stream channel degradation in 22 counties of western Iowa with deep loess soils. The HCA provides cost share to counties to build streambed stabilization structures, building about 20 structures per year. To date, HCA structures protect 195 bridges; numerous utility lines including electric, phone, gas, sewer, water lines; and an estimated 830 acres (336 ha) of farmland, equivalent to stopping 11.3 million tons (10.25 million metric tons) of sediment from being swept away into the Missouri River and Mississippi River systems.

However, HCA members need to know where the areas of active stream erosion are to wisely locate structure sites. To do this, an aerial assessment of streams in western Iowa was performed, and streams were classified based on the channel evolution model.

Introduction

In western Iowa, 22 counties contain deposits of loess ranging from 13 feet (3.96 m) to more than 200 feet (60.96 m) deep. Loess is highly erodible and susceptible to stream channel erosion and degradation. It is estimated that stream channel downcutting and widening in western Iowa has caused $1.1 billion in damage to bridge and utility infrastructure (Hadish et al. 1994). Stream degradation has also eroded thousands of acres of valuable farmland, increased stream sediment loads, and decreased water quality.

Stream channel dredging, straightening, and land use changes since the early 1900s have caused many problems. After straightening, the streambed is steeper because the stream still had the same amount of fall per mile, yet there was now a shorter distance over which that fall occurred. The steeper slope increased water velocities. To reduce the water velocity, streams either had to meander or downcut. Due to the high erodibility of loess soils, the streams began downcutting, causing accelerated soil erosion in western Iowa (fig. TS3B–1).

Figure TS3B–1  Straightened versus meandering stream (Walnut Creek, Pottawattamie County, IA)
As the streambed downcuts, it destabilizes bridge pilings (fig. TS3B–2), and the vertical streambanks will slump to a more stable slope, in effect widening the stream and necessitating longer bridges.

Nickpoints, which are naturally occurring overfalls (fig. TS3B–3), will continue to erode and advance upstream, eventually affecting the entire watershed. Bed degradation of stream channels will force the channel’s tributaries to also adjust to the lowered base level, often initiating gully formation where no channel had previously existed.

Streambed stabilization is the key to preventing further erosion and protecting infrastructure (fig. TS3B–4). After the stream downcut 6 feet in 7 years, the bank stabilization was left high and dry. Dams or weirs at regular intervals will help streams stabilize by changing their profile from a steep incline to a stable stair-step pattern. Structures normally have a raised weir section, like a low-head dam. Most structures use steel sheet pile driven into the streambed 20 to 25 feet (6.1–7.6 m) and riprap to protect the banks (fig. TS3B–5).
Streambed stabilization structures have many benefits. They decrease the slope of the streambed, reducing water velocities. They prevent further downcutting, protecting farmland, bridge pilings, and utility lines from future erosion. They create an upstream backwater condition which allows sediment to settle out, reducing sediment loads and improving water quality. The backed-up water often helps to protect bridge pilings by submerging them.

Previous research

A six-stage channel evolution model (fig. TS3B–6, developed by Simon and Hupp (1986), was used to describe the dominant channel processes occurring along stream reaches.

An aerial reconnaissance of streams in 18 western Iowa counties was conducted in 1993 and 1994 by the HCA. They were flown during the spring to prevent tree cover and other vegetation and snow and ice from obscuring the view of the channels (Hadish 1997). The stream channels were recorded through the door of a low-flying helicopter by a hand-held video camera while a microphone recorded a narration onto video tapes. This narration described location information and other observations. The video focused on the streambed, streambanks, and the flood plain, along with other features like gullies, nickpoints, and grade control structures.

In conjunction with a set of manually transcribed notes, the videotape and narration was used to classify streams by Simon’s six-stage channel evolution model. Classified stream reach locations were transferred to USGS 1:100,000 scale topographic base maps by hand. These hard copy maps were then converted to a digital data set using the Geographic Resources Analysis Support System (GRASS) geographic information system (GIS) software package. GRASS GIS is an open source free software GIS that is largely command line driven. In 2000, the NRCS reclassified streams in four counties and classified streams in two additional counties using the same process.

In the 1993 to 1994 study, 107 streams covering 1,540 miles (2,478 km) were videotaped. More than 90 percent of the study area at that time was unstable, in stages 2, 3, 4, and 5, with the greatest amount (55.9 percent) in stage 4 (Hadish et al. 1994). Stage 5 was concentrated in the downstream portions of larger drainages, whereas stage 3 reaches occurred in the upper reaches of main streams and on small tributaries. The data supported the notion of a recovery process, described by the channel evolution model, where degradation progresses from the lower to the upper stream reaches followed by aggradation (Hadish 1997).

Methods

Many issues led the HCA to undertake another stream classification. First, the 1993 to 1994 data was largely outdated and not very helpful in streambed stabilization project planning. Second, it is still important to determine where streams are actively eroding and make predictions as to where future erosion will occur. Third, the HCA wanted to determine the impact of streambed stabilization structures on stream degradation. Fourth, with two sets of data, channel evolution in the region can be modeled by comparing the progression of erosion between flights. Finally, current stream videos and maps can be used by county engineers and NRCS district offices for other purposes such as county infrastructure inspections, land use planning and zoning, and conservation practice evaluations.

To update the stream classification, the decision was made to videotape those streams that were previously flown, had grade control structures on them, had drainage areas of more than 10 square miles, or were known to be experiencing streambed degradation. The stream classification began in the spring of 2002 and continued during the fall of 2002 and spring of 2003, when flying ended due to foliage or inclement weather.

The same basic process was again used to classify streams, but there were significant changes due to technological advances. Videotaping was done through the open door of a small helicopter with a hand-held digital video camera while a global positioning system (GPS) receiver recorded the position. The Video Mapping System (VMS), engineered by Red Hen Systems of Fort Collins, Colorado, integrates video and still images with simultaneous location information recorded by the GPS. The GPS signal is stored on one of the audio tracks of the videocassette. For added error
Stage 1: Includes streams that have not been modified and tend to be very stable. These streams tend to meander across their flood plain and have a dense vegetative cover on the banks down to the low-flow line of the channel.

Stage 2: Associated with streams recently modified or channeled by construction. Degradation at this stage depends upon such characteristics as streambed slope, angle of the banks, and cross-sectional area.

Stage 3: Degradation occurs most rapidly in the streambed. The increased channel slope increases the flow velocity, which causes the channel to deepen and its bank slopes to become steeper. As the channel downcuts, the toe of bank slopes are undercut, and bank failures will occur. Full-scale degradation takes place during this stage.

Stage 4: Evident by channel widening. Mass erosion of bank soils and vegetation is predominant, creating a scalloped appearance in the banks. Streambed degradation slows during this stage.

Stage 5: The bed of the channel begins to stabilize; bank failures decrease and re-vegetation occurs. The height and slope of the banks is reduced.

Stage 6: Channels show re-vegetation as it becomes increasingly stable and bank widening stops. Vegetation may extend in a dense cover up the sideslopes.
reduction, in case the GPS unit stopped working, notations on county maps and a journal were kept to note locations, dates visited, and keyed to the corresponding videocassettes on which that information had been recorded. The video was brought to the office where it was indexed by VMS, which associates a segment of videotape with the recorded GPS locations and translates this information between the GPS receiver, camcorder, and computer.

Once a tape was indexed, it was ready for classification, where attributes such as the six-stage channel evolution model were added to the information collected by GPS by examining the videotaped segment and applying a value to each stream reach. When that process is completed, the data are then exported out of the VMS software package as an ESRI formatted shapefile. The shapefiles were then brought into ESRI's ArcGIS, a menu and button driven software package, where further data manipulation such as overlaying the data with previously collected data sets and other spatially referenced information was performed.

One of the finished products that will be important to the overall impact of the project is the creation of DVDs for distribution to project partners. After the videotape was analyzed, a set of DVDs showing streams in their county was made available to county engineers and NRCS district offices to use in stream channel stabilization project planning.

Results

The stream reaches that were flown in the spring and fall of 2002 and the spring of 2003 are shown in figure TS3B–7. Also shown in figure TS3B–7 is the 22-county HCA region, loess thickness, and locations of known grade control structures (207 HCA, 300 EWP, 6 Iowa DOT, and 39 landowner-installed grade control structures).

To determine the progression of erosion through time and the impact streambed stabilization structures have had on controlling stream degradation, two segments of stream that were classified in 1994 and again in 2002 are shown for comparison in figure TS3B–8. In 1994, Graybill and Jordan Creeks in Pottawattamie County were predominately stage 4, with stage 3 farther upstream in the watersheds. Only two structures had been built on these streams at that time. The arrow points to a bridge where 23 feet (7 m) of degradation was recorded between 1972 and 1993. By 2002, 14 additional structures had been built on these two creeks. The streams have become less erosive and more stable, with the average stream at stage 5.

Degradation has migrated about 5 miles (8 km) upstream with stages 3 and 4 occurring only near the headwaters of the streams.

Conclusion

The use of aerial videography and GIS has proven to be a rapid and effective technique for rapidly assessing streams.

In the subject study, degradation does not appear to extend into areas where there is no loess soil cover, that is, beyond the loess-till boundary (line of 0 loess depth) in the northern half of western Iowa. Degradation, particularly gully erosion, becomes more pronounced in areas where loess soils are thickest.

Most large streams (those streams with a watershed greater than 70 square miles (181 km²)) are no longer degrading, but aggrading. Many show evidence of deep incision in the past; however, they have filled with sediment eroded from upstream as degradation has moved upstream. This recovery process was also noted by Hadish (1997). The likelihood of active degradation becomes greater with distance northeast away from the Missouri River because many streams near the river experienced degradation first as degradation has migrated upstream. They have since become stabilized by the influx of sediment from upstream. Degradation only occurs where streams have been channelized downstream. The closer a stream reach is to the original channelization, the greater the degradation that has occurred on that stream reach.

Grade control structures appear to have helped stabilize large stretches of stream. Streambeds up to 1 mile (1.6 km) upstream from the structures have been stabilized (fig. TS3B–8). For example, the rapid stabilization of Graybill and Jordan Creeks is largely due to the construction of grade control structures at regular intervals.
Figure TS3B–7 Stream reaches flown and classified in 2002 and 2003, grade control structure locations, loess depth, and 22 western Iowa counties in the HCA

Loess depth, stream reaches flown, and structure locations

Loess depth in feet
- 0
- 15
- 20
- 25
- 35
- 50
- 75
- 100

Streams flown (spr. 02, fall 02, spr 03)
To be flown
- HC county structures
- HC landowner structures
- EWP structures
- DOT structures
- HC counties

Only an example, not accurate
Figure TS3B–8  Comparison of two classified stream reaches on Graybill and Jordan Creeks in 1994 and 2002

Loess depth in feet
- Not classified
- Stage 1
- Stage 2
- Stage 3
- Stage 4
- Stage 5
- Stage 5i
- Stage 6
- Stage 6i

Structures
- Gas pipeline
- HCA county
- HCA landowner
- EWP - NRCS

- Other streams
- Highways
- Roads
- City or town
- County line

1994 Pottawattamie County

2002 Pottawattamie County

Okland
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

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Abstract</strong></td>
<td>TS3C–1</td>
</tr>
<tr>
<td><strong>Problem identification and trend analysis</strong></td>
<td>TS3C–1</td>
</tr>
<tr>
<td>Watershedwide problems</td>
<td>TS3C–1</td>
</tr>
<tr>
<td>Channel problems</td>
<td>TS3C–2</td>
</tr>
<tr>
<td><strong>Procedures for streambank investigations and analysis</strong></td>
<td>TS3C–2</td>
</tr>
<tr>
<td>Geomorphic values</td>
<td>TS3C–4</td>
</tr>
<tr>
<td>Background data collection (prior to field visit)</td>
<td>TS3C–4</td>
</tr>
<tr>
<td>Field data collection</td>
<td>TS3C–8</td>
</tr>
<tr>
<td>Data analysis and assessment</td>
<td>TS3C–9</td>
</tr>
<tr>
<td>Departure analysis</td>
<td>TS3C–11</td>
</tr>
<tr>
<td><strong>I&amp;E spreadsheet details</strong></td>
<td>TS3C–12</td>
</tr>
</tbody>
</table>

## Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>TS3C–1</th>
<th>Lane’s Balance for determining the effect of human activity on streams</th>
<th>TS3C–2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure</td>
<td>TS3C–2</td>
<td>Channel evolution model</td>
<td>TS3C–3</td>
</tr>
<tr>
<td>Figure</td>
<td>TS3C–3</td>
<td>Regional curves showing bankfull dimensions by drainage area</td>
<td>TS3C–6</td>
</tr>
<tr>
<td>Figure</td>
<td>TS3C–4</td>
<td>Valley slope subroutine from stream stabilization spreadsheet</td>
<td>TS3C–7</td>
</tr>
<tr>
<td>Figure</td>
<td>TS3C–5</td>
<td>Bankfull indicators used for field identification</td>
<td>TS3C–8</td>
</tr>
<tr>
<td>Figure</td>
<td>TS3C–6</td>
<td>Cross-sectional subroutine from stream stabilization spreadsheet</td>
<td>TS3C–10</td>
</tr>
<tr>
<td>Figure</td>
<td>TS3C–7</td>
<td>Typical stream morphology illustrating radius of curvature</td>
<td>TS3C–11</td>
</tr>
<tr>
<td>Figure</td>
<td>TS3C–8</td>
<td>Stream stabilization I&amp;E form from spreadsheet</td>
<td>TS3C–13</td>
</tr>
</tbody>
</table>
Abstract

Inventory and evaluation of stream stability requires an understanding of the cause of the perceived problems. Sometimes, causes of instability are visible onsite, but many times it is necessary to consider activities in other reaches of the stream or in the overall watershed. Also, the problem may not be anthropogenic at all, but rather a naturally occurring process that is incompatible with the existing riparian land use. This technical supplement introduces the concepts of stream stability and equilibrium along with a channel evolution model (CEM) as background material. It then presents a detailed procedure for data collection and analysis to facilitate the understanding of the dynamics of a subject stream. Published data and field-collected measurements are analyzed and compared; when all valid data match closely, the level of confidence in the analysis is high, and an assessment of the situation can proceed. The suggested procedure relies heavily on a spreadsheet tool developed by Illinois NRCS to collect and compare all available relevant data, but the same analysis can be successfully accomplished without this specific tool.

Problem identification and trend analysis

Causes of channel and bank instability can be broadly grouped into four areas of common causes: downstream, upstream, watershedwide factors, and local factors. Downstream factors involve lowering of the downstream base level, which can significantly impact upstream reaches. Upstream factors alter the incoming discharge of water and/or sediment by installation of features such as dams and diversion channels. Watershedwide factors are the result of major land use changes such as urbanization. Local factors result from geotechnical failures, sparse riparian vegetation, and unstable planform. These local causes may be exacerbated by upstream, downstream, or watershedwide factors or they may be the primary cause.

One common misconception often found is the assumption that a stable stream should not erode its banks. The fact is that stable streams are not static; they typically migrate more slowly than one that has been destabilized by anthropogenic forces. The difference between stable and unstable is not always a clear distinction as streams in dynamic equilibrium will continually migrate slowly across their flood plains. The distinction is in the rate of lateral migration being slow enough in stable streams that the riparian zone remains essentially intact through the entire process. Stable streams should, however, remain essentially static in relation to their overall profile; that is, they will not exhibit any large scale degradation or aggradation.

Watershedwide problems

Hundreds of years of human activity on the landscape have made significant changes in the major elements controlling stream balance. People have:

- cleared the timber
- plowed the prairie
- drained the wetlands
- straightened the streams
- levied the flood plains
- built cities with large areas of concrete, asphalt, and rooftops

Results of such activity on stream dynamics have generally had the effect of increasing runoff and stream slope and reducing flood plain width. In many watersheds, the land use changes are a significant factor in increased runoff. In rural areas, this may be due to more intense agricultural activities replacing woodland and grass land with cultivated land. In urban areas, the increase of impermeable surfaces within the watershed results in an increased volume of water. Additionally, the urban development of a watershed typically results in permanent land cover, either in impermeable surfaces or lawns, which produces little sediment to be delivered to the system.

Lane’s Balance (fig. TS3C–1) is a tool for understanding the relationship between factors affecting channel configuration (Federal Interagency Stream Restoration Working Group (FISRWG) 1998). Stability is represented when the scale is balanced and the system has achieved an equilibrium condition. Both the increased runoff from impervious areas and the reduced sediment loads will tend to tip Lane’s Balance to channel...
degradation in the stream system, as illustrated with the arrow in figure TS3C–1. Increased runoff represents higher energy in the streamflow, and reduced sediment load means there is less work for that energy to do. The excess streamflow energy is dissipated by eroding the streambanks or scouring out the bed of the channel (degradation), providing more sediment and bringing the system to a new equilibrium.

Another aid in identifying the processes at work in a stream is the CEM (fig. TS3C–2 (Simon 1989)). This model describes a predictable series of changes that a channel may transition through following some disturbance. The CEM is addressed in more detail in NEH654.03.

**Channel problems**

Channel modifications nearly always contribute to channel instability at some point. Some of the more obvious modifications are channelization, dam construction, and levees. Some less obvious, but still significant changes, include clearing and snagging, gravel mining, and channel lining or paving. The changes induced by these channel modifications can be dramatic, but more typically, they appear rather insignificant to the casual observer, especially in the short term. Time then becomes a significant element to consider in the problem identification phase, as the lag time between channel or watershed changes and the full effects of those changes can be decades. Because the impacts of channel modifications are cumulative over time, it is often difficult to identify a single modification that is responsible for an adverse condition.

The designer’s most important task is to be aware of the overall condition of the stream and identify trends toward or away from the equilibrium or balanced condition. Only then can alternatives be considered.

**Procedures for streambank investigations and analysis**

The underlying assumption to the designer’s investigation and analysis is that every stream has a stable dimension, slope, and planform to safely carry the water and sediment generated from its watershed under the current climate and land use. That is not to
Figure TS3C–2  Channel evolution model (CEM) (Simon 1989)

Class I. Sinuous, premodified
\( h < h_c \)

Class II. Channelized
\( h < h_c \)

Class III. Degradation
\( h > h_c \)

Class IV. Degradation and widening
\( h > h_c \)

Class V. Aggradation and widening
\( h > h_c \)

Class VI. Quasi-equilibrium
\( h < h_c \)

\( h_c \) = Critical bank height

= Direction of bank or bed movement

Flood plain

Terrace

Aggraded material

Slumped material

Oversteepened reach

Precursor nickpoint

Primary nickpoint

Plunge pool

Direction of flow

Top bank

Secondary nickpoint

Aggradation zone

Aggraded material

Bank

Bankfull

Precursor nickpoint

Primary nickpoint

Plunge pool

Direction of flow

Top bank

Secondary nickpoint

Aggradation zone

Aggraded material

Bank

Bankfull
say that the stream is in a static condition, but rather that a stable stream maintains the same dimensions, slope, and planform while moving slowly within its flood plain position. The investigative procedure is a process of determining what the stable conditions of each unique stream segment should be and what the current conditions are, comparing the two conditions, and then attempting to understand the reasons for any differences. Only then can the designer analyze the condition of the stream and recommend action to improve an unsatisfactory condition and move the stream toward a stable state or, at a very minimum, prevent action that would further destabilize the stream.

The Illinois NRCS spreadsheet program, designed to assist in gathering and analyzing the data required for inventory and evaluation (I&E) of an Illinois stream segment, will be presented as a part of the suggested investigative procedure. Some of the data and analysis are very specific to Illinois, particularly gage data and regression curves. If the spreadsheet is used outside of Illinois, the reference stream gage section and the U.S. Geological Survey (USGS) flood-peak discharge prediction section will not apply. The collection form and its accompanying subroutines appear later in this supplement. The spreadsheet program can be found in its most current form on the Illinois NRCS Web site: http://www.il.nrcs.usda.gov/technical/engineer/engsprdshts.html.

**Geomorphic values**

There is a natural variability to hydraulic geometry relationships. It is important to recognize that this variability represents a valid range of stable channel dimensions due to such variables as geology, vegetation, land use, sediment load, sediment grain size, and runoff characteristics. The values suggested in the following procedure for bankfull discharge, width-to-depth ratio, sinuosity, radius of curvature-to-bankfull width ratio, and entrenchment ratio are based on measured observations from streams in Illinois, as well as published ranges from various research done elsewhere. Values for these relationships should not be assumed to be more accurate or precise than intended. These relationships can be used as a preliminary guide to stability in stream reaches, but other techniques and local data should be considered.

**Background data collection (prior to field visit)**

The first step in the investigation phase is to gather existing data for the project area. The information gathered will make the initial field visit much more productive and allow for some preliminary analysis to be done with less field time.

**Step 1** On the I&E spreadsheet, enter the location and identification information including county, legal description, stream name, name(s) of decisionmakers or landowners, and UTM coordinates (if desired). These appear at the top of the spreadsheet I&E form.

**Step 2** Aerial photography is the first data set to acquire. Using the most recent aerial photography available, compare with older aerial photos to determine:

- Channel alignment changes (straightening and shortening of the channel length)—Calculate channel sinuosity (old and new).
- Lateral migration rates—By measuring from discernible features such as known points, roads, and section lines, and determining the total migration rate for several years, a reasonable estimate can be made of average annual migration.
- Changes in the channel width over time—Has the channel top width gotten larger? Widening could be a sign of past downcutting, or excessive bed load causing aggradation.
- Changes in the bed features such as central bars and size of point bars—Increased bar size could be a sign of excessive bed load.
- Scour patterns in the flood plain
- Locations of any existing levees

**Step 3** From USGS topographic maps (or other suitable maps), determine the watershed boundaries of the stream reach. Calculate drainage area (if available, nearby gage data can be used to help determine the drainage area), and enter in square miles on the spreadsheet.
**Step 4** Regional curve bankfull dimensions are supplied by the spreadsheet program based on drainage area, based on work by Dunne and Leopold (1978) (fig. TS3C–3 (FISRWG 1998)). The data are based on typical relationships and may not be applicable to a specific watershed or area. For example, curve B bankfull widths and depths correlate reasonably well with observations of several hundred rural streams in Illinois, but should be used cautiously (if at all) in an urban setting. Development of regional curve bankfull dimensions for streams in the subject hydro-physiographic area should be pursued for best results.

**Step 5** Look for reference streamflow gaging data. USGS and some state and local governments may own or operate gaging equipment on the stream you are investigating. If not, look for the nearest gage data available in a watershed with similar soils, climate, and land use to the one you are investigating.

   a. Gage data are available online at [http://www.usgs.org](http://www.usgs.org) for USGS-operated gages.

   b. The Illinois NRCS stream stabilization spreadsheet has a pull-down menu of USGS gage data in and near the selected county. The 2-year return interval maximum discharge, Q2, calculated from the actual gage data will be displayed for the selected gage along with the station number and its drainage area. Results of the USGS regression analysis (USGS 1987) are also displayed, if available; they are not available for urban streams in Northeastern Illinois as the regression analysis does not represent urban hydrology. This feature is applicable only to Illinois streams. Further information on stream gage analysis is provided in NEH654.05.

**Step 6** To determine the USGS flood-peak discharge predictions for the subject stream, the spreadsheet needs a value for valley slope (USGS 1987). Rainfall and regional factor are automatically supplied based on the county selection, and the predicted Q2 discharge from the regression equation will be displayed. It will always display the typical range for bankfull, which is 40 percent to 80 percent of the Q2 discharge, corresponding to the approximate 1 to 1.5-year return interval storm event commonly representing bankfull flow in Illinois. If the subject stream is not in Illinois, use other data if available.

   a. For the regression analysis, valley slope is defined as “the difference of elevations divided by distance between points 10 percent and 85 percent of the total distance measured along the low-water channel of the stream from the site to the basin divide” (USGS 1987). Divide the difference in elevation by total flowline distance between points, using the topographic map with delineated drainage area determined previously.

   b. If desired, the spreadsheet valley slope subroutine (fig. TS3C–4) may be used. The subroutine prompts entries of topographic contour elevations and corresponding distances along the flow line of the channel. It automatically determines elevations at the critical points using linear interpolation and plots a profile of the channel to provide a visual model of the process.

**Step 7** The sinuosity of the local stream site is best determined from a recent aerial photo. Identify the points where contour lines immediately upstream and downstream of the project site cross the stream channel. Measure the stream length along the channel between the two points, along with the valley length (a straight line measurement) between the same two points. Enter these distances on the spreadsheet, along with the contour interval, and the resulting sinuosity will automatically be determined.
Figure TS3C–3  Regional curves showing bankfull dimensions by drainage area

- A: San Francisco Bay region at 30 inches annual precipitation
- B: Eastern United States
- C: Upper Green River, Wyoming
- D: Upper Salmon River, Idaho (Emmett 1975)
Figure TS3C-4  Valley slope subroutine from stream stabilization spreadsheet

USGS "Water Resources Investigations Report 87-4207" definition of valley slope:
"the difference of elevations divided by distance between points 10% and 85% of the total
distance measured along the low-water channel of the stream from the site to the basin divide"

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Distance from Previous Contour (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>655</td>
<td></td>
</tr>
<tr>
<td>660</td>
<td>2000</td>
</tr>
<tr>
<td>665</td>
<td>1920</td>
</tr>
<tr>
<td>670</td>
<td>1480</td>
</tr>
<tr>
<td>680</td>
<td>1170</td>
</tr>
<tr>
<td>685</td>
<td>990</td>
</tr>
<tr>
<td>690</td>
<td>1000</td>
</tr>
<tr>
<td>695</td>
<td>1130</td>
</tr>
<tr>
<td>697</td>
<td>500</td>
</tr>
</tbody>
</table>

Measured Data from Topographic Map:

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>Stream Distance from Downstream Contour to Site (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>655</td>
<td>250</td>
</tr>
</tbody>
</table>

Valley Slope (ft/m): 16.9

Clear Calls

Valley Slope at Tayne K/10/2007

project: John T
Assisted by: Wayne K
Date: 1/10/2007

Distance (ft)

Elevation (ft)

Valley Slope:
- stream channel
- site
- valley slope

Return to I&E Form
Field data collection

With the background data gathered and an understanding of the perceived problems and risks, the designer is ready to make a field visit to the site. Actual field measurements from the subject site are used to customize the analysis. The local stream morphology section of the spreadsheet is a way to record and interpret field observations of the bankfull condition.

*Step 1* Observe the roughness of the channel, which is affected by vegetation, obstructions, irregularities in cross section, and meandering. Select a value for Manning’s $n$ from the pull-down menu on the I&E spreadsheet, based on channel description.

*Step 2* During the field visit, walk at least two meander lengths of the stream channel, identifying bankfull indicators. Mark the elevations of indicators with flags, and use a hand level or other survey instrument to determine the height above existing flowline. Best indicators are the first flat depositional surface, top of washed root zone, and a break in slope angle on the streambank.

Refer to figure TS3C–5 (Steffen, Roseboom, and Kinney 2000) for guidance on locating bankfull indicators. The regional curve predictions for channel dimensions (fig. TS3C–3) are mean depths. Bankfull indicators identified in the field will be measured at maximum bankfull depth, and maximum depth may be 0.5 to 2.0 times the mean bankfull depth predicted by the regional curve data. Therefore, during the field investigation, do not expect bankfull indicators to be found at the mean depth predictions unless the channel cross section is a flat bottomed rectangle. A further description on the identification of bankfull indicators is provided in NEH654.05.

*Step 3* After measuring several bankfull indicator elevations, look for converging evidence to support your selection of indicators. When selected indicators are zeroed in to within a few tenths of a foot, take an average, and use the result as your field identified bankfull stage. Also, at a riffle location, measure the distance across the channel at the bankfull elevation. Note: If the channel is undergoing active downcutting (CEM stage 3 or 4 (fig. TS3C–2)), there will not be any reliable bankfull indicators.

Figure TS3C–5   Bankfull indicators used for field identification

Bankfull stage (approximate 1.5 year or 67 percent chance)

- First, flat depositional surface
- Lowest extent of woody vegetation
- Top of washed root zones
- Topographic break in slope
- Top of point bar or other deposits
- Change in nature and amount of debris deposits
- Change in size of substrate materials
- Zone of washed rock

TS3C–8  (210–VI–NEH, August 2007)
Step 4  Survey a cross section at the nearest riffle (fig. TS3C–6), extending out on each side at least to the flood plain elevation. The survey data will be used to calculate the cross-sectional area at the field identified bankfull stage. To determine a representative channel slope, survey at least several hundred feet along the streamflow line, at riffle locations. Since channel slopes are often quite flat, it is critical to take accurate measurements at a minimum of three or more riffles to determine channel slope.

Step 5  Measure the radius of curvature, Rc, (fig. TS3C–7 (FISRWG 1998)) of the channel bend(s) in the project area. Alternatively, this can be done using a recent aerial photo, if desired.

Step 6  During the field visit, measure the characteristics of the bed load. Larger cobbles indicate higher velocity flow. Sieve a bed load sample and do a pebble count, or estimate the D₉₀ bed-load size (the size mesh through which 90 percent of the bed load would pass). Do the same for the D₅₀ bed-load size. More information on sediment sampling is provided in NEH654 TS13A.

Data analysis and assessment

Analysis of the field data involves first determining the value of several standard parameters used to describe stream morphology: width-to-depth ratio, entrenchment ratio, sinuosity, and the ratio of radius of curvature to bankfull width. These parameters will be used to assess the condition of the stream and the potential for stabilization. Bankfull discharge and flow velocity are determined in several ways from the field data. The ultimate goal is to develop confidence in the analysis by matching discharge and velocity measurements from as many sources as possible.

Step 1  Plot the riffle cross section on the cross-sectional spreadsheet subroutine (fig. TS3C–6) and enter a flow depth equal to the maximum bankfull depth as determined from the field bankfull indicators. Cross-sectional area, velocity, discharge, and hydraulic radius will be computed using Manning’s equation and displayed on the subroutine page. If the actual channel slope data is absent on the I&E sheet, the cross-sectional subroutine will use a slope estimate based on entries from the sinuosity determination.

Step 2  Width-to-depth ratio is determined from the bankfull width and the mean bankfull depth.

Step 3  Bankfull width can be entered directly from the field measurement, or measured from the plotted cross section.

Step 4  Mean bankfull depth can be determined by dividing the cross-sectional area at the field-determined maximum bankfull elevation by the stream width at the maximum bankfull elevation.

Step 5  The entrenchment ratio compares the bankfull width to the width of flow when the stream reaches twice the maximum bankfull depth for the bankfull discharge. On the I&E spreadsheet, enter maximum bankfull depth (from the cross section taken at the riffle) and the width of the channel or flood plain at twice the depth; the entrenchment ratio will be automatically determined.

Step 6  Enter the measured radius of curvature; its ratio to bankfull width is automatically calculated by the spreadsheet.

Step 7  Enter the discharge calculated by the cross-sectional subroutine at maximum bankfull depth as the selected Q on the I&E spreadsheet, or select your own best estimation of bankfull discharge based on all of the foregoing data (including the regression analysis and other background investigation).

Step 8  Enter the field-determined bed-load sizes on the spreadsheet.

Step 9  The spreadsheet will display a series of four bankfull velocity checks:

- velocity required to move D₉₀ bed load
- velocity from cross-sectional subroutine (using Manning's equation on actual surveyed cross section and slope)
- velocity calculated from basic field data (using a modified Manning's equation with mean depth in place of hydraulic radius)
- velocity from the selected Q entry, using V=Q/A and a cross-sectional area determined from the basic field data section
Figure TS3C–6  Cross-sectional subroutine from stream stabilization spreadsheet

Natural Open Channel Flow

\[ Q = \frac{1.486}{n} \frac{A R^2}{S^n} \]

assuming uniform, steady flow

Project: John T  
Assisted by: Wayne K  
Date: 1/10/2007  
Channel Slope (S): 0.004580  
Manning's n: 0.035  
Flow Depth: 3.3  

Survey Data:

<table>
<thead>
<tr>
<th>Rod (ft)</th>
<th>Distance (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.7</td>
<td>-17.0</td>
</tr>
<tr>
<td>6.3</td>
<td>-15.0</td>
</tr>
<tr>
<td>11.5</td>
<td>-6.0</td>
</tr>
<tr>
<td>12.2</td>
<td>-4.0</td>
</tr>
<tr>
<td>12.3</td>
<td>0.0</td>
</tr>
<tr>
<td>12.1</td>
<td>5.0</td>
</tr>
<tr>
<td>10.8</td>
<td>6.0</td>
</tr>
<tr>
<td>9.9</td>
<td>8.0</td>
</tr>
<tr>
<td>4.2</td>
<td>14.0</td>
</tr>
</tbody>
</table>

Selected Flow Depth: 3.3 ft  
Channel Flow (Q): 218.6 cfs  
Channel Velocity: 4.8 ft/sec  
Cross-Sectional Area (A): 45.7 sq.ft.  
Hydraulic Radius (R): 2.1 ft

Trial Depth 2  Trial Depth 3

-20.0  -15.0  -10.0  -5.0  0.0  5.0  10.0  15.0  20.0

Distance (ft)

-20.0  -15.0  -10.0  -5.0  0.0  5.0  10.0  15.0  20.0

Rod (ft)

-20.0  -15.0  -10.0  -5.0  0.0  5.0  10.0  15.0  20.0
**Step 10** Velocities from all four calculations should be very close and should be sufficient to move the $D_{90}$ bed load. If more than 1.0 feet per second difference is observed between these four values, review to see if there is a mistake in data entry. If not, the bankfull indicators may be in error and need to be rechecked.

**Step 11** After all the velocities compare well, compare the bankfull dimensions with those predicted by the regional curves, and compare the selected Q with the discharge predicted by the gage data and/or the regression equation. Modify entries as needed to develop confidence that the stream condition is understood. The field indicators should be the main guide, not the regional curve data or the regression equation predictions, as the field indicators are specific to the stream being investigated. Also, if the stream segment is in channel evolution stage 3 or 4, there will be no reliable bankfull indicators, and the designer will be forced to rely on flow relationships developed from other similar watersheds and experience gained from previous comparisons.

**Departure analysis**

Now that the designer has determined the bankfull or channel forming discharge in the stream segment, some analysis of the stream condition compared to stable streams can begin.

**Condition 1:** Is the flood plain elevation at or near the elevation of maximum bankfull depth?

*Yes.* The channel is connected to the flood plain. Discharges larger than bankfull begin to spread out over the flood plain, slowing velocities and dissipating energy. The channel has not experienced significant downcutting. CEM stage 1 or 6 would apply: a stable configuration. The entrenchment ratio (width at twice maximum bankfull depth/ bankfull width) will be greater than 2.5.

*No.* The channel is not connected to the flood plain. Discharges larger than bankfull will remain inside the channel with little or no opportunity to spread out onto the flood plain. This is evidence of current or past downcutting. The channel evolution process is active and its morphology is adjusting to regain equilibrium with flow characteristics. Incised channels such as this are likely to continue to erode laterally to build a flood plain. CEM stage could be 2, 3, 4, or 5. The entrenchment ratio will be less than 2.5. Entrenchment ratio will be smallest in stage 2 or 3 channels and then increase to about 2.5 or more as channel nears a new equilibrium in stage 6. The exception to this condition will be low-gradient, channelized streams with insufficient energy to erode the channel boundary, even when entrenched.

**Condition 2:** Is the channel bed in riffle locations comprised of bed-load material or is it residual (hard) silt, clay, or bedrock?

*Bed-load material.* The channel is probably not actively downcutting. Bed-load material is not being swept away by streamflow. If the entrenchment ratio is low (less than 2.5), the channel is most likely in the widening phase of the CEM, stage 4 or 5.
Residual (hard) silt, clay, or bedrock. Bed-load material is being swept out of this reach of channel, leaving the residual material exposed at the riffle locations. The channel is actively downcutting (CEM stage 3). If the streambed is not stabilized, this reach of stream will go through all six CEM stages and the degradation will advance upstream until it meets resistance in the form of bedrock, bridge floor, and culvert. Channels can be downcutting even when the entrenchment ratio is over 2.5. Streams are not considered entrenched until they degrade to twice the maximum bankfull depth, but degradation begins as soon as the bottom begins to be eroded.

**Condition 3:** Is the width-to-depth ratio less than 10 with an entrenchment ratio less than 1.4 (a deep, narrow channel)?

Yes. Width-to-depth ratios can be small (less than 10) in low gradient, fine-grained, or sinuous channels. However, these channel types are always connected to the flood plain in stable situations. Therefore, width-to-depth ratios less than 10, combined with entrenchment, are good indicators that downcutting has occurred in the past or is actively occurring at present (CEM stage 2, 3, 4 or 5). If, in addition, the sinuosity is low (less than 1.2), it is likely that the stream has been channelized to create the entrenched condition.

No. If width-to-depth is greater than 20, suspect an overwidened stream segment and sediment transport problems (CEM stage 5). This condition could indicate an aggrading stream segment.

**Condition 4:** Is the velocity calculated from the cross-sectional subroutine of the I&E spreadsheet much faster or much slower than that required to move the D$_{90}$ bed-load material?

**Much faster**—Excessive velocities indicate that bed-load material is too small to resist existing velocities. Therefore, downcutting is probably occurring (CEM stage 3). Check the status of condition 2. Streams with only very fine-grained bed-load material will have excessive velocities compared to D$_{90}$ material size. Vertical stability of these streams cannot be assessed using bed-load material size estimates.

**Much slower**—Slow velocity could indicate an aggrading system where the heavy bed load generated upstream cannot be transported through the system. These conditions often occur in delta areas above impoundments or at confluences with larger streams. They also occur when channel velocities change due to slope changes (at the downstream end of a channelized reach), when width-to-depth ratios increase dramatically or when there is an exceptionally large contribution of bed load just upstream.

**Condition 5:** Is the radius of curvature-to-bankfull width (Rc/W) ratio less than 1.8?

Yes. The situation is outside of the normal range of planform stability. It may be necessary to realign the channel or walk away from the project. Natural, stable channel radius of curvature-to-bankfull width ratios vary widely, but most commonly range from 2.3 to 2.7 or higher. With a radius of curvature-to-bankfull width ratio less than 1.8, the possibility of a channel cutoff at this point increases dramatically.

**I&E spreadsheet details**

The inventory and evaluation function of the stream I&E spreadsheet includes the following introduced in the discussion of suggested I&E procedure (figs. TS3C–4, TS3C–6, and TS3C–8) in this technical supplement:

- streambank I&E form
- cross-sectional subroutine
- valley slope subroutine

In addition to the above, the spreadsheet also includes design sheets to determine dimensions and material quantities for certain standard stream stabilization practices, and automatically fills out the applicable Illinois standard drawings:

- rock riffles
- stone toe protection
- stream barbs
**Figure TS3C–8**  Stream stabilization I&E form from spreadsheet

<table>
<thead>
<tr>
<th>County</th>
<th>Jefferson ▼</th>
<th>T. 4S</th>
<th>R. 1E</th>
<th>Sec. 22</th>
</tr>
</thead>
<tbody>
<tr>
<td>Date</td>
<td>1/10/2007</td>
<td>By</td>
<td>Wayne K</td>
<td></td>
</tr>
<tr>
<td>Stream Name</td>
<td>Happy Creek</td>
<td>UTM Coord.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Landowner Name</td>
<td>John T</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drainage Area</td>
<td>2.67 sq. mi.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Regional Curve Predictions:**

<table>
<thead>
<tr>
<th>Bankfull dimensions</th>
<th>Width</th>
<th>22 ft.</th>
<th>Cross Sectional Area</th>
<th>44 sq. ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>2.0 ft.</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Reference Stream Gage:**

<table>
<thead>
<tr>
<th>Sevenmile Creek near Mt. Vernon ▼</th>
<th>Station No.</th>
<th>05595800</th>
<th>Gage Q₂</th>
<th>1030 cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jefferson County, IL</td>
<td>Drainage Area</td>
<td>21 sq mi</td>
<td>Regression Q₂</td>
<td>1410 cfs</td>
</tr>
</tbody>
</table>

**USGS Flood-Peak Discharge Predictions:**

<table>
<thead>
<tr>
<th>Valley Slope: ft./mi. (user-entered)</th>
<th>Rainfall: 3.40 in (2 yr, 24 hr)</th>
<th>Regional Factor: 0.983</th>
<th>Typical Range for Bankfull Discharge: 80 to 180 cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>16.9 ft/mi (from worksheet)</td>
<td>0.0032 ft./ft.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Local Stream Morphology:**

<table>
<thead>
<tr>
<th>Channel Description: (b) Same as (a), but more tones and weeds ▼</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manning's &quot;n&quot;</td>
</tr>
</tbody>
</table>

**Basic Field Data:**

<table>
<thead>
<tr>
<th>Bankfull Width</th>
<th>13 ft.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contour Interval</td>
<td>5 feet</td>
</tr>
</tbody>
</table>

**Mean Bankfull Depth**: 3.2 ft.

**Width/Depth Ratio**: 4.06

**Max. Bankfull Depth**: 4.2 ft.

**Width at twice max. depth**: 300 ft. (8.4 ft.)

**Entrenchment Ratio**: 23.08

**Radius of Curvature (Rc)**: 0 ft.

**Bankfull Velocity Check**: (typical Illinois streams will have average bankfull velocity between 3 and 5 ft/sec.)

<table>
<thead>
<tr>
<th>Bedload: D₉₀₅</th>
<th>1 in.</th>
</tr>
</thead>
<tbody>
<tr>
<td>D₅₀₅</td>
<td>▼ in.</td>
</tr>
<tr>
<td>Velocity required to move D₉₀₅:</td>
<td>2.1 ft./sec.</td>
</tr>
<tr>
<td>Velocity from Cross-Section data:</td>
<td>4.78 ft./sec.</td>
</tr>
<tr>
<td>Velocity from basic field data:</td>
<td>6.26 ft./sec.</td>
</tr>
<tr>
<td>Velocity from selected Q:</td>
<td>5.4 ft./sec.</td>
</tr>
</tbody>
</table>

**Channel Evolution Stage**: III

**Stream Type (Rosgen)**

**Notes**

(210–VI–NEH, August 2007)
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.

(210–VI–NEH, August 2007)
Overview of United States Bats

Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Introduction</td>
<td>TS3D–1</td>
</tr>
<tr>
<td>Habitat</td>
<td>TS3D–1</td>
</tr>
<tr>
<td>Status and impacts</td>
<td>TS3D–2</td>
</tr>
<tr>
<td>Conservation</td>
<td>TS3D–3</td>
</tr>
</tbody>
</table>

Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table TS3D–1</td>
<td>General habitat, distribution, and status of federally protected bat species in the United States</td>
<td>TS3D–4</td>
</tr>
</tbody>
</table>

Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS3D–1</td>
<td>Southeastern bat (<em>Myotis austroriparius</em>)</td>
<td>TS3D–1</td>
</tr>
<tr>
<td>Figure TS3D–2</td>
<td>Maternal colony of Rafinesque's big-eared bats (<em>Corynorhinus rafinesqui</em>) roosting beneath a concrete bridge</td>
<td>TS3D–2</td>
</tr>
<tr>
<td>Figure TS3D–3</td>
<td>Healthy riparian areas are important to bats for roosting and foraging</td>
<td>TS3D–2</td>
</tr>
<tr>
<td>Figure TS3D–4</td>
<td>Sauta Cave National Wildlife Refuge serves as critical protected habitat for gray and Indiana bats.</td>
<td>TS3D–3</td>
</tr>
</tbody>
</table>
Introduction

In the past, bats (order Chiroptera) have been one of the most feared and misunderstood creatures (fig. TS3D–1). Today, as researchers are beginning to unravel the secrets of the world's only flying mammals, bats are declining worldwide. Misconceptions and folklore concerning bats have been passed along for generations, and unfortunately, this has led to the senseless killing of colonies of millions of bats, sometimes in a single destructive act. Most bats only produce one pup a year; therefore, recolonizing a decimated population and replenishing a once occupied cave may take decades. With thanks to extensive conservation and education, bats are making a comeback both in the United States and worldwide.

The diversity of bat species, the habitats they occupy, and their behavioral and social features are impressive (Fenton 1997). Occurring worldwide except for the Polar regions and a few isolated islands, there are more than a thousand known bat species, and this number continues to increase (Mickleburgh, Hutson, and Racey 2002; Engstrom and Reid 2003). Some species, such as Mexican free-tailed bats (Tadarida brasiliensis), undertake extensive seasonal migrations. Other species migrate for food, such as the long-nosed bats (Leptonycteris spp.) and the Mexican long-tongued bat (Choeronycteris mexicana), which are thought to travel along nectar corridors. Species such as endangered gray bats (Myotis grisescens) and Indiana bats (M. sodalis) migrate more locally between summer roosts and winter hibernacula. Elevated migration routes may also occur. They are thought to be a strategy of the spotted bat (Euderma maculatum), which lives in southwestern regions.

Bats have an extraordinary dietary diversity. Different species feed specifically on insects, fish, frogs, blood, fruit, or nectar. In the United States, bats are voracious feeders on night-flying insects, and three species of nectar/pollen feeding bats live in the extreme southern regions bordering Mexico. Insectivorous species typically consume more than 50 percent of their body weight in bugs nightly (Harvey, Altenbach, and Best 1999). Consider the 20 million Mexican free-tail bats from Bracken Cave, Texas, the largest concentration of mammals in the world. They devour approximately 200 tons of pests a night. Only 150 big brown bats (Eptesicus fuscus) are required to protect farmers from 33 million corn rootworms (Diabrotica spp.) each summer Bat Conservation International (BCI) 2000. The pollinating species of bats in the Southwest is important for the survival of agaves and columnar cacti. The beneficial and economical role bats play for humans and ecosystems is clear.

Habitat

An increased availability of roost diversity usually corresponds to an increased population and diversity of bat species (Findley 1995). Cave bats or forest-dwelling species are generally colonial species that roost in caves, crevices, hollow trees, or under loose bark. The Indiana bat, for example, uses caves for hibernation during the winter months and tree cavities or beneath exfoliating bark of various trees in the summer (Kurta et al. 1992). Some cavity dwelling species will also use manmade structures such as mines, bridges, culverts, and bat houses (fig. TS3D–2). Several suitable roosts
in an area may provide a colony of bats with the necessary thermal variation required throughout the day and allow escape from parasites or predators (Lewis 1995).

Tree bat species typically roost solitarily or in small groups in foliage or moss at different canopy levels. Young red bats (*Lasiusurus borealis*), for example, have been observed roosting higher in trees than adults (Constantine 1966). Studies have suggested that female and male bats use different habitats and roosting sites (Brigham 1991; Cryan, Bogan, and Altenbach 2000). Findley (1995) suggests that differences in morphology, flight maneuverability, and echolocation proficiency affect partitioning of foraging and roosting habitat between species. Thus, species, age, sex, reproductive condition, or migratory status may account for differences in roost and habitat selection among bats. Overall, a diverse landscape and forest stratification with a multifaceted arrangement of potential roosting and foraging areas, even in suburban areas, appears to be important for healthy bat populations (Evelyn, Stiles, and Young 2004).

Since fresh water is critical to their survival, bats are closely associated with riparian environments (Martin 2001) (fig. TS3D–3). Riparian areas are of particular importance to bats, possibly due to their high resource of flying insects (Barclay 1991), especially for species in arid regions (Bell 1980).

Riparian areas also offer an abundance of snags, which are important roosting sites for many species of bats. Tuttle (1976) suggests that roost selection may be determined by proximity to required resources such as water, forage areas, and hibernation sites, thus, reducing energy expenditure by reducing travel distance. For example, there is a decreased growth rate and higher mortality rate among juvenile gray bats where greater distance is traveled from roosting sites (caves) to their preferred foraging habitat, which is over water. Research also suggests that riparian zones act as important travel corridors, space for open flight, and forest edges which are frequently used for feeding and migration by some bat species (Wunder and Carey 1994).

### Status and impacts

Of the 45 species of bats that occur in the continental United States and Hawaii, 6 species are considered to be federally endangered and 20 are species of concern (Harvey, Altenbach, and Best 1999). The International Union for Conservation of Nature and Natural Resources lists 10 bat species on the Red List of Threatened Species (IUCN 2002) (table TS3D–1). Species of concern, former category 2 candidates, are those sensitive species in which data pertaining to biological vulnerability and threat is not yet available to justify a

**Figure TS3D–2** Maternal colony of Rafinesque's big-eared bats (*Corynorhinus rafinesquii*) roosting beneath a concrete bridge

**Figure TS3D–3** Healthy riparian areas are important to bats for roosting and foraging
threatened or endangered status. Due to their nocturnal behavior, capability of flight, and the frequent remote location of their roosts, bats are one of the most difficult groups to research and to monitor. Although many species of bats in the United States appear to be declining, little is known about the populations and ecology for many species (Arnett 2003; O’Shea, Bogan, and Ellison 2003).

Intentional killing, vandalism, cave exploration and commercialization, and closure of abandoned mine entrances have greatly reduced roosting habitat for many bat species in the United States (Harvey, Altenbach, and Best 1999). Disturbance to hibernating bats can be detrimental due to the potential loss of needed energy reserves, which must last until summer emergence. Endangered gray bats, for example, are especially vulnerable since 95 percent of the population hibernate in only 11 caves in the Southeast (Harvey, Altenbach, and Best 2001). Disturbance to maternal colonies when newborn are present may also be injurious to bats since frightened mothers may drop their young or abandon the roost (Harvey et al. 1999). Natural disasters such as flooding can also effect populations, but human disturbance is the primary cause of their decline.

Loss of healthy riparian systems, especially in the southwest where permanent water sources are in decline, also negatively impact bat populations. Use of pesticides and other toxicants may contaminate water and food sources, and the loss of mature trees and snags may limit roost availability. Other causes of decline may include clearcutting, strip mining, and human encroachment into dwindling habitats (Martin 2000).

**Conservation**

Fortunately, a greater understanding of these beneficial creatures has led to great strides in their conservation. For example, protection of caves and mines through properly designed gates has shown considerable success for endangered and sensitive species (Tuttle 1977; Tuttle and Taylor 1998) (fig. TS3D–4). Placement of artificial roosts in roost-deficient areas or where colonies have been evicted from homes or buildings has also been valuable to various bat species. Burke (1999), Brittingham and Williams (2000), and Arnett and Hayes (2000) attracted bats to flat-bottomed bridges by installing specially constructed bat boxes. Continued research, innovation, management, and education have been, and will be, critical for the future of this unique group of animals.

Nonprofit organizations such as BCI have been educating people, advancing research efforts, and establishing collaboration efforts around the world for 20 years. The North American Bat Conservation Partnership (NABCP) was established in 1999 to support continentwide conservation efforts. They formed an alliance of working groups, researchers, nongovernmental organizations, and state and Federal agencies from Canada, the United States, and Mexico. This partnership has identified conservation priorities through a strategic plan, which will guide the future direction of research, education, and management (Keeley, Fenton, and Arnett 2003).

Another collaboration, the Program for the Conservation of Migratory Bats between Mexico and the United States (PCMM), was developed in 1995 due to declining bat populations in Mexico. Its objectives are to protect and conserve migratory species and to sustain their ecological roles and evolutionary processes (Medellin 2003). Partnerships and collaboration efforts such as these have demonstrated their important role in curtailing the rapid decline of bats in the United States and worldwide.

**Figure TS3D–4** Sauta Cave National Wildlife Refuge serves as critical protected habitat for gray and Indiana bats.
### Table TS3D–1 General habitat, distribution, and status of federally protected bat species in the United States

<table>
<thead>
<tr>
<th>Species</th>
<th>Status</th>
<th>USFW</th>
<th>IUCN</th>
<th>U.S. distribution</th>
<th>General habitat</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Phyllostomatidae</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>California leaf-nosed bat (<em>Macrotus californicus</em>)</td>
<td>SOC</td>
<td>VU</td>
<td></td>
<td>Southern CA, AZ extending into the southern tip of NV, and the extreme western portion of NM</td>
<td>Lowland desert habitat; abandoned mine tunnels may be used as day roosts and night roosts may include buildings, bridges, porches, or rock shelters</td>
</tr>
<tr>
<td>Mexican long-tongued bat (<em>Choeronycteris mexicana</em>)</td>
<td>SOC</td>
<td>LR/nt</td>
<td></td>
<td>Southern portion of CA, AZ, and southern tip of NM</td>
<td>Occupies a range of habitats from arid thorn shrub to tropical deciduous forest and mixed oak conifer; inhabits caves, buildings, and abandoned mines</td>
</tr>
<tr>
<td>Lesser long-nosed bat (<em>Leptonycteris curasoae yerbabuenae</em>)</td>
<td>FE</td>
<td>VU</td>
<td></td>
<td>South central and southeastern part of AZ and the extreme southern region of Mexico</td>
<td>Desert-scrub habitat; occupies abandoned mines and caves in areas consisting of agaves, yuccas, saguaros, and organ pipe cacti</td>
</tr>
<tr>
<td>Greater long-nosed bat (<em>Leptonycteris nivalis</em>)</td>
<td>FE</td>
<td>EN</td>
<td></td>
<td>Big Bend region of TX</td>
<td>Occupies a range of habitats from sparsely vegetated deserts to pine-oak woodlands; generally inhabits deep caverns, but will use hollow trees, mines, culverts, and buildings</td>
</tr>
<tr>
<td><strong>Vespertilionidae</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spotted bat (<em>Euderma maculatum</em>)</td>
<td>SOC</td>
<td>—</td>
<td></td>
<td>West central U.S.</td>
<td>Mostly occupies rocky arid to semiarid terrain such as desert, scrub areas, or ponderosa pine forest</td>
</tr>
<tr>
<td>Allen’s big-eared bat (<em>Idionycteris phyllotis</em>)</td>
<td>SOC</td>
<td>—</td>
<td></td>
<td>Extreme southern NV, southern third of UT, throughout AZ, and the southwestern quarter of NM</td>
<td>Riparian habitats above 3,000 feet; common in coniferous forests and pine-oak forest canyons; maternity colonies of 30 to 150 individuals have been found in mine shafts, boulder piles, lava beds, and under bark of large ponderosa pine snags</td>
</tr>
<tr>
<td>Hawaiian hoary bat (<em>Lasiurus cinereus semotus</em>)</td>
<td>FE</td>
<td>—</td>
<td></td>
<td>Hawaiian Islands: Kauai, Oahu, Maui, and Hawaii</td>
<td>Coastal and lowland forested areas; on Kauai, occurs primarily in open wet areas near forests; roosts in trees or rock crevices</td>
</tr>
<tr>
<td>Southeastern bat (<em>Myotis austroriparius</em>)</td>
<td>SOC</td>
<td>—</td>
<td></td>
<td>Wide spread distribution in the Southeast</td>
<td>Roost primarily in caves in the North and in the South will utilize buildings, bridges, hollow trees; maternity colonies have been located mainly in caves and hardwood swamp areas</td>
</tr>
<tr>
<td>Western small-footed bat (<em>Myotis ciliolabrum</em>)</td>
<td>SOC</td>
<td>—</td>
<td></td>
<td>Western U.S.</td>
<td>Arid habitats associated with cliffs, talus fields, prairies; roosts in crevices, clay banks, beneath rocks in the ground, and under bark in barns; hibernates in caves and mines</td>
</tr>
<tr>
<td>Western long-eared bat (<em>Myotis evotis</em>)</td>
<td>SOC</td>
<td>—</td>
<td></td>
<td>Western U.S.</td>
<td>Coniferous forests, typically only at higher elevations in southern areas (between 7,000 and 8,500 feet) and semiarid shrublands, sage, chaparral, and agricultural areas; roost in tree cavities, beneath exfoliating bark of both living trees and dead snags, buildings, cliffs, and sink holes; pregnant bats often roost at ground level in rock crevices, fallen logs, and in the crevices of sawed-off stumps</td>
</tr>
<tr>
<td>Species</td>
<td>Status</td>
<td>U.S. distribution</td>
<td>General habitat</td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------</td>
<td>--------</td>
<td>------------------</td>
<td>-----------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gray bat (<em>Myotis grisescens</em>)</td>
<td>FE</td>
<td>Cave regions of AR, MO, KY, TN, and AL</td>
<td>Year-round cave residents although different caves are utilized for winter and summer; hibernation sites are typically deep caves with large rooms capable of trapping cold air; maternal caves usually contain streams with configurations capable of trapping heat</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Eastern small-footed bat (<em>Myotis leibii</em>)</td>
<td>SOC</td>
<td>Eastern U.S.</td>
<td>Hibernate in caves and mines often near entrances, cracks in the floor, and under rock slabs in quarries; use caves, buildings, and barns in summer</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arizona bat (<em>Myotis lucifugus occultus</em>)</td>
<td>SOC</td>
<td>Southwestern U.S.</td>
<td>Ponderosa pine, oak woodlands, riparian forest in desert areas; roosts in caves, crevices, bridges, rarely in mines</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Indiana bat (<em>Myotis sodalis</em>)</td>
<td>FE</td>
<td>Cave regions in the Eastern U.S.</td>
<td>Caves used in winter and maternal colonies inhabit hollow trees or under exfoliating bark usually in floodplain deciduous forests or upland stands adjacent to riparian or floodplain forests; generally several suitable trees are required for roost switching</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fringed bat (<em>Myotis thysanodes</em>)</td>
<td>SOC</td>
<td>Western U.S.</td>
<td>Oak and pinion woodlands most common, also ranges from fir-pine areas to desert-scrub; roosts in caves, mines, and buildings</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cave bat (<em>Myotis velifer</em>)</td>
<td>SOC</td>
<td>Southern KS, western OK, and the southwestern states</td>
<td>Cave regions from south central Kansas to central Texas, rocky canyons, and desert flood plains; summer roosting sites include caves, mines, and sometimes buildings and under bridges</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Long-legged bat (<em>Myotis volans</em>)</td>
<td>SOC</td>
<td>Western U.S.</td>
<td>Forested mountainous regions usually at elevations of 4,000 to 9,000 feet most common, also stream arid and streamside environments; caves and mine tunnels used in winter, summer roosts are tree cavities, crevices and under bark, rock and stream bank crevices, and buildings</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yuma bat (<em>Myotis yumanensis</em>)</td>
<td>SOC</td>
<td>Western U.S.</td>
<td>Variety of habitats with a nearby permanent water source; most often roost in buildings or bridges, sometimes in mines or caves, bachelors sometimes use abandoned cliff swallow nests</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rafinesque’s big-eared bat (<em>Corynorhinus rafinesquii</em>)</td>
<td>SOC</td>
<td>Southeastern U.S.</td>
<td>Historical distribution similar to that of cypress swamps; northern populations hibernate in caves and mines and more southern populations use cisterns or wells, maternal colonies utilize large hollow trees, abandoned homes and buildings, and bridges</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Virginia big-eared bat (<em>Corynorhinus townsendii virginianus</em>)</td>
<td>FE</td>
<td>KY, NC, VA, WV</td>
<td>Limestone karst regions associated with mature hardwood forests; uses caves and abandoned mines as both summer maternity roosts and winter hibernacula, rock shelters</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ozark big-eared bat (<em>Corynorhinus townsendii ingens</em>)</td>
<td>FE</td>
<td>AR, OK, possibly MO</td>
<td>Limestone karst regions associated with mature hardwood forests; uses caves and abandoned mines as both summer maternity roosts and winter hibernacula</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Table TS3D–1

General habitat, distribution, and status of federally protected bat species in the United States—Continued

<table>
<thead>
<tr>
<th>Species</th>
<th>Status</th>
<th>U.S. distribution</th>
<th>General habitat</th>
</tr>
</thead>
<tbody>
<tr>
<td>Western big-eared bat (Corynorhinus townsendii pallescens)</td>
<td>SOC</td>
<td>VU*</td>
<td>Along the west coast</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Roosts include abandoned buildings, bridges, and tunnels</td>
</tr>
<tr>
<td>Townsend's (Pacific) big-eared bat (Corynorhinus townsendii)</td>
<td>SOC</td>
<td>VU*</td>
<td>Western U.S.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Roosts include abandoned buildings, bridges, and tunnels</td>
</tr>
<tr>
<td><strong>Molossidae</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Florida mastiff bat (Eumops glaucinus floridanus)</td>
<td>SOC</td>
<td>—</td>
<td>Southern tip of FL</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Hardwood hammocks</td>
</tr>
<tr>
<td>Western mastiff bat (Eumops perotis californicus)</td>
<td>SOC</td>
<td>—</td>
<td>Southwest U.S.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Areas with natural springs; roosts in crevices high in cliffs</td>
</tr>
<tr>
<td>Underwood's mastiff bat (Eumops underwoodii)</td>
<td>SOC</td>
<td>LRnt</td>
<td>South central AZ</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Organ Pipe Cactus National Monument, Baboquivari Mountains; roosts in woodpecker</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>cavities within saguaro cacti</td>
</tr>
<tr>
<td>Big free-tailed bat (Nyctinomops macrotis)</td>
<td>SOC</td>
<td>—</td>
<td>Southwest U.S.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rocky habitats; roosts in crevices in cliffs, known to use buildings</td>
</tr>
<tr>
<td>Mexican free-tailed bat (Tadarida brasiliensis)</td>
<td>—</td>
<td>LR/nt</td>
<td>Southern U.S. (largest populations in the West)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Occupy various habitats ranging from desert to pine-oak forests; utilize</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>limestone caves and abandoned mines in the Southwest, manmade</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>structures such as bridges and buildings in the Southeast, colonies also</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>found in hollow trees</td>
</tr>
</tbody>
</table>

FE = Federally endangered  
SOC = Species of concern  
EN = Endangered  
VU = Vulnerable  
LR/nt = Lower risk/near threatened  

Notes:  
* Subspecies are not distinguished
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

<table>
<thead>
<tr>
<th>Purpose</th>
<th>TS3E–1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data requirements</td>
<td>TS3E–1</td>
</tr>
<tr>
<td>Stream reach</td>
<td>TS3E–1</td>
</tr>
<tr>
<td>Plan view (planform) type and level I classification</td>
<td>TS3E–1</td>
</tr>
<tr>
<td>Valley types</td>
<td>TS3E–2</td>
</tr>
<tr>
<td>Channel slope—level I</td>
<td>TS3E–2</td>
</tr>
<tr>
<td>Morphological description (level II classification)</td>
<td>TS3E–2</td>
</tr>
<tr>
<td>Channel slope—level II</td>
<td>TS3E–9</td>
</tr>
<tr>
<td>Bankfull discharge validations</td>
<td>TS3E–9</td>
</tr>
<tr>
<td>Entrenchment</td>
<td>TS3E–9</td>
</tr>
<tr>
<td>Channel material</td>
<td>TS3E–9</td>
</tr>
<tr>
<td>Width-to-depth ratio</td>
<td>TS3E–14</td>
</tr>
<tr>
<td>Sinuosity</td>
<td>TS3E–14</td>
</tr>
<tr>
<td>Procedure</td>
<td>TS3E–14</td>
</tr>
<tr>
<td>Interpretations and uses of the Rosgen stream classification</td>
<td>TS3E–14</td>
</tr>
<tr>
<td>Stream management</td>
<td>TS3E–16</td>
</tr>
<tr>
<td>Channel evolution</td>
<td>TS3E–16</td>
</tr>
<tr>
<td>Planning stream restoration measures</td>
<td>TS3E–16</td>
</tr>
<tr>
<td>Communication</td>
<td>TS3E–16</td>
</tr>
<tr>
<td>Prediction</td>
<td>TS3E–16</td>
</tr>
<tr>
<td>Trends and dominant processes</td>
<td>TS3E–18</td>
</tr>
<tr>
<td>Conclusion</td>
<td>TS3E–20</td>
</tr>
</tbody>
</table>
### Tables

<table>
<thead>
<tr>
<th>Table TS3E–1</th>
<th>Plan view characteristics of Rosgen stream types</th>
<th>TS3E–2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table TS3E–2</td>
<td>Aerial and surface views of major stream types</td>
<td>TS3E–4</td>
</tr>
<tr>
<td>Table TS3E–3</td>
<td>Valley type, morphological description, and stream type association</td>
<td>TS3E–8</td>
</tr>
<tr>
<td>Table TS3E–4</td>
<td>Stream morphometry and landform</td>
<td>TS3E–15</td>
</tr>
<tr>
<td>Table TS3E–5</td>
<td>Summary of delineative criteria for broad level classification</td>
<td>TS3E–17</td>
</tr>
<tr>
<td>Table TS3E–6</td>
<td>Summary of characteristics of Rosgen stream types by watershed conditions</td>
<td>TS3E–19</td>
</tr>
</tbody>
</table>

### Figures

<table>
<thead>
<tr>
<th>Figure TS3E–1</th>
<th>Major stream types</th>
<th>TS3E–3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS3E–2</td>
<td>Hierarchical level II key</td>
<td>TS3E–10</td>
</tr>
<tr>
<td>Figure TS3E–3</td>
<td>Measuring stream gradient</td>
<td>TS3E–11</td>
</tr>
<tr>
<td>Figure TS3E–4</td>
<td>Field measurement of entrenchment ratio</td>
<td>TS3E–11</td>
</tr>
<tr>
<td>Figure TS3E–5</td>
<td>Entrenchment ratios of major stream types</td>
<td>TS3E–12</td>
</tr>
<tr>
<td>Figure TS3E–6</td>
<td>Pebble count</td>
<td>TS3E–12</td>
</tr>
<tr>
<td>Figure TS3E–7</td>
<td>Plotted particle size distribution</td>
<td>TS3E–13</td>
</tr>
<tr>
<td>Figure TS3E–8</td>
<td>Evolutionary stages of channel adjustment</td>
<td>TS3E–18</td>
</tr>
</tbody>
</table>
Purpose

Rivers are complex natural systems. A classification system is often used to stratify river reaches into groups that share common physical characteristics. A stream classification system provides better communication among those studying river systems and promotes a better understanding of river processes. The river classification system presented in this technical supplement is based on measurable physical parameters.

The stream classification presented in this technical supplement is condensed from the more detailed version by Rosgen (1994). It is intended for planning purposes, but is not sufficient for design. Appropriate data for use in river classification systems can be obtained from simple measurements and estimates.

The objectives of the Rosgen stream classification are to:

- assimilate a relatively complex mix of mathematical relationships that describes a type of river and simplifies it into a system that can be understood
- provide a consistent and reproducible frame of reference for those working with river systems and communicating stream morphology among a variety of disciplines and interested parties and enable people to talk in common terms about streams
- predict a river’s behavior from its appearance and better understand cause and effect relationships (the specific measured stages in channel evolution models)
- provide a mechanism to extrapolate site-specific data to stream reaches having similar characteristics
- encourage thinking about stream processes relative to channel evolutionary changes and trends
- provide a tool to define a target such as the stable reference reach or desired form to aid in departure analysis and for setting objectives for restoration or rehabilitation

Data requirements

The Rosgen stream classification can be met through a hierarchical assessment of channel morphology measured based on bankfull dimensions. In order, the hierarchical attributes are:

- single-threaded or multiple-threaded channels
- entrenchment ratio
- width-to-depth ratio
- sinuosity
- slope
- material size $D_{50}$ median particle size bed material

Stream reach

The classification applies to segments or reaches of the stream as defined by the user. However, any given stream reach should be comparatively uniform in its physical and biological characteristics.

Stream reaches can be defined from U.S. Geological Survey (USGS) 7.5-minute quadrangles, aerial photographs of appropriate resolution, and confirmed by field reconnaissance. Soil and geologic maps may also provide helpful information for delineating stream reaches.

Plan view (planform) type and level I classification

Level I classification is related to basin relief, landform, and valley morphology. This broadest characterization level is used only where general classification is required. The dimensions, patterns, and profiles are based on information from topographic and/or landform maps and aerial photography. The intent of level I classification is for a broad characterization that integrates landform and fluvial features of valley morphology with channel relief pattern, shape, and dimension (table TS3E–1).
A geomorphic characterization describing A through G stream types completes the level I classification. The stream type is based upon the measures of the stream in plan view from topographical or ortho-digital maps (fig. TS3E–1 (Rosgen 1996)). The plan view is classified as straight, sinuous (meandering), sinuous with active point bars, or braided (numerous intertwining channels separated by longitudinal and/or transverse bars). Some reaches may have actively eroding banks, anastomosing (multiple narrow and deep channels with extensive well-vegetated bars and flood plains), tortuous (extremely contorted meanders), or highly sinuous low width-to-depth bankfull channels.

Aerial photographs of sufficient resolution to show the plan view of the channel bed are good tools depending on the channel size and age of air photos. Judgments should be verified by field reconnaissance, since channels are dynamic and can change their plan view character over time. Photographs and field observations are best obtained at times of low flow and optimum visibility. Flood stages or vegetation can mask important features from observation and could result in misidentification of the plan view type.

A general longitudinal profile, which can be inferred from topographic maps, serves as the basis for breaking the stream reaches into broad slope categories that reflect profile morphology.

The shape of the cross section indicating a narrow and deep stream or a wide and shallow stream can be inferred at the broad level I characterization. The manner in which the channel is incised into the valley can be also be deduced at this level. For example, A and G stream types are narrow, deep, confined, and entrenched. F stream types are wide and shallow and are entrenched. Stream types D and C are wider and shallow with well-developed flood plains. E stream types are narrow and deep with well-developed flood plains. B stream types typically have moderately developed flood-prone areas in narrower but steeper valleys than D, C, and E types. Table TS3E–2 includes aerial and ground photos of the eight major stream types, A through G.

### Valley types

Stream type, width-to-depth ratio, sinuosity, entrenchment, and other morphological features are dependent upon the valley development. For an initial broad level association of stream types, valley types are invaluable to level I classification and as a general indication of morphological pattern. Table TS3E–3 provides more detailed description of the 11 valley types used in the Rosgen stream classification system.

### Channel slope—level I

Over substantial distances, elevations, and channel lengths can sometimes be determined from a combination of USGS 7.5-minute quadrangles and aerial photos. Aerial photographs can be used to calculate sinuosity. Thus, slope can often be calculated by first multiplying sinuosity times stream length from the quad maps, then second, by dividing the difference in elevation by the total sinuous stream length.

<table>
<thead>
<tr>
<th>Table TS3E–1</th>
<th>Plan view characteristics of Rosgen stream type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stream type</td>
<td>Plan view</td>
</tr>
<tr>
<td>Aa+, A</td>
<td>Relatively straight</td>
</tr>
<tr>
<td>B</td>
<td>Slightly sinuous</td>
</tr>
<tr>
<td>F, G</td>
<td>Moderately sinuous, F or Bs may have active point bars</td>
</tr>
<tr>
<td>C</td>
<td>Sinuous with active point bars</td>
</tr>
<tr>
<td>D</td>
<td>Multiple thread, braided</td>
</tr>
<tr>
<td>DA</td>
<td>Multiple thread, anastomosed</td>
</tr>
<tr>
<td>E</td>
<td>Tortuous and/or highly sinuous</td>
</tr>
</tbody>
</table>
Figure TS3E–1  Major stream types

Longitudinal, Cross-sectional, and Plan Views of Major Stream Types

Stream types: Aa+, A, B, C, D, DA, E, F, G

Dominant stage range: Aa+ > 10%, A 4 - 10%, B 2 - 4%, C < 2%, D < 4%, DA < 0.5%, E < 2%, F < 2%, G 2 - 4%

Cross section view and Plan view for each stream type.
### Table TS3E–2  
Aerial and surface views of major stream types

<table>
<thead>
<tr>
<th>Stream type and bedforms</th>
<th>Aerial view</th>
<th>Surface view</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td><img src="image1" alt="Aerial view" /></td>
<td><img src="image2" alt="Surface view" /></td>
</tr>
<tr>
<td>Step-pool and/or cascade and/or chute bed</td>
<td><img src="image3" alt="Aerial view" /></td>
<td><img src="image4" alt="Surface view" /></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stream type and bedforms</th>
<th>Aerial view</th>
<th>Surface view</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td><img src="image5" alt="Aerial view" /></td>
<td><img src="image6" alt="Surface view" /></td>
</tr>
<tr>
<td>Step-pool and/or plane-bed and/or pool-riffle</td>
<td><img src="image7" alt="Aerial view" /></td>
<td><img src="image8" alt="Surface view" /></td>
</tr>
</tbody>
</table>
### Table TS3E–2  
Aerial and surface views of major stream types—Continued

<table>
<thead>
<tr>
<th>Stream type and bedforms</th>
<th>Aerial view</th>
<th>Surface view</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>C</strong></td>
<td><img src="image1" alt="Aerial View" /></td>
<td><img src="image2" alt="Surface View" /></td>
</tr>
<tr>
<td>Pool-riffle and/or plane-bed and/or ripple-dune</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stream type and bedforms</th>
<th>Aerial view</th>
<th>Surface view</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>D</strong></td>
<td><img src="image3" alt="Aerial View" /></td>
<td><img src="image4" alt="Surface View" /></td>
</tr>
<tr>
<td>Braided</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Some pool-riffles develop on patterns of convergence and divergence</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table TS3E-2  Aerial and surface views of major stream types—Continued

<table>
<thead>
<tr>
<th>Stream type and bedforms</th>
<th>Aerial view</th>
<th>Surface view</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DA</strong></td>
<td><img src="image1" alt="Aerial View" /></td>
<td><img src="image2" alt="Surface View" /></td>
</tr>
<tr>
<td>Anastomose</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Braided pool-riffle or ripple-dune</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Stream type and bedforms</th>
<th>Aerial view</th>
<th>Surface view</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>E</strong></td>
<td><img src="image3" alt="Aerial View" /></td>
<td><img src="image4" alt="Surface View" /></td>
</tr>
<tr>
<td>Pool-riffle or ripple-dune</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depositonal bars sometimes not present</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
## Table TS3E–2  Aerial and surface views of major stream types—Continued

<table>
<thead>
<tr>
<th>Stream type and bedforms</th>
<th>Aerial view</th>
<th>Surface view</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>F</strong></td>
<td><img src="image1.png" alt="Aerial View" /></td>
<td><img src="image2.png" alt="Surface View" /></td>
</tr>
<tr>
<td>Pool-riffle or ripple-dune</td>
<td><img src="image3.png" alt="Aerial View" /></td>
<td><img src="image4.png" alt="Surface View" /></td>
</tr>
<tr>
<td>Depositonal bars sometimes not present</td>
<td><img src="image5.png" alt="Aerial View" /></td>
<td><img src="image6.png" alt="Surface View" /></td>
</tr>
<tr>
<td><strong>G</strong></td>
<td><img src="image7.png" alt="Aerial View" /></td>
<td><img src="image8.png" alt="Surface View" /></td>
</tr>
<tr>
<td>Step-pool with some instances of plane-bed forms</td>
<td><img src="image9.png" alt="Aerial View" /></td>
<td><img src="image10.png" alt="Surface View" /></td>
</tr>
</tbody>
</table>
Table TS3E–3  Valley type, morphological description, and stream type association

<table>
<thead>
<tr>
<th>Valley type</th>
<th>Description</th>
<th>Stream type association</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Steep V-shaped confined, highly dissected fluvial slopes greater than 2 percent</td>
<td>A and Aa+</td>
</tr>
<tr>
<td>II</td>
<td>Moderate relief gentle sloping side slopes with a parabolic valley bottom form often in colluvial valleys</td>
<td>B</td>
</tr>
<tr>
<td>III</td>
<td>Primarily depositional, usually steep, greater than 2 percent valley slope with debris-coluvium or alluvial fan landform</td>
<td>A, B, G, and D</td>
</tr>
<tr>
<td>IV</td>
<td>Gentle gradient canyons, gorges and confined alluvial valleys such as the Grand Canyon. Valley floors are typically less than 2 percent</td>
<td>F</td>
</tr>
<tr>
<td>V</td>
<td>U-shaped glacial-fluvial troughs with slopes generally less than 4 percent. Landforms typically include lateral or terminal moraines, alluvial terraces and flood plains. Trough is typically the result of glacial scouring process</td>
<td>C, D, G</td>
</tr>
<tr>
<td>VI</td>
<td>Fault control valleys, structurally controlled and dominated by colluvial slope building processes. Moderately steep with slopes less than 4 percent. G stream types observed under fault disequilibrium</td>
<td>Mostly B with C and F; some G</td>
</tr>
<tr>
<td>VII</td>
<td>Steep highly dissected fluvial slopes typically in either colluvium, alluvium or in residual soil. Active lateral and vertical accretion (Badlands of SD)</td>
<td>A and G</td>
</tr>
<tr>
<td>VIII</td>
<td>Mature wide gently valley slopes with well developed flood plain features adjacent to river terraces. Alluvial terraces and flood plains are predominate landforms. Depending on local streambed and riparian conditions D, F; and G stream types can be found. Gentle slopes with the alluvial valley fills</td>
<td>C and E, D, F, and G</td>
</tr>
<tr>
<td>IX</td>
<td>Glacial outwash and/or eolian sand dunes. Moderate to gentle slopes. High sediment supply either single- or multiple-threaded channels</td>
<td>C and D</td>
</tr>
<tr>
<td>X</td>
<td>Very broad and very gentle slopes with extensive flood plain development. Often associated with lacustrine and gentle alluvial slopes. G and F streams are common when local base grades have been changed</td>
<td>E or C and DA, G, and F</td>
</tr>
<tr>
<td>XI</td>
<td>Large river deltas and tidal flats constructed of fine alluvial materials originating from riverine and estuarine depositional processes. Extremely gentle slopes with base grade controlled by sea or lake levels. Most often distributary channels, wave, or tide dominated</td>
<td>DA C and E</td>
</tr>
</tbody>
</table>
Morphological description (level II classification)

The level II classification process provides a more detailed morphological description of the stream based on field collected data. It includes assessments of channel entrenchment, dimensions, patterns, profiles, and bed materials. It uses a more finely resolved hierarchical criterion to stratify types and address general characteristics such as sediment supply, stream sensitivity to disturbance, potential for natural recovery, channel responses to flow regime change, and fish habitat potential. The morphological description level requires the computation of the entrenchment ratio, width-to-depth ratio, sinuosity, slope, and $D_{50}$ or dominant particle size determination (fig. TS3E–2 (Rosgen 1996)).

Channel slope—level II

The channel slope should be determined for each stream reach being classified. It consists of the difference in elevation of the water surface or bed through the reach divided by the length of the channel reach. The elevations are usually obtained at the upper and lower ends of each bed feature. When water surface slope is field measured it is preferable to measure through at least two meander wavelengths. For example, the average slope is calculated from the top of the riffle to the top of the next downstream riffle (fig. TS3E–3). The channel length used in the calculation is the centerline length of the channel between the two points used for elevations.

In practice, the average low-flow water surface slope is the same as the average bankfull stage slope and is an accurate representation of slope needed for classification. The average slope of the water surface is generally measured through 20 to 30 channel widths. The higher water surface profile can be obtained from standard hydraulic methods.

Bankfull discharge validations

Dimensions measured and characterized in Rosgen’s geomorphic stream classification system are based on bankfull discharge. A complete description of bankfull discharge is provided in NEH654.05. It is advised that a field validation of bankfull discharge and associated return intervals be completed at USGS or similar gages. This validation is used to develop and/or check regional curves for specific hydro-physiographic areas.

Although a bankfull discharge of $1.5Q$ is generally considered to be the typical return interval (Williams 1978), it is not at all uncommon to find ranges between 1.1 to 2.0 or higher. Data show that a return interval difference from a 1.1- to a 1.5-year event can have as much as 68 percent more flow (Southerland 2003). The bankfull dimensions associated with this difference in flow can likely lead to incorrect and/or inconsistent classification.

Entrenchment

Entrenchment is a measure of the extent of vertical containment of a channel relative to its adjacent flood plain. Entrenchment is defined as the ratio of the width of the flood-prone area to the bankfull width of the channel (fig. TS3E–4). The flood-prone width is measured at an elevation of two times the maximum depth at the bankfull stage. In figure TS3E–4, the flood-prone width is 305 feet, and the bankfull width is 25 feet. The entrenchment ratio is 12.2.

The top of banks does not always indicate bankfull stage. In deeply entrenched channels, the flood-prone area may be contained entirely within the banks. Entrenchment ratios for various stream types are shown on figure TS3E–5 (Rosgen 1996). The flood-prone area is measured at the riffle facet of the profile at level II classification. The entrenchment ratio may vary by 0.2 units without necessarily changing the classification.

Usually, field measurements will be necessary. In some cases, widths of flood-prone areas and bankfull stages can be made from aerial photos and topographic maps at the level I characterization. It is recommended that a typical cross section of each stream reach and its associated flood plain be obtained.

Channel material

The channel material consists of the soil, rock, and vegetation that occur in the bed and banks of the channel. For classification, the dominant bed material is of primary interest. This consists of the sediment or rock
Figure TS3E–2  Hierarchical level II key

KEY to the Rosgen CLASSIFICATION of NATURAL RIVERS. As a function of the “continuum of physical variables” within stream reaches, values of Entrenchment and Sinuosity ratios can vary by +/- 0.2 units; while values for Width/Depth ratios can vary by +/- 2.0 units.
Figure TS3E–3  Measuring stream gradient

Surveyor’s rod located at top of a series of riffle-pool reaches and held at water surface; for a minimum of four locations, or three riffle-pool cycles

Surveyor’s level located on streambank near cross section

Note: Riffle to riffle gradient approximates the average water surface slope

Figure TS3E–4  Field measurement of entrenchment ratio

In this example:
Vertical height (6.5)÷distance (400 ft)=Gradient (.016 ft/ft)

In this example:
\( d = 2.5 \) ft
\( 2d = 5.0 \) ft

Cross sections  Figures are not to scale
exposed in the bed and on about the lowermost third of the banks of the stream reach. The measure used is the median grain size, or $D_{50}$, of the bed material. Several sampling traverses may be necessary to represent the channel material in a given stream reach (fig. TS3E–6). Plant material, leaves, and so forth are not counted as bed-load material. When organic material is found, bed load at the particle size sampling interval (usually 1-ft intervals) should be pulled from beneath. If the debris is too large for bed load extrication below it, consistently draw particles at the same 1-foot interval from the side of the woody debris, while remaining on the same path. A detailed description of sediment sampling is provided in NEH654 TS13A.

Based on the $D_{50}$ size, the channel material is classified into one of six particle size categories:

- 1 – bedrock (>2048 mm)
- 2 – boulder (256 mm to 2047.9 mm)
- 3 – cobble (64 mm to 255.9 mm)
- 4 – gravel (2 mm to 63.9 mm)
- 5 – sand (0.062 mm to 1.99 mm)
- 6 – silt/clay (<0.062 mm)

Plot material sizes showing cumulative and percent distributions on log normal scaled paper. Estimate $D_{50}$.

In figure TS3E–7, $D_{50}$ is 34 millimeters (gravel size).

Figure TS3E–5  Entrenchment ratios of major stream types

<table>
<thead>
<tr>
<th>Stream type</th>
<th>Entrenchment ratio</th>
<th>Figure TS3E–5</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.0-1.4*</td>
<td>Entrenched</td>
</tr>
<tr>
<td>B</td>
<td>1.41-2.2*</td>
<td>Moderately entrenched</td>
</tr>
<tr>
<td>C</td>
<td>2.2+*</td>
<td>Slightly entrenched</td>
</tr>
</tbody>
</table>

Entrenchment ratio = \( \frac{\text{Flood-prone width}}{\text{Bankfull width}} \)

*Entrenchment ratio may vary by ± 0.2 units

Flood-prone width = water level @ \( 2 \times \text{max/depth} \)
Figure TS3E–6  Pebble count

This reach is approximately 24 channel widths in length.

Example: Reach configured as:

<table>
<thead>
<tr>
<th></th>
<th>Pool</th>
<th>Riffle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>75 ft</td>
<td>175 ft</td>
</tr>
<tr>
<td>Percentage</td>
<td>30%</td>
<td>70%</td>
</tr>
</tbody>
</table>

Samples for pebble count: 30% in pools, 70% in riffles

Figure TS3E–7  Plotted particle size distribution
**Width-to-depth ratio**

This consists of the ratio of the top width of the channel at bankfull stage to the average depth at bankfull. The average depth is computed by dividing the cross-sectional area of the channel by the width. The width divided by the depth (width-to-depth ratio) is typically the most sensitive indicator in level II classification. However, the width-to-depth ratio can vary by 2.0 units without necessarily changing the classification (Rosgen 1996).

It is recommended that a typical cross section of each stream reach be surveyed and appropriate measurements obtained from a graphical plot of the cross section. The bankfull stage can be estimated from field evidence or predicted using standard hydrologic techniques.

**Sinuosity**

This measure indicates the degree of meandering and channel migration within a valley that the channel exhibits in plan view. It consists of the ratio of channel length to valley length. The channel length of the stream reach is measured along the thalweg of the channel. The valley length is the length of the reach measured along a line paralleling the local trend of the stream valley.

Aerial photographs of appropriate resolution, soil maps, geologic maps, and USGS 7.5 quadrangles provide convenient means for measuring sinuosity.

**Procedure**

The simplified version of Rosgen’s stream classification is implemented by applying the following procedure to each stream reach of interest.

Identify a reach of at least 20 bankfull widths. Define drainage area and use relative USGS gage data, if available.

**Step 1** Identify plan view type and determine whether the channel type is multiple-threaded (three or more channels) at bankfull.

**Step 2** Determine the entrenchment ratio.

**Step 3** Find the width and average depth of bankfull event, and compute the width-to-depth ratio.

**Step 4** Determine the channel slope (water surface).

**Step 5** Using the data from steps 1 and 2, figure TS3E–1, and table TS3E–1, assign a capital letter designation representing stream type. If results are ambiguous, give more weight to plan view type than channel slope or entrenchment. This corresponds to the level I classification of Rosgen (1994).

**Step 6** Determine $D_{50}$ size of the dominant bed material, and classify in terms of size (sand, gravel).

**Step 7** Assign a number from 1 to 6, depending on results of step 6. This corresponds to the level II classification of Rosgen (1994).

**Step 8** Determine the channel and valley lengths of the stream reach and compute sinuosity.

**Step 9** Check the values of channel slope, entrenchment ratio, width-to-depth ratio, and sinuosity against typical ranges for those values associated with the channel type (refer to fig. TS3E–2).

**Step 10** If steps 4 and 9 are completed with satisfactory results, proceed with interpretations.

**Step 11** If the stream reach fails to fit into a category in step 4 or if one or more values in step 9 lie outside the indicated ranges, additional studies are necessary. Anthropogenic alterations to some streams are so recent that the form may be in a transitory state and difficult to classify. However, with additional analysis a classification trend possibly may be identified. Table TS3E–4 (Harrelson, Rawlins, and Potyondy 1994) provides both the stream morphometry and landform features of the major stream types. Information such as this may aid in this further analysis.

**Interpretations and uses of the Rosgen stream classification system**

The Rosgen stream classification system is intended as an evaluation tool. It conveys important information about the stability of the stream reach and about the degree of compatibility that certain types of stream
<table>
<thead>
<tr>
<th>Stream type</th>
<th>General description</th>
<th>Entrench ratio</th>
<th>Width-to-depth ratio</th>
<th>Sinuosity</th>
<th>Slope</th>
<th>Landform/soils/features</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aa+</td>
<td>Very steep, deeply entrenched, debris transport streams</td>
<td>&lt;1.4</td>
<td>&lt;12</td>
<td>1.0 – 1.2</td>
<td>&gt;.10</td>
<td>Very high relief. Erosional, bedrock, boulder, or depositional features; debris flow potential. Deeply entrenched streams. Vertical steps with deep scour pools; waterfalls</td>
</tr>
<tr>
<td>A</td>
<td>Steep, entrenched, cascading, step-pool streams. High energy/debris transport with depositional soils. Very stable if bedrock or boulder-dominated channel</td>
<td>&lt;1.4</td>
<td>&lt;12</td>
<td>1.0 – 1.2</td>
<td>.04–10</td>
<td>High relief. Erosional bedrock forms. Entrenched and confined streams with cascading reaches. Frequently spaced, deep pools in associated step-pool bed morphology</td>
</tr>
<tr>
<td>B</td>
<td>Moderately entrenched, moderate gradient dominated channel, with infrequently spaced pools. Very stable plan and profile. Stable banks</td>
<td>1.4 – 2.2</td>
<td>&gt;12</td>
<td>1.2</td>
<td>&gt;.02</td>
<td>Moderate relief, colluvial riffle deposition, and/or residual soils. Moderate entrenchment and width-to-depth ratio. Narrow, moderately sloping valleys. Rapids predominate with occasional pools</td>
</tr>
<tr>
<td>C</td>
<td>Low gradient, meandering point-bar, riffle-pool, alluvial channels with broad, well-defined flood plains</td>
<td>&gt;2.2</td>
<td>&gt;12</td>
<td>1.4</td>
<td>&lt;.02</td>
<td>Broad valleys w/terraces, in association with flood plains, alluvial soils. Slightly entrenched with well-defined meandering channel. Riffle-pool bed morphology</td>
</tr>
<tr>
<td>D</td>
<td>Braided channel with longitudinal and transverse bars. Very wide channel with eroding banks</td>
<td>N/a</td>
<td>&gt;40</td>
<td>N/A</td>
<td>&lt;.04</td>
<td>Broad valleys with alluvial and colluvial fans. Glacial debris and depositional features. Active lateral adjustment with abundance of sediment supply</td>
</tr>
<tr>
<td>DA</td>
<td>Anastomosing (multiple channels) narrow and deep with expansive well-vegetated flood plain and associated wetlands. Very gentle relief with highly variable sinuosity's, stable streambanks</td>
<td>&gt;4.0</td>
<td>&lt;40</td>
<td>Variable</td>
<td>&lt;.005</td>
<td>Broad, low-gradient valleys with fine alluvium and/or lacustrine soils. Anastomosed (multiple channel) geologic control creating fine deposition with well-vegetated bars that are laterally stable with broad wetland flood plains. Stream type common in estuaries</td>
</tr>
<tr>
<td>E</td>
<td>Low gradient, meandering riffle-pool stream with low width-to-depth ratio and little deposition. Very efficient and stable. High meander width ratio</td>
<td>&gt;2.2</td>
<td>&lt;12</td>
<td>1.5</td>
<td>&lt;.02</td>
<td>Broad valley/meadows. Alluvial materials with flood plain and/or lacustrine soil. Highly sinuous with stable well-vegetated banks. Riffle-pool morphology with very low width-to-depth ratio</td>
</tr>
<tr>
<td>F</td>
<td>Entrenched meandering riffle-pool channel on low gradients with high width-to-depth ratio</td>
<td>&lt;1.4</td>
<td>&gt;12</td>
<td>1.4</td>
<td>&lt;.04</td>
<td>Entrenched in highly weathered material. Gentle gradients usually less than .02 ft/ft, but may range up to .04 ft/ft with a high width-to-depth ratio. Meandering, laterally unstable with high bank erosion rates. Riffle-pool morphology.</td>
</tr>
<tr>
<td>G</td>
<td>Entrenched gully step-pool and low width-to-depth ratio on moderate gradients</td>
<td>&lt;1.4</td>
<td>&lt;12</td>
<td>1.2</td>
<td>.02–.039</td>
<td>Gully, step-pool morphology with moderate slopes and low width-to-depth ratio. Narrow valleys, or deeply incised in alluvial or colluvial materials (fans or deltas). Unstable with grade control problems and high bank erosion rates</td>
</tr>
</tbody>
</table>
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

<table>
<thead>
<tr>
<th>Stream type</th>
<th>Sensitivity to disturbance</th>
<th>Recovery potential</th>
<th>Sediment supply</th>
<th>Streambank erosion potential influence</th>
<th>Vegetation controlling</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Very low</td>
<td>Excellent</td>
<td>Very low</td>
<td>Very low</td>
<td>Negligible</td>
</tr>
<tr>
<td>A2</td>
<td>Very low</td>
<td>Excellent</td>
<td>Very low</td>
<td>Very low</td>
<td>Negligible</td>
</tr>
<tr>
<td>A3</td>
<td>Very high</td>
<td>Very poor</td>
<td>Very high</td>
<td>High</td>
<td>Negligible</td>
</tr>
<tr>
<td>A4</td>
<td>Extreme</td>
<td>Very poor</td>
<td>Very high</td>
<td>Very high</td>
<td>Negligible</td>
</tr>
<tr>
<td>A5</td>
<td>Extreme</td>
<td>Very poor</td>
<td>Very high</td>
<td>Very high</td>
<td>Negligible</td>
</tr>
<tr>
<td>A6</td>
<td>High</td>
<td>Poor</td>
<td>High</td>
<td>High</td>
<td>Negligible</td>
</tr>
<tr>
<td>B1</td>
<td>Very low</td>
<td>Excellent</td>
<td>Very low</td>
<td>Very low</td>
<td>Negligible</td>
</tr>
<tr>
<td>B2</td>
<td>Very low</td>
<td>Excellent</td>
<td>Very low</td>
<td>Very low</td>
<td>Negligible</td>
</tr>
<tr>
<td>B3</td>
<td>Low</td>
<td>Excellent</td>
<td>Low</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>B4</td>
<td>Moderate</td>
<td>Excellent</td>
<td>Moderate</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>B5</td>
<td>Moderate</td>
<td>Excellent</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
<tr>
<td>B6</td>
<td>Moderate</td>
<td>Excellent</td>
<td>Moderate</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>C1</td>
<td>Low</td>
<td>Very good</td>
<td>Very low</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>C2</td>
<td>Low</td>
<td>Very good</td>
<td>Very low</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>C3</td>
<td>Moderate</td>
<td>Good</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Very high</td>
</tr>
<tr>
<td>C4</td>
<td>Very high</td>
<td>Good</td>
<td>High</td>
<td>Very high</td>
<td>Very high</td>
</tr>
<tr>
<td>C5</td>
<td>Very high</td>
<td>Fair</td>
<td>Very high</td>
<td>Very high</td>
<td>Very high</td>
</tr>
<tr>
<td>C6</td>
<td>Very high</td>
<td>Good</td>
<td>High</td>
<td>Very high</td>
<td>Very high</td>
</tr>
<tr>
<td>D3</td>
<td>Very high</td>
<td>Poor</td>
<td>Very high</td>
<td>Very high</td>
<td>Moderate</td>
</tr>
<tr>
<td>D4</td>
<td>Very high</td>
<td>Poor</td>
<td>Very high</td>
<td>Very high</td>
<td>Moderate</td>
</tr>
<tr>
<td>D5</td>
<td>Very high</td>
<td>Poor</td>
<td>Very high</td>
<td>Very high</td>
<td>Moderate</td>
</tr>
<tr>
<td>D6</td>
<td>High</td>
<td>Poor</td>
<td>High</td>
<td>High</td>
<td>Moderate</td>
</tr>
<tr>
<td>DA4</td>
<td>Moderate</td>
<td>Good</td>
<td>Very low</td>
<td>Low</td>
<td>Very high</td>
</tr>
<tr>
<td>DA5</td>
<td>Moderate</td>
<td>Good</td>
<td>Low</td>
<td>Low</td>
<td>Very high</td>
</tr>
<tr>
<td>DA6</td>
<td>Moderate</td>
<td>Good</td>
<td>Very low</td>
<td>Very low</td>
<td>Very high</td>
</tr>
<tr>
<td>E3</td>
<td>High</td>
<td>Good</td>
<td>Low</td>
<td>Moderate</td>
<td>Very high</td>
</tr>
<tr>
<td>E4</td>
<td>Very high</td>
<td>Good</td>
<td>Moderate</td>
<td>High</td>
<td>Very high</td>
</tr>
<tr>
<td>E5</td>
<td>Very high</td>
<td>Good</td>
<td>Moderate</td>
<td>High</td>
<td>Very high</td>
</tr>
<tr>
<td>E6</td>
<td>Very high</td>
<td>Good</td>
<td>Low</td>
<td>Moderate</td>
<td>Very high</td>
</tr>
<tr>
<td>F1</td>
<td>Low</td>
<td>Fair</td>
<td>Low</td>
<td>Moderate</td>
<td>Low</td>
</tr>
<tr>
<td>F2</td>
<td>Low</td>
<td>Fair</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Low</td>
</tr>
<tr>
<td>F3</td>
<td>Moderate</td>
<td>Poor</td>
<td>Very high</td>
<td>Very high</td>
<td>Moderate</td>
</tr>
<tr>
<td>F4</td>
<td>Extreme</td>
<td>Poor</td>
<td>Very high</td>
<td>Very high</td>
<td>Moderate</td>
</tr>
<tr>
<td>F5</td>
<td>Very high</td>
<td>Poor</td>
<td>Very high</td>
<td>Very high</td>
<td>Moderate</td>
</tr>
<tr>
<td>F6</td>
<td>Very high</td>
<td>Fair</td>
<td>High</td>
<td>Very high</td>
<td>Moderate</td>
</tr>
<tr>
<td>G1</td>
<td>Low</td>
<td>Good</td>
<td>Low</td>
<td>Low</td>
<td>Low</td>
</tr>
<tr>
<td>G2</td>
<td>Moderate</td>
<td>Fair</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Low</td>
</tr>
<tr>
<td>G3</td>
<td>Very high</td>
<td>Poor</td>
<td>Very high</td>
<td>Very high</td>
<td>High</td>
</tr>
<tr>
<td>G4</td>
<td>Extreme</td>
<td>Very poor</td>
<td>Very high</td>
<td>Very high</td>
<td>High</td>
</tr>
<tr>
<td>G5</td>
<td>Extreme</td>
<td>Very poor</td>
<td>Very high</td>
<td>Very high</td>
<td>High</td>
</tr>
<tr>
<td>G6</td>
<td>Very high</td>
<td>Poor</td>
<td>High</td>
<td>High</td>
<td>High</td>
</tr>
</tbody>
</table>

1/ Includes increases in streamflow magnitude and timing and/or sediment increases
2/ Assumes natural recovery once cause of instability is corrected
3/ Includes suspended and bed load from channel derived sources and/or from stream adjacent slopes
4/ Vegetation that influences width-to-depth ratio stability
stream processes, sensitivity, and behavior. However, this sort of assessment needs to be made within the context of the topographic setting, as well as the channel evolution and watershed history.

The Rosgen stream classification system, as with many classification systems, describes a static condition that is not necessarily related to a specific process or change and, therefore, does not provide a direct mechanism for predicting a new stable channel form in disturbed watersheds (Gillian 1996, Cherry, Wilcock, and Wolman 1996). In addition, due to the dependence of the classification upon the present morphological characteristics, the approach does not have the ability to take into account previous or anticipated hydrologic changes. The classification of a stream to a particular type does not, by itself, imply that a stream is stable or unstable. It only indicates that the stream pattern, dimension, profile, and bed material are within the specified limits and variances of the classification system.

**Trends and dominant processes**

The Rosgen stream classification can be used to assess general trends in stream behavior and also to provide a guide to the dominant processes that a stream system can experience. Table TS3E–6 summarizes the characteristics of the Rosgen stream types by watershed conditions.

It is important to recognize that the science of fluvial geomorphology is based primarily on observation. As a result, predicted trends and changes tend to represent average conditions. Assessment and design for a specific project area requires the use of physically based calculations (Goodwin 1999).
### Table TS3E-6  Summary of characteristics of Rosgen stream types by watershed conditions

<table>
<thead>
<tr>
<th>Rosgen stream type</th>
<th>Watershed type</th>
<th>Sediment load</th>
<th>Energy of stream</th>
<th>Energy dissipation in stream is typically by:</th>
<th>May be appropriate for design in:</th>
</tr>
</thead>
</table>
| 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 |

(210–VI–NEH, August 2007)
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.
Developing Regional Relationships for Bankfull Discharge Using Bankfull Indices
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.

(210–VI–NEH, August 2007)
### Contents

<table>
<thead>
<tr>
<th>Purpose</th>
<th>TS5–1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Regional curves</td>
<td>TS5–1</td>
</tr>
<tr>
<td>USGS gage station selection criteria</td>
<td>TS5–1</td>
</tr>
<tr>
<td>Stability of channel and drainage area network</td>
<td>TS5–1</td>
</tr>
<tr>
<td>Gages on streams with flood control or diversions</td>
<td>TS5–2</td>
</tr>
<tr>
<td>Urban versus rural land use drainage areas</td>
<td>TS5–2</td>
</tr>
<tr>
<td>Equipment and human resources</td>
<td>TS5–2</td>
</tr>
<tr>
<td>In-office data collection</td>
<td>TS5–2</td>
</tr>
<tr>
<td>Gage data</td>
<td>TS5–2</td>
</tr>
<tr>
<td>Use of discharge notes</td>
<td>TS5–3</td>
</tr>
<tr>
<td>Site reconnaissance</td>
<td>TS5–4</td>
</tr>
<tr>
<td>Station setups, benchmarks and reference marks</td>
<td>TS5–5</td>
</tr>
<tr>
<td>Cross sections</td>
<td>TS5–5</td>
</tr>
<tr>
<td>Profiles</td>
<td>TS5–6</td>
</tr>
<tr>
<td>Bankfull indicators</td>
<td>TS5–6</td>
</tr>
<tr>
<td>Characterization of bed material</td>
<td>TS5–7</td>
</tr>
<tr>
<td>Data processing and analysis</td>
<td>TS5–7</td>
</tr>
<tr>
<td>HEC–RAS model input</td>
<td>TS5–7</td>
</tr>
<tr>
<td>Calibrating to USGS rating curves</td>
<td>TS5–7</td>
</tr>
<tr>
<td>Selecting the channel-forming discharge</td>
<td>TS5–7</td>
</tr>
<tr>
<td>Hydraulic geometry relationships at bankfull</td>
<td>TS5–7</td>
</tr>
<tr>
<td>Figures</td>
<td>Description</td>
</tr>
<tr>
<td>--------------</td>
<td>-------------------------------------------------------</td>
</tr>
<tr>
<td>Figure TS5–1</td>
<td>Example plan view of a site</td>
</tr>
<tr>
<td>Figure TS5–2</td>
<td>Station setup is just upstream of gage house</td>
</tr>
<tr>
<td>Figure TS5–3</td>
<td>Cross-sectional survey</td>
</tr>
<tr>
<td>Figure TS5–4</td>
<td>Comparison of USGS rating curve with HEC–RAS</td>
</tr>
<tr>
<td>Figure TS5–5</td>
<td>Regional curves for hydraulic geometry</td>
</tr>
</tbody>
</table>
Purpose

This technical supplement presents a basic approach for the development of regional relationships for bankfull discharge using bankfull indices. This technical supplement provides guidelines to identifying bankfull stages along riparian stream corridors and procedures to determine the bankfull discharge associated with the bankfull stage. The bankfull discharge is used as a surrogate for the channel-forming discharge. While this technical supplement is primarily focused on the development of curves that are used in the Rosgen geomorphic channel design approach (NEH654.11), they are applicable to other assessment and design tools, as well.

Regional curves

Regional curves are constructed from observations and measurements of stable riffle cross sections on gaged rivers and streams. They are empirical by nature. The measured bankfull data are plotted versus the contributing drainage area flowing through the measured cross section(s). The regression equations express mathematical relationships between the bankfull channel dimensions: cross-sectional area, top width, mean depth, and the contributing drainage area.

Regional curves are a useful planning tool for natural stream design, stream restoration/stream enhancement, and fish habitat improvement or enhancement projects. They may provide estimations of the bankfull channel dimensions and bankfull discharge for any unaged 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, vegetative cover, land use, and management practices all affect the timing and magnitudes of runoff and, therefore, affect the size of the bankfull channels. Mathematically, better correlations exist between the bankfull hydraulic geometry and bankfull discharge. In watersheds with similar drainage areas, magnitude and duration of bankfull discharges can vary and, hence, hydraulic geometries at bankfull stage will vary due to the shape and cover of the watershed. Regime curves (hydraulic geometry vs. effective discharge) improve correlation over regional curves. Hydraulic geometry is described in further detail in NEH654.07 and NEH654.09.

USGS gage station selection criteria

Regional curves are constructed from stream survey measurements at U.S. Geological Survey (USGS) gaging stations. The period of record should accurately reflect the expected land uses and reported drainage size of the watershed. Gage data should be collected to represent a wide array of drainage areas with similar selection criteria.

Selection criteria for USGS gages to be used for regional curves are based on several factors: length of record, channel and drainage network stability, land use (rural vs. urban), drainage area size and shape, and the degree of (flood) control within the watershed. These criteria should match for the gages used to develop the regional curves. A combination of factors must be avoided. In addition, the criteria for the gages used to develop the curve should also match the intended project or evaluation area. Additional issues are described.

Stability of channel and drainage area network

The period of record should accurately reflect the expected land uses and reported drainage size of the watershed for the entire period of record. It may be difficult to recognize a natural drainage system that has undergone some degree of land use change or instability in its past. When researching gages, look for clues which may indicate that the watershed's hydrology and sediment production has changed over the length of record. Such clues may be obtained from a series of historic aerial photos, land use maps, mining records, road development, grazing or farm practices, development patterns, or fire records. These practices can affect the timing and volume of runoff, as well as the sediment production in a given watershed.
Gages on streams with flood control or diversions

Bankfull channel dimensions normally correspond to the uncontrolled drainage area. The assumption behind taking measurements of streams at known drainage areas and discharges is that a link can be made between the stream’s geometric parameters and peak rates of discharge with uncontrolled drainage areas. Dams, detention basins, and diversions affect the watershed hydrology by storing part or all of the peak discharges and releasing an attenuated flow regime. This will also affect sediment transport, which is critical to channel formation and maintenance. Dams and storage basins may store all incoming bed-material load and only a small portion of the finer wash load. Flood control affects the timing of peak discharges and may prevent the normal bankfull event from occurring downstream of the impoundment. Therefore, it is generally recommended to avoid gaging stations in watersheds that have flood control and water diversions.

Urban versus rural land use drainage areas

Urbanization generally increases the amount of impervious surface in a watershed. Additionally, stormwater and sewer conveyance systems combine to increase the volume of runoff and magnitude of runoff for a given storm event. Urbanization of a drainage area tends to reduce the recurrence interval for the bankfull event; from say, a 1.5-year recurrence interval to a 1.2-year recurrence interval. Urbanization may also increase flow velocities, increasing the forces and stresses imposed on the beds and banks of the channels. This is brought about in a number of ways such as changes in alignment (meander patterns, cutoffs, and increased stream gradient), encroachment on the flood plain, reduction in the boundary roughness, and changes in the median bed-material particle sizes. Urban areas are unique and should be separated from natural drainage areas to account for these changes in hydrology and sediment transport regimes. For example, regional curves may be constructed separately to represent natural forested and/or rangeland areas, rural farmed areas, or urban areas.

Equipment and human resources

Creating regional curves requires a team for collecting, analyzing, extracting, and transforming data into information. The size of the survey crew varies, depending on the site and intended use of the curves.

Surveys can be conducted with any standard survey instruments including a theodolite, total station, automatic level, or a laser level. The following equipment is usually needed to conduct these surveys:

- stable tripod
- telescoping rods, prisms
- two-way radios
- field notebook(s)
- compass
- measuring tape
- camera
- waders
- flagging
- station pins and nails
- orange vests
- personal flotation devices
- ruler (in millimeters, for rocks)
- data collection sheets

Other items may include a Global Positioning System (GPS) unit for precise locations, range finder to expedite the surveying process, buckets and shovel for sampling bed and bank materials, set of sieves for determining grain size, and scale for weighing samples.

In-office data collection

Gage data

The USGS Web site (http://waterdata.usgs.gov/nwis/rt) contains information such as station name and number, latitude and longitude of the gage site, drainage area, period of record, number of years of record,
and peak annual discharges and corresponding peak gage heights for each year in the period of record (n years). An estimate of the 1.5-year discharge is derived using techniques described in NEH654.05 and is used as a surrogate for bankfull discharge recurrence interval.

For the gages of interest, the practitioner should contact the local USGS data chief and request station descriptions, current rating tables, and summaries of discharge measurement notes. However, it is important to note that while each gage station may have a unique rating curve, the relationship between gage height and discharge is not necessarily unique. The rating curve may shift over the long term, as the cross-sectional shape and/or elevation changes, and it may shift over the course of a hydrograph due to the unsteady loop effect or due to changing bedforms.

The user should collect an existing gage analysis, or information sufficient to conduct such an analysis, following the procedures described in NEH654.05. By cross-referencing the estimated flows with the rating table, the user can define specific gage heights as the elevations for specific return intervals or specific chances of exceedance.

Discharge measurement notes are useful in that they provide specific cross-sectional flow areas, top widths of flow, and velocities for specific gage heights and discharge measurements. Plotting measurements below and above the bankfull discharge allows the user to estimate flow area, top width, and velocity at the bankfull or channel-forming flow.

Aerial photographs, topographic maps, and geology maps of the watershed of interest should be examined. These maps and photos can reveal details of the watershed and land use patterns that indicate the conditions that help shape the drainage network.

The topographic maps will also provide information required for stream characterization that is not easily obtained from the ground survey. Reach slope, sinuosity, and meander belt width can be estimated from these maps. Average reach slope can be estimated from the topographic maps by measuring the planimetric distance between contour intervals. It is recommended that the practitioner identify two to four consecutive contour intervals both downstream and upstream from the gage and measure the streamwise distance between the contour intervals.

A cross-reference to the USGS rating curve will provide the gage height for the 1.5-year discharge. When at the site, the user should locate the staff gage and identify a relatively flat depositional feature above or below this gage height corresponding to the bankfull discharge. If the staff gage has been removed, the user should locate an existing reference mark (from the station description) that refers back to a gage elevation. With a measuring tape, measure up or down to the gage height corresponding to the 1.5-year discharge, and again, look for the first flat depositional feature around this elevation. The practitioner should study this feature and the corresponding material size. Of particular interest are moss lines, debris lines, changes in slope and other distinguishing features. The elevation of these features relative to the water surface may be useful in identifying bankfull stages away from the gage site.

Note that the bankfull discharge elevation may vary significantly from the 1.5-year recurrence interval that is a normal surrogate for bankfull discharge in natural streams. As stated previously, the recurrence interval for the bankfull flow may be more frequent in developed watersheds.

**Use of discharge notes**

Discharge measurement notes can also provide insight into the hydraulic characteristics of the stream. Discharge measurement notes are a summary of discharge measurements taken throughout the period of record. They include date of measurement, gage height, discharge, top width of water in the cross section, cross-sectional area of flow, and mean velocity in the measurement cross section. The location where measurements take place is usually described in the station description. It is common to have two cross-sectional locations—one on the control feature of the stream for low-flow measurements and one across the bridge for high flows.

Energy slope and Manning’s $n$ are not included in the measurement notes. After calculating an average reach slope from topographic maps, Manning’s $n$ can be calculated using Manning’s equation by approximating the hydraulic radius by the hydraulic depth $d = \text{flow}$.
area/top width from the discharge measurement notes for a given discharge. Manning’s $n$ can vary considerably with depth of flow. Streams characteristically have high roughness at low flows and become hydraulically smoother as depth of flow increases. It is also important to note that this is a normal depth assumption and may not represent the flow levels due to any backwater effects that may occur. More information on the normal depth assumptions and computer modeling approaches is provided in NEH654.06.

Site reconnaissance

Before setting up the surveying equipment, a reconnaissance along the reach is prudent to select optimum station setups and minimize the overall number of setups and turns. During this reconnaissance, team members should assess the reach to determine if it is a stable form of the river, as it would have developed under natural conditions. The team can make this assessment by asking the following questions:

- Is there a low water ford or cattle access present that changes the channel geometry?
- Is accelerated bank erosion occurring?
- Are there undercut banks and trees falling in?
- Has bank vegetation been grazed, removed, sprayed, or cleared away?
- Is there one long continuous pool upstream from the gage?

The team should also assess the location of sections to be surveyed. Identification of riffle locations or the heads of glides, selection of cross-sectional locations, flagging bankfull indicators, and deciding the length of the reach to survey prior to setup may actually save field time. Figure TS5–1 provides an example of a sur-
vey using a total station survey instrument and shows station setups, benchmarks, thalweg profile, bank lines, cross sections, instream weir, and pipe crossing.

**Station setups, benchmarks, and reference marks**

The first step in beginning a survey is to tie survey elevations into the gage datum using the USGS reference marks. From the USGS station gaging description, the team should find all existing reference marks. These are published elevations with respect to the gage datum and allow the survey to be tied to an official datum. These marks may be chiseled Xs or chiseled squares on bridge abutments, gage houses, elevations of check bars on an outside wire weight gage, USGS or U.S. Department of Transportation (DOT) brass caps, staff gages, or bolts in trees or telephone poles. A shovel may be needed to scrape away dirt and overgrown weeds over concrete surfaces. The team should assure that at least two reference marks are visible from the initial station setup.

When using a total station instrument, the resection method for determining the station location requires coordinates for two known elevations. A measuring tape, compass, and calculator will be required to determine these coordinates in northings and eastings. The coordinate system may be arbitrary on setup, but afterwards, it must remain consistent, or the true alignment will be lost. Figure TS5–2 shows a station setup just upstream of a gage house along a riffle section where USGS discharge measurements are conducted.

**Cross sections**

Estimating a bankfull discharge may be accomplished by surveying a single section that is upstream of the gage and correlating it to the gage rating curve. However, for regional curve development, several cross sections for two to three full meander wavelengths for a detailed HEC–RAS model is recommended. Since the profile of the river reach will vary between the relatively steep riffle sections and the long relatively flat pool sections, the use of the HEC–RAS model will allow the practitioner to reconstruct the bankfull water surface elevations along the survey reach back to the gage site and, ultimately, prepare the rating table to determine discharges.

The survey data are to be used to develop a HEC–RAS model, so a cross section that represents the rating table is required. This cross section will be very important in calibrating the model. The station description usually describes where in the reach (in relation to the gage) low-flow discharges are measured. More than likely, this is on a riffle or upstream in a pool from a manmade control point, such as a cross-channel weir. Surveying a cross section over the end of the pressure transducer pipe is also wise, for this may be the section that represents the USGS rating table. Several cross sections should be surveyed downstream from the gage. The furthest one from the gage must be sufficiently far enough downstream that any erroneous assumptions of starting the flow conditions at normal depth are negligible at the gaging cross section. A good location for the first cross section may be in the next downstream riffle section, usually six to eight bankfull widths downstream.

The practitioner should survey several cross sections in the middle of three to four riffle sections above the gage cross section. This will help assure that the average reach geometry is not dependent on just one or two cross sections. All cross sections should start at or above the 100-year flood plain, or high on the valley wall, and extend across the valley to the opposite valley wall, or end above the 100-year flood plain. It
is normal protocol to define a cross section looking downstream with the stationing (in the cross section) increasing from left to right.

Figure TS5–3 shows a cross-sectional view near the gage house shown in figure TS5–2. The HEC–RAS computed rating curve at this cross section was compared to the USGS rating curve to complete calibration. Note in the cross-sectional view that the bankfull elevation corresponds to the top of a gravel bar feature near the left bank.

Profile—channel bed, water surface, bankline, flood plain, and terraces

Between cross sections, the survey should locate the thalweg profile, water depth, bankline profiles, and flat depositional features adjacent to the stream, known as the active flood plain. With a four-person team, one person operates the instrument with three people each with a survey rod; one along the right bank, one along the thalweg, and one along the left bank. This technique lends itself well to defining bankfull elevations because there will be at least two opinions on bankfull features. Every shot of the survey should include a recorded description of the particle size of the bed material that is found under the survey rod.

Bankfull indicators

Bankfull flow elevations and discharges are associated with sediment transport and, therefore, are closely tied to particle sizes moved and deposited in gravel and cobble dominated bed streams. In sand-bed streams, there may not be a differentiation of particle sizes from the channel and the active flood plain, but there

Figure TS5–3  Cross-sectional survey
should be a break in slope. Flat depositional features, breaks in slope, height of point bars, and vegetation features are other bankfull indicators that should be used. One of many bankfull indicators is a change in particle size distribution from gravels to fine grained sands. More information on bankfull indicators is provided in NEH654.05.

**Characterization of bed material**

The typical technique used for sampling the bed material is the Wolman pebble count. Wolman pebble counts are conducted in the riffle sections for several purposes and are described in more detail in NEH654 TS13A.

**Data processing and analysis**

**HEC–RAS model input**

The cross-sectional data are used to build a conventional HEC–RAS hydraulic model. It is recommended to use the thalweg stationing to set the channel distances between cross sections (required input to HEC–RAS model). All water surface elevations generated by the model will be in reference to the channel distances, which may be different from the bankline distances.

**Calibrating to USGS rating curves**

After the initial input of cross-sectional data, a HEC–RAS computational model run can be made to determine if the model has sufficient cross-sectional data to compute the actual water surface elevations recorded along the reach measured during the day of survey. Plotting computed water surface elevations along with channel bed and measured water surface elevations is helpful in pointing out areas along the profile that could use refinement or more definition. Depending on the level of agreement, additional refinement may be done by either returning to the field to take more measurements or by adding in interpolated cross sections based on the thalweg profile. It should be noted that this approach may be problematic in streams where the flows were very low at the time of the survey.

When the model definition is robust enough to match measured low-water surface elevations, calibration of the model by changing Manning’s coefficients and contraction/expansion coefficients can proceed to match the USGS rating curve at the gaging cross section. Figure TS5–4 shows a comparison of rating curves between the calibrated model results at the gage cross section. As shown in this figure, the model calibration is good up to discharges of 4,000 cubic feet per second, which is well beyond the bankfull discharge of 1,420 cubic feet per second.

**Selecting the channel-forming discharge**

Once the model is calibrated to the USGS rating curve, a selection of the channel-forming discharge can be made. This will entail running a range of discharges in the HEC–RAS model and comparing computed water surface elevations along the longitudinal profile to measured bankfull indicators and associated bankfull elevations. The criterion for consistency is that the profile of bankfull-stage elevations should plot approximately parallel to the longitudinal profile of the water surface at some given discharge through the reach (Kilpatrick and Barnes 1964). The channel-forming discharge is the discharge that comes closest to the surveyed bankfull indicators: flood plains, benches, breaks in slope, change in particle sizes, and vegetation indicators along the reach.

**Hydraulic geometry relationships at bankfull**

Once the channel-forming discharge or bankfull discharge is known and the corresponding water surface elevations computed, the hydraulic geometry in the stable riffle cross sections can be estimated. Cross-sectional flow area, hydraulic radius, hydraulic depth, and top width can be selected as output variables from the HEC–RAS Profile Output Table. The hydraulic geometry for the reach is best represented by an average of three or four stable riffle cross sections. The hydraulic geometry relationships at bankfull should then be plotted with respect to drainage area on the regional curve (fig. TS5–5). These relationships are useful in a variety of channel assessment and design applications.
Figure TS5–4  Comparison of USGS rating curve with HEC–RAS
Figure TS5–5  Regional curves for hydraulic geometry

- **DA vs. Q**: $Q = 16.9 \times DA^{1.07}$ (Corr = 0.90)
- **DA vs. XSA**: $XSA = 11.32 \times DA^{0.76}$ (Corr = 0.90)
- **DA vs. T**: $T = 11.33 \times DA^{0.476}$ (Corr = 0.85)
- **DA vs. d (mean depth)**: $d = 1.0 \times DA^{0.284}$ (Corr = 0.70)

**Drainage area (mi²)** vs. **Discharge (ft³/s), area (ft²), width and depth (ft)**

- **Q = 1,450 ft³/s/1.3 yr RI**
- **XSA = 268 ft²**
- **Top width = 90 ft**
- **Mean depth = 3.0 ft**
Guidelines for Sampling Bed Material
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.

(210–VI–NEH, August 2007)
Contents

Purpose of bed-material sampling  
Site selection to representative sampling
Sand-bed streams
  Cross-sectional approach
  Reach approach
Gravel-bed streams
  Surface sediment sampling
  Sediment intrusion into spawning gravels
Selection of a sampling procedure
Step-by-step field sampling procedures
Other bed-material characteristics
Bank material
Conclusion

Tables

Table TS13A–1  Bed-material sampling sites

Figures

Figure TS13A–1  Gradation pattern on a point bar
Figure TS13A–2  Bed sampling locations for sand-bed streams
Figure TS13A–3  Gravel-bed sediment profile showing vertical variation
Figure TS13A–4  Coarse-bed stream sampling hierarchy
Figure TS13A–5  Gravelometer held above stream
Figure TS13A–6  Piston sampler
Purpose of bed-material sampling

The characteristics of a given stream are linked to the composition of the material that comprises its channel bed, bank, and sediment flow. Knowledge of streambed material is necessary for a variety of engineering and environmental purposes. The size and gradation of the streambed material may affect the source, transport, and fate of pollutants; fish habitat; resource management; morphological trends; and stream restorations.

Bed-material sampling programs must be carefully designed to meet the particular needs of a specific study. Studies may include objectives related to the following:

- **Contaminants**—Typically attach to cohesive sediment and, therefore, are distributed over a wide area, especially in areas where flow velocity is low. Sampling for a contaminant concentrates on depositional zones in the stream and overbank.

- **Aquatic habitat**—Fish habitat studies may focus on the suitability of the streambed for spawning. Sampling for this type of study is often extensive, identifying lateral, longitudinal, and temporal variations in the surface layer over a wide area of the stream. An assessment of vertical variations may also be of critical importance, as the composition of the material immediately below the surface, especially the fines content, may be of importance in the evaluation of spawning habitats for some species.

- **Gravel mining**—Resource management studies are frequently concerned with the need or feasibility of sand and gravel mining. Core or substrate sampling that identifies vertical variation of the streambed is essential for this type of study.

- **Stream assessment and design**—Morphologic and engineering studies are concerned with changes in the character of the river over time. These studies require knowledge of the grain size distribution of both the bed surface material and subsurface material for sediment transport calculations, critical shear stress determinations, determining potential for particle sorting and armoring, and determining hydraulic roughness.

Complex studies may need to secure data to meet a combination of objectives and purposes. However, sediment data collected for one purpose will not necessarily be applicable for another. While the issues and recommendations presented here are generally applicable, the focus is on bed sampling for stream assessment and design.

Site selection for representative sampling

Sufficient sampling of the streambed should be conducted to determine the spatial variability, size, and gradation of the bed material. No simple rule exists for locating representative sampling sites or reaches. The general rule is to carefully select sampling locations and avoid anomalies that would bias either the calculated sediment discharge or the calculated bed stability. Sampling locations must be representative of the hydraulic and sedimentation processes that occur in that reach of the river. The site should be morphologically stable. To ensure data reflect reach-averaged river conditions, there should be no tributary inflow in the proximity of the site, as it may interfere with the homogeneity of the section by supplying sediment for deposition. The site should not be located adjacent to a zone of active bank erosion, as the material deposited in the channel near the eroding area may not be representative of the reach. Although bridges provide good access, bridge crossings are typically not appropriate sampling sites because either they are located at natural river constrictions or their abutments and piers create constrictions and local scour. Dead-water areas behind sand bars or other obstructions should be avoided, as these are not representative of average flow conditions.

The location of the bed sample should be chosen with the target analysis in mind. Table TS13A–1 provides guidance for where a bed-material sample might be taken as a function of the type of geomorphic 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>
<thead>
<tr>
<th>Purpose of analysis</th>
<th>Sample location</th>
</tr>
</thead>
<tbody>
<tr>
<td>To estimate the maximum permissible velocity in a</td>
<td>Rifflle</td>
</tr>
<tr>
<td>threshold stream</td>
<td></td>
</tr>
<tr>
<td>To estimate the minimum permissible velocity in a</td>
<td>Areas of local deposition</td>
</tr>
<tr>
<td>threshold stream</td>
<td></td>
</tr>
<tr>
<td>To estimate sediment yield for an alluvial stream</td>
<td>Crossing or middle bar</td>
</tr>
<tr>
<td>To quantify general physical habitat substrate</td>
<td>Bars, riffles, and pools</td>
</tr>
<tr>
<td>condition</td>
<td></td>
</tr>
</tbody>
</table>
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.

**Reach approach**

An alternative to the cross-sectional approach is the reach approach. A reach is defined as a portion of the stream with similar morphology (identified by its homogeneity). Generally, five cross sections are laid out in the homogeneous reach. If there is a gage in the reach, locating the center cross section near the gage is preferred. This facilitates relating the sediment data to measured hydrologic and hydraulic data. If the stream reach is straight, the spacing of the cross sections should be approximately two to five stream widths, and if the reach is meandering, the spacing should occur within one meander length (fig. TS13A–2). The same criteria used in the cross-sectional approach to determine the number of verticals to take along each section are applied here. The reach approach applies best to rivers with meanders of different wavelengths and amplitudes.

**Gravel-bed streams**

Coarse beds (gravel, cobble, and boulder) are characterized by significant vertical, spatial, and temporal bed-material variability. A vertical stratification in the bed material can be formed as the finer material is winnowed from the surface. A sketch of the resulting sediment profile is provided in figure TS13A–3. Another distinctive characteristic of gravel-bed streams is a coarse surface layer that may form in both the low-flow channel and on bars. Frequently, the low-flow channels of coarse bed streams are armored with large cobbles and boulders, while bars consist primarily of sand and gravel.

Since the spatial variability in most coarse bed streams is high, securing representative samples is difficult. River bars are frequently chosen as sampling sites because they are considered the most representative of the sediment moving in the stream, and they are usually dry during sampling. Specific bar types have been determined to be more representative than others. A bar type hierarchy established to aid site selection (Bray 1972; Yuzyk 1986) is shown in figure TS13A–4. Mid-channel and diagonal bars are most ideal sampling sites because they are exposed to the highest velocities, which transport the largest materials. Point bars are not as ideal because velocities are highly variable,
Figure TS13A–2  Bed sampling locations for sand-bed streams

**Meandering stream**

- Meander length
- Gage

**Cross section**

- Panel widths = 1/3 to 1/5 width of channel (W)
- Sampling locations—centroid of each panel

**Straight channel**

- Space = 2 to 10 channel widths

**Cross section**

- Initial point
- Width
- LWE
- RWE
- W/S

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.
Surface sediment sampling

Bulk or volumetric sampling is generally considered to be the standard sampling procedure. It involves the removal of a predetermined volume of material large enough to be independent of the maximum particle size. In general, the minimum depth of a volumetric sample should be at least twice the diameter of the maximum particle size, and the minimum weight should be 200 times the weight of the largest particle of interest (Diplas and Fripp 1992). This can lead to unrealistically large samples for many gravel-bed streams, and extrapolation may be necessary. The sample is then sieved, and the analysis is interpreted as a grain size frequency distribution by weight.

As previously noted, in coarse or gravel-bed streams, the top layers may be stratified by size due to armor 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, armor, 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 difficulty 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...
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:

\[
P(V-W)_i = C p(S)_i D_{i+1}^{-1} \quad \text{(eq. TS13A–1)}
\]

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

\[
C = \sum p(S)_i D_{i+1}^{-1} \quad \text{(eq. TS13A–2)}
\]

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
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, kaolinite, and montmorillonite. The dispersed particle fall velocity, flocculated fall velocity of the suspension, clay and nonclay mineralogy, organic content, and cation exchange capacity characterize cohesive sediment. The fluid is characterized by the concentration of important cations, anions, salt, pH, and temperature. More detailed information is presented in Tidal Hydraulics, EM 1110–2–1607 (USACE 1991c).

**Bank material**

Many channel stability issues result from a combination and interaction of a number of different causes. These causes can include not only fluvial erosion forces but also seepage problems, as well as properties of the soil. In addition, the bank material can help define the stability of the channel section and may be responsible for a significant percentage of the total sediment load. Therefore, it is often important to determine the characteristics of the stream bank. This is often done coincident with the bed-material sampling. More information on issues related to the assessment and analysis of bank material is provided in NEH654.09.

**Conclusion**

Bed-material sampling is frequently conducted to make sediment transport calculations. For this purpose, the sampling program should identify not only a representative bed-material gradation, but also any lateral, longitudinal, vertical, and/or temporal variation in bed-material composition. Water depth, velocity, and bed-material size are the most important factors used to identify appropriate samplers and sampling procedures. In sand-bed streams, the sample is typically taken from the upper 5 centimeters of the bed surface. In gravel-bed streams with coarse surface layers, samples of both the surface and subsurface layers are required. Surface sampling of large particles can be done by hand using a pebble count method. However, a pebble count can be biased if there is a significant size fraction that is below 8 millimeters in size. For smaller particles, an adhesive surface sampling approach is often considered necessary.
Technical Supplement 13B

Sediment Budget Example

(210–VI–NEH, August 2007)
Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.
# Technical Supplement 13B

## Sediment Budget Example

### Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Introduction</td>
<td>TS13B–1</td>
</tr>
<tr>
<td>Field reconnaissance</td>
<td>TS13B–2</td>
</tr>
<tr>
<td>Hydrology</td>
<td>TS13B–2</td>
</tr>
<tr>
<td>Average hydraulic parameters</td>
<td>TS13B–4</td>
</tr>
<tr>
<td>Sediment transport rating curve</td>
<td>TS13B–4</td>
</tr>
<tr>
<td>Diversion channel design</td>
<td>TS13B–6</td>
</tr>
<tr>
<td>Sediment budget</td>
<td>TS13B–6</td>
</tr>
<tr>
<td>Further analysis</td>
<td>TS13B–7</td>
</tr>
</tbody>
</table>

### Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS13B–1</td>
<td>Calculated bed-material sediment yield, Dark Canyon Draw</td>
<td>TS13B–6</td>
</tr>
<tr>
<td>TS13B–2</td>
<td>Calculated bed-material sediment yield, diversion channel</td>
<td>TS13B–7</td>
</tr>
</tbody>
</table>

### Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS13B–1</td>
<td>Carlsbad and surrounding areas</td>
<td>TS13B–1</td>
</tr>
<tr>
<td>TS13B–2</td>
<td>Mixed-gravel bedform, Dark Canyon Draw</td>
<td>TS13B–2</td>
</tr>
<tr>
<td>TS13B–3</td>
<td>Bed-material gradations, Dark Canyon Draw</td>
<td>TS13B–3</td>
</tr>
<tr>
<td>TS13B–5</td>
<td>Bed-material sediment transport rating curves, Dark Canyon Draw</td>
<td>TS13B–5</td>
</tr>
<tr>
<td>TS13B–6</td>
<td>Cross section, Dark Canyon Draw diversion channel</td>
<td>TS13B–6</td>
</tr>
</tbody>
</table>
Introduction

A sediment budget analysis was conducted by the U.S. Army Corps of Engineers (USACE) as part of the reconnaissance level planning study for a flood-damage reduction project for the City of Carlsbad, New Mexico (Copeland 1995). This example describes the sediment budget analysis used to identify the magnitude of possible sediment problems that might be associated with one of the proposed project designs. One potential source of flooding was Dark Canyon Draw, a tributary of the Pecos River (fig. TS13B–1). One of the flood damage reduction alternatives being considered was a bypass channel that would divert Dark Canyon Draw around the city of Carlsbad. The proposed diversion would begin near the city airport and flow northeasterly to the Pecos River to a location about 5 miles downstream from the city.

The sediment budget analysis was conducted to determine the magnitude of possible sediment degradation or aggradation problems that might occur with a proposed design for the diversion channel. Depending on the diversion channel design, several sedimentation and channel stability problems could occur. If a threshold channel is constructed that is designed with little or no sediment transport potential, then bed material delivered from upstream would deposit at the diversion entrance. Sediment deposits would have to be removed periodically. If a channel is designed to carry the incoming sediment load, the channel would undergo a period of adjustment as the bed and banks become established. Bed armoring could progress quickly or slowly, with extensive degradation, depending on the consistency of the material through which the diversion channel is cut and the sequence of annual runoff that occurs. Finally, if the diversion channel is too efficient in terms of sediment transport capacity, it could degrade and induce additional channel degradation upstream from the diversion location.

Figure TS13B–1 Carlsbad and surrounding areas
Field reconnaissance

Preliminary assessments of channel stability and potential sediment impacts were determined during the site assessment and investigation phase of the study conducted prior to the project design phase. Data collected during this phase of the study were used in the sediment budget analysis, which was conducted after channel design.

Dark Canyon Draw transitions from a wide, shallow alluvial channel, characteristic of southwestern United States alluvial fans, at its canyon mouth to an incised arroyo at its confluence with the Pecos River. Gravel mining is currently active in the lower reaches of Dark Canyon Draw between the Pecos River and the city airport and has been occurring for many years. The channel had been both widened and deepened due to the gravel mining. The channel also showed signs of incision/degradation upstream from the airport. The bed and banks of the incised channel were capable of supplying significant quantities of sediment to the stream. The bed surface of Dark Canyon Draw consisted primarily of coarse gravel and cobbles. Banks were generally composed of loose alluvial material ranging in size from clays and silts to boulders. The channel tended to migrate laterally, eroding banks, and creating remnant gravel bars in former channels. Armoring was generally observed in the existing low-flow channel. However, the channel would migrate at high flows, mobilizing significant amounts of sediment from the gravel bars and from eroded bank materials.

Bed-material samples were collected during the field reconnaissance. Sample size class distributions were determined using the Wolman (1954) pebble count method and the volumetric bulk method. Due to the limited scope of the sediment impact assessment, samples were collected at only two sites. Both surface and subsurface samples were collected at the mouth of the canyon several miles upstream from the proposed diversion channel. There was no coarse surface layer at the second site, located on a gravel bar about 1 mile downstream from the canyon mouth. The thoroughly mixed bedform was an indication that active-layer mixing had occurred during the last flow event at this site (fig. TS13B–2). Median grain size ranged between 22 and 55 millimeters for all the samples. The gradation determined at the downstream site was selected as the representative gradation for the sediment budget analysis because it was characteristic of a fully mobile bed. Bed-material gradations determined from these samples are shown in figure TS13B–3.

Hydrology

Hydrographs used in the sediment budget analysis were developed using the HEC–1 hydrograph package (USACE 1998b). These were used to calculate sediment yield for flood events. The peak discharge for the 1 percent exceedance flood was 2,000 cubic meters per second (75,000 ft³/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 ft³/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–3  Bed-material gradations, Dark Canyon Draw

The figure shows the bed-material gradations for Dark Canyon Draw near dam site and Dark Canyon Draw 1 mile downstream from canyon mouth. The gradations are represented by different lines and markers indicating various grain sizes and percent finer by weight. The U.S. standard sieve opening (in) and U.S. standard sieve numbers are also depicted for reference. The Hydrometer values are shown on the right side of the graph.
shapes similar to the 1 percent chance exceedance hydrograph. The flow-duration curve is shown in figure TS13B–4.

**Average hydraulic parameters**

A typical reach in the existing Dark Canyon Draw channel was selected from a HEC–2 backwater model (USACE 1990b). The typical reach chosen for this analysis was about 2 miles long and located adjacent to the Carlsbad Airport. The reach was considered to be in a state of nonequilibrium due to its proximity to gravel mining operations. A reach further upstream, less influenced by gravel mining operations, would have been preferred for determining long-term sediment yield. However, the existing backwater model did not extend any further upstream. It was recommended that additional cross-sectional surveys be obtained upstream for more detailed sediment studies.

Water-surface elevations and hydraulic variables were calculated using the HEC–2 model for a range of discharges. Average values for hydraulic variables were then determined using the reach-length weighted averaging procedure in SAM (Thomas, Copeland, and McComas 2003).

**Sediment transport rating curve**

The bed-material sediment yield from Dark Canyon Draw is important when considering sediment transport and channel stability questions. The bed-material sediment load consists of the sediment sizes that exchange with the streambed, as they are transported downstream. The bed-material yield is most likely to be relatively small compared to the total sediment yield because the bed of Dark Canyon Draw consists primarily of gravels and cobbles. The wash load component of the total sediment yield will be transported through the system to the Pecos River unless it is trapped by a reservoir or introduced into a ponded area.

Sediment transport was calculated using several sediment transport equations available in the SAM program. The equations chosen included at least some data from gravel-bed rivers in their development. As can be seen from the sediment discharge rating curves (fig. TS13B–5), predicted sediment transport rates cover a wide range. No data are available on Dark Canyon Draw to aid in the selection of a transport equation. However, the guidance program in SAM identified the North Saskatchewan and Elbow Rivers in Saskatchewan, Canada, as having similar median bed grain sizes, depths, velocities, and slopes as Dark Canyon Draw at high flow. The guidance program from the available set of equations in SAM determined that the Schoklitsch equation (Shulits 1935) best reproduced measured data on the North Saskatchewan and Elbow Rivers. Calculated sediment transport rating curves were compared using different sediment transport functions, as shown in figure TS13B–5. The conclusion is that the Schoklitsch equation will produce a relatively low sediment yield. To cover the uncertainty range in the calculated bed-material sediment yield, two additional sediment transport equations were chosen to calculate yield. The Parker equation (Parker 1990) was used to represent a high sediment transport load, and the Einstein (1950) equation was chosen to represent an intermediate sediment transport load.
Figure TS13B–5  Bed-material sediment transport rating curves, Dark Canyon Draw

![Bed-material sediment transport rating curves](image)

- **Brownlie (1981)**
- **Enstein (bed load)**
- **Meyer-Peter and Muller (1948)**
- **Parker**
- **Schoklitsch**
- **Yang (1979, 1984)**
Diversion channel design

The following criteria were chosen for the diversion channel design:

- a composite channel geometry with a low-flow channel designed to carry the effective discharge
- the overbank flow designed using threshold criteria for the 1 percent chance exceedance flood

Assigned side slopes were 1V:3H, with Manning’s roughness coefficient of 0.05 for the side slope. The project cross section for the diversion channel to be evaluated with the sediment budget analysis is shown in figure TS13B–6.

Sediment budget

The magnitude of potential aggradation or deposition problems in the Dark Canyon channel can be determined by calculating bed-material sediment yield through a typical reach of the existing channel and comparing it to calculated sediment yield in the project reach.

Bed-material sediment yield was calculated for the existing channel using the flow-duration sediment transport curve method and SAM. Sediment yields were calculated for the 1 percent and 10 percent chance exceedance floods using synthetic hydrographs, and for average annual conditions, using the flow-duration curve. Bed-material sediment yields were calculated using three different sediment transport equations. Results are shown in table TS13B–1.

Sediment yield was determined in the diversion channel using the same procedure that was used to calculate sediment yield in the typical reach of the existing channel. Sediment trapping efficiency was then determined for flood hydrographs and for average annual conditions.

### Table TS13B–1

<table>
<thead>
<tr>
<th>Bed-material transport function</th>
<th>1 percent exceedance flood</th>
<th>10 percent exceedance flood</th>
<th>Average annual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m³</td>
<td>yd³</td>
<td>m³</td>
</tr>
<tr>
<td>Schoklitsch</td>
<td>2,400</td>
<td>3,100</td>
<td>530</td>
</tr>
<tr>
<td>Einstein</td>
<td>11,300</td>
<td>14,800</td>
<td>3,300</td>
</tr>
<tr>
<td>Parker</td>
<td>27,700</td>
<td>36,200</td>
<td>4,100</td>
</tr>
</tbody>
</table>

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>
<thead>
<tr>
<th>Sediment transport function</th>
<th>1 percent exceedance flood</th>
<th>10 percent exceedance flood</th>
<th>Average annual</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>m³</td>
<td>yd³</td>
<td>% of inflow</td>
</tr>
<tr>
<td>Schoklitsch</td>
<td>1,600</td>
<td>2,050</td>
<td>66</td>
</tr>
<tr>
<td>Einstein</td>
<td>7,500</td>
<td>9,800</td>
<td>66</td>
</tr>
<tr>
<td>Parker</td>
<td>17,100</td>
<td>22,400</td>
<td>62</td>
</tr>
</tbody>
</table>

1/ Sediment yield volume calculated assuming specific weight of deposit of 1,500 kg/m³ (93 lb/ft³)
Soil Properties and Special Geotechnical Problems Related to Stream Stabilization Projects

(210–VI–NEH, August 2007)
Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.
# Technical Supplement 14A

## Soil Properties and Special Geotechnical Problems Related to Stream Stabilization Projects

<table>
<thead>
<tr>
<th>Contents</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose</td>
<td>TS14A–1</td>
</tr>
<tr>
<td>Introduction</td>
<td>TS14A–1</td>
</tr>
<tr>
<td>Soil classification</td>
<td>TS14A–1</td>
</tr>
<tr>
<td>Soil shear strength</td>
<td>TS14A–3</td>
</tr>
<tr>
<td>Relatively permeable soils</td>
<td>TS14A–3</td>
</tr>
<tr>
<td>Soils with relatively low permeability</td>
<td>TS14A–3</td>
</tr>
<tr>
<td>Stiff, fissured clays</td>
<td>TS14A–3</td>
</tr>
<tr>
<td>Stability evaluations of bank slopes</td>
<td>TS14A–5</td>
</tr>
<tr>
<td>Evaluation of streambank characteristics contributing to streambank failure</td>
<td>TS14A–8</td>
</tr>
<tr>
<td>Typical slope instability problems and behavior of common soil types</td>
<td>TS14A–8</td>
</tr>
<tr>
<td>Solutions to typical slope instability problems</td>
<td>TS14A–9</td>
</tr>
<tr>
<td>Information required for slope stability evaluation of slopes</td>
<td>TS14A–14</td>
</tr>
<tr>
<td>Sloughing and piping/sapping of streambanks</td>
<td>TS14A–14</td>
</tr>
<tr>
<td>Recognizing the problem</td>
<td>TS14A–19</td>
</tr>
<tr>
<td>Traditional solutions</td>
<td>TS14A–19</td>
</tr>
<tr>
<td>Soil bioengineering solutions to piping/sapping problems</td>
<td>TS14A–20</td>
</tr>
<tr>
<td>Shallow slope failures in blocky structured (fissured) highly plastic clays</td>
<td>TS14A–20</td>
</tr>
<tr>
<td>Predicting stable slopes in plastic clays</td>
<td>TS14A–22</td>
</tr>
<tr>
<td>Analyses where the effective cohesion is significant</td>
<td>TS14A–23</td>
</tr>
<tr>
<td>Recognizing the problem</td>
<td>TS14A–23</td>
</tr>
<tr>
<td>Soil bioengineering solutions to shallow slope failures in blocky clays</td>
<td>TS14A–24</td>
</tr>
<tr>
<td>Traditional solutions</td>
<td>TS14A–24</td>
</tr>
<tr>
<td>Dispersive clays</td>
<td>TS14A–25</td>
</tr>
<tr>
<td>Recognizing the problem</td>
<td>TS14A–25</td>
</tr>
<tr>
<td>Crumb test reactions</td>
<td>TS14A–26</td>
</tr>
<tr>
<td>Soil bioengineering solutions to dispersive clay problems</td>
<td>TS14A–26</td>
</tr>
<tr>
<td>Traditional solutions</td>
<td>TS14A–26</td>
</tr>
</tbody>
</table>
Soil Properties and Special Geotechnical Problems Related to Stream Stabilization Projects

Low plasticity sands and silts
Analysis of slope stability in low plasticity sands and silts .................. TS14A–27
Equations for analyzing infinite slope stability ................................ TS14A–30
Seepage parallel to slope .................................................................. TS14A–30
Horizontal seepage .......................................................................... TS14A–30

Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS14A–1</td>
<td>Description of coarse-grain soil relative density</td>
<td>TS14A–2</td>
</tr>
<tr>
<td>TS14A–2</td>
<td>Description of fine-grain soil consistency</td>
<td></td>
</tr>
<tr>
<td>TS14A–3</td>
<td>Estimated soil properties</td>
<td></td>
</tr>
<tr>
<td>TS14A–4</td>
<td>Estimated values for dry unity weight of sands</td>
<td>TS14A–27</td>
</tr>
<tr>
<td>TS14A–5</td>
<td>Estimated values of relative density for described soils</td>
<td>TS14A–28</td>
</tr>
</tbody>
</table>

Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS14A–1</td>
<td>Hand held torvane device</td>
<td>TS14A–2</td>
</tr>
<tr>
<td>TS14A–2</td>
<td>Empirical correlation between effective phi angle (φ) and plasticity index (PI) from triaxial tests on normally consolidated clays</td>
<td>TS14A–5</td>
</tr>
<tr>
<td>TS14A–3</td>
<td>Factor of safety computed before (a, FS = 1.107) and after (b, FS = 0.99) erosion of toe of slope</td>
<td>TS14A–6</td>
</tr>
<tr>
<td>TS14A–4</td>
<td>Failed streambank in Tarboro, NC, following high channel flows following Hurricane Floyd</td>
<td>TS14A–7</td>
</tr>
<tr>
<td>TS14A–5</td>
<td>Shallow sloughing failure in zone of saturated sand</td>
<td>TS14A–9</td>
</tr>
<tr>
<td>TS14A–6</td>
<td>Typical slope stability failure in clay soil type</td>
<td>TS14A–9</td>
</tr>
<tr>
<td>TS14A–7</td>
<td>Before and after pictures of project, Tannehill Branch Givens Park in Austin, TX</td>
<td></td>
</tr>
</tbody>
</table>
Figure TS14A–8  Before and after pictures of project, West Bouldin Creek at South 6th Street in Austin, TX

Figure TS14A–9  Before and after pictures of project, Shoal Creek in Austin, TX

Figure TS14A–10 Barb structures installed to protect toe of slope from erosion

Figure TS14A–11 Sheet pile wall at toe of slope in Tarboro, NC

Figure TS14A–12 Gabion wall constructed at toe of slope enables 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

Figure TS14A–13 Reinforced fill at toe of slope in Austin, TX

Figure TS14A–14 West Bouldin Creek at South 6th Street, Austin project during last stages of construction

Figure TS14A–15 Other slope stabilization projects

Figure TS14A–16 Construction of a geotextile reinforced slope with large derrick stone armoring at toe of slope

Figure TS14A–17 Sloughing of saturated low plasticity zone in lower streambanks caused by seepage forces

Figure TS14A–18 Progression of sloughing type of failure mechanism in a streambank

Figure TS14A–19 Slickensides in exposure of clay soil

Figure TS14A–20 Stream slope that failed when the stream was deepened to increase flow capacity

Figure TS14A–21 Progression of events that can result in bank instability

Figure TS14A–22 Infinite slope equation for slope with small cohesion value

Figure TS14A–23 Blocky structured clay in streambank exposure

Figure TS14A–24 Photographs of exposure of dispersive clays in side slopes of streams in TX and OK

Figure TS14A–25 Typical reactions in crumb test, a test for dispersive soils
<table>
<thead>
<tr>
<th>Figure TS14A–26</th>
<th>A streambank where highly dispersive clays were treated by applying and mixing in hydrated lime to the slope soils</th>
<th>TS14A–27</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS14A–27</td>
<td>An example of a chart used to estimate effective friction angles for different types of soils</td>
<td>TS14A–29</td>
</tr>
</tbody>
</table>
Purpose

The purpose of this technical supplement is to describe special geotechnical problems related to stream stabilization projects.

Topics addressed in this section include:

- parameters used for classifying soils into engineering behavior groups
- recognizing streambank instability and erosion problems that have geotechnical root causes
- piping/sapping of streambanks
- surficial failures in blocky-structured, highly plastic clays
- severe erosion in dispersive clays
- remedial methods for stabilizing slopes where oversteepening is a result of erosion of the toe of the slope

Introduction

Soil bioengineering measures increase stream roughness and slow the water velocity near the slope face. They also armor and reinforce the surface soils. However, some problems with instability and excessive erosion of streambanks are not readily solved by soil bioengineering techniques alone. Problems involving rotational failures of streambanks, piping (sapping) of bank soils, and shallow slides in highly plastic soils are difficult to solve using only soil bioengineering techniques. Erosion on streambanks in highly dispersive clay soils also cannot be solved with soil bioengineering measures alone. If appropriate remedial solutions are to be designed, engineers and planners must recognize and understand special instability problems that have underlying geotechnical causes.

Analyzing bank slopes for geotechnical stability requires an understanding of a complex system of forces. Evaluating how to protect the soils in the slopes from the erosive forces of flowing water acting against otherwise unprotected streambanks frequently is only part of the task. Even if banks are protected from the erosive forces of the water in the channel, external forces including seepage from the bank and gravity acting on soils in the bank can induce slope failures. The forces involved in bank instability problems include gravity acting on the soils in the slope, the internal resistance of soils in the slope, seepage forces in the soils in the slope, as well as the tractive stresses imposed on the soils by flowing water.

Designing various methods for streambank stabilization, such as retaining walls, reinforced fills, sheet piles, and others, requires specialized engineering experience and knowledge. Analytical methods require parameters that are either estimated from other soil properties or obtained in laboratory testing designed for obtaining them.

Soil classification

The Unified Soil Classification System (USCS) is used to group soils based on similar engineering behavior. The USCS is described in two American Society for Testing and Materials International (ASTM) Standards. ASTM D2487 details classifying soils in the USCS using laboratory data. ASTM D2488 describes methods for estimating the classification of a soil from field tests. Classifying soils by the USCS requires data on the following parameters:

- The percentage by dry weight of the total sample that is of three size categories: fines, sands, and gravels. The USCS only considers the portion of a deposit finer than 3 inches. Larger particles are described, but not included in classification procedures. A more detailed description of the three particle size groups is:

  - Percent fines is the percent of the sample finer than the #200 sieve. These particles are smaller than 0.075 millimeter. Particles finer than the #200 sieve include silt and clay size particles that are usually also evaluated with Atterberg limit tests described later in this section. Percent fines is one of the most important parameters in identifying soil types.

  - Percent sand is the percentage of the sample consisting of sand size particles, which are particles 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 sand is one of the most important parameters in identifying soil types.

  - Percent gravel

(210–VI–NEH, August 2007)  TS14A–1
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.

---

### Table TS14A–1

<table>
<thead>
<tr>
<th>Density description</th>
<th>Evaluation/description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>A ½-in-diameter rod can be pushed easily by hand into soil</td>
</tr>
<tr>
<td>Loose</td>
<td>Soil can be excavated with a spade. A 2-in, square, wooden peg can easily be driven to a depth of 6 in</td>
</tr>
<tr>
<td>Medium dense</td>
<td>Soil is easily penetrated with a ½-in rod driven with a 5-lb hammer</td>
</tr>
<tr>
<td>Dense</td>
<td>Soil requires a pick for excavation. A 2-in, square, wooden peg is hard to drive to a depth of 6 in</td>
</tr>
<tr>
<td>Very dense</td>
<td>Soil is penetrated only a few cm with a ½-in rod driven with a 5-lb hammer</td>
</tr>
</tbody>
</table>

### Table TS14A–2

<table>
<thead>
<tr>
<th>Saturated consistency</th>
<th>Evaluation/description</th>
<th>Estimated undrained shear strength (lb/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
<td>Thumb will penetrate greater than 1 in. Soil is extruded between fingers</td>
<td>&lt;250</td>
</tr>
<tr>
<td>Soft</td>
<td>Thumb will penetrate about 1 in. Soil molded by light finger pressure</td>
<td>250–500</td>
</tr>
<tr>
<td>Medium</td>
<td>Thumb will penetrate about ¼ in. Soil molded by strong finger pressure</td>
<td>500–1,000</td>
</tr>
<tr>
<td>Stiff</td>
<td>Indented with thumb</td>
<td>1,000–2,000</td>
</tr>
<tr>
<td>Very stiff</td>
<td>Indented by thumb nail</td>
<td>2,000–4,000</td>
</tr>
<tr>
<td>Hard</td>
<td>Thumbnail will not indent</td>
<td>&gt;4,000</td>
</tr>
</tbody>
</table>
**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 $1 \times 10^{-4}$ 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 $1 \times 10^{-4}$ 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 ($\phi$) 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

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Moist unit weight (lb/ft³)</th>
<th>Saturated unit weight (lb/ft³)</th>
<th>Undrained shear strength properties</th>
<th>Drained shear strength properties</th>
<th>Angle of wall friction (steel pile), δ</th>
<th>Wall/soil adhesion (lb/ft²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose sand</td>
<td>95–125</td>
<td>120–130</td>
<td>0 28</td>
<td>0 28</td>
<td>0.5φ</td>
<td>0</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>110–130</td>
<td>125–135</td>
<td>0 32</td>
<td>0 32</td>
<td>0.5φ</td>
<td>0</td>
</tr>
<tr>
<td>Dense sand</td>
<td>110–140</td>
<td>130–140</td>
<td>0 38</td>
<td>0 38</td>
<td>0.5φ</td>
<td>0</td>
</tr>
<tr>
<td>Very soft clay</td>
<td>85–100</td>
<td>85–100</td>
<td>0–250 0</td>
<td>0 See note 2</td>
<td>0.5φ</td>
<td>0–250</td>
</tr>
<tr>
<td>Soft clay</td>
<td>100–120</td>
<td>100–120</td>
<td>250–500 0</td>
<td>0 See note 2</td>
<td>0.5φ</td>
<td>250–500</td>
</tr>
<tr>
<td>Medium clay</td>
<td>110–125</td>
<td>110–125</td>
<td>500–1,000 0</td>
<td>0 See note 2</td>
<td>0.5φ</td>
<td>500–750</td>
</tr>
<tr>
<td>Stiff clay</td>
<td>115–130</td>
<td>115–130</td>
<td>1,000–2,000 0</td>
<td>50–100 See note 2</td>
<td>0.5φ</td>
<td>750–950</td>
</tr>
<tr>
<td>Very stiff clay</td>
<td>120–140</td>
<td>120–140</td>
<td>2,000–4,000 0</td>
<td>100 See note 2</td>
<td>0.5φ</td>
<td>950</td>
</tr>
<tr>
<td>Hard clay</td>
<td>&gt;130</td>
<td>&gt;130</td>
<td>&gt;4,000 0</td>
<td>100 See note 2</td>
<td>0.5φ</td>
<td>950</td>
</tr>
</tbody>
</table>

Notes:
1/ See tables TS14A–1 and TS14A–2 for qualitative descriptions of soil types.
2/ See figure TS14A–2.
3/ Wall/soil adhesion is typically 0 for drained (long-term) conditions.
the fully softened phi angle and the residual phi angle of these soils are independent of the original strength of the clay and such factors as water content and liquidity index. The strength of these soil types seems to depend only on the size, shape, and mineralogical composition of the constituent particles and the effective normal stress (Stark and Hisham 1997). Fully softened phi angles are usually assumed to be in the range of 18 to 26 degrees and residual phi angles in the range of 6 to 18 degrees. This special type of soil is described further in the following sections with photographs and problems that are associated with the soil type.

## Stability evaluations of bank slopes

Stream channel banks can fail when conditions change that affect the stability of the slope. Examples of changes in conditions include changes in the potentiometric surface (water table) in the slope; changes in the slope configuration including increased height of the slope due to stream bed degradation, bank erosion, or toe erosion; and load added to the top of the streambank such as adding spoil.

A slope is stable as long as the internal forces in the bank soils resisting failure exceed those causing failure. Computerized analyses are available to enable engineers to evaluate how changed conditions can impact this ratio of forces. The ratio of the resisting force to the causative or driving forces is usually termed the factor of safety of a slope.

\[
FS = \frac{\sum \text{RESISTING Forces}}{\sum \text{DRIVING Forces}}
\]

Analyses compute these forces for assumed or known potential failure surfaces using parameters to represent soils in the slope and ground water conditions. If the factor of safety is greater than 1.0, a failure is not predicted. In existing failed slopes, analyses are conducted to determine what changes can be made to increase the factor of safety to a desirable value.

---

Figure TS14A–2 Empirical correlation between effective phi angle (\(\phi\)) and plasticity index (PI) from triaxial tests on normally consolidated clays

![Empirical correlation between effective phi angle (\(\phi\)) and plasticity index (PI) from triaxial tests on normally consolidated clays](image-url)
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–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.
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.
Soil Properties and Special Geotechnical Problems Related to Stream Stabilization Projects

Figure TS14A–5 shows a shallow slide occurring from a zone of saturated sand sloughing from seepage forces. Figure TS14A–6 shows the results of a deep-seated rotational failure in clayey soils. A later section in the document describes this type of failure in detail.

Sands and gravels in streambank slopes typically fail with a shallow sloughing type failure. These failures occur when the bank soils are subjected to oversteepening by toe erosion, or when subjected to seepage forces. The phenomenon of sapping refers to the sloughing of saturated zones of sand below the water table in the exposed streambanks. Slope failures in soils that have clay fines with significant plasticity typically have a circular appearance and are relatively deep-seated. These types of slides are usually precipitated by downcutting of the streambed. Slides of this type may be extensive and affect property some distance from the stream.

Solutions to typical slope instability problems

Instability in the side slopes of streambanks can be prevented or repaired after it occurs. The following outlines preventative methods and methods used to remediate problems, using the same outline as above for the basic causes.

One approach to prevent problems caused by erosion of the streambank toe is to protect it from attack by flowing water. A wide variety of methods can be used including:

- riprap and other armoring techniques including cellular blocks and similar hard armor methods
- soil bioengineering methods such as crib walls at the toe of the slope
- realignment of the channel to reduce scour
- barbs and other methods for deflecting flows away from the toe of the slope

Figures TS14A–7, TS14A–8, and TS14A–9 show examples of methods used to protect the toe of slopes in a project in the city of Austin, Texas. Figure TS14A–10 shows the use of stream barbs to prevent erosion of the toe of a slope. Protecting the toe of a repaired slope from subsequent erosion is essential to the success of most streambank stabilization projects.

Figure TS14A–5
Shallow sloughing failure in zone of saturated sand

Figure TS14A–6
Typical slope stability failure in clay soil type. Failure is a low-radius, deep rotational failure. (Photo credit City of Fargo, ND)
Figure TS14A–7  Before and after pictures of project, Tannehill Branch Givens Park in Austin, TX. Note method for protecting toe of slope from erosion. (Photo courtesy of City of Austin, TX)

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)
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)

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 angle.
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) ![Image](https://via.placeholder.com/150)

(b) ![Image](https://via.placeholder.com/150)

**Figure TS14A–14**  
West Bouldin Creek at South 6th Street, Austin, TX, project during last stages of construction

(a) ![Image](https://via.placeholder.com/150)

(b) ![Image](https://via.placeholder.com/150)
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
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.

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

(a) Cohesive horizon
    Potentiometric surface
    Cohesionless silt/sand horizon

(b) Original ground line
    Overhanging blocks topple into channel
    Failed slope ≈ 4H:1V
    Degraded channel bottom

(c) Original ground line
    Toppled blocks
    Wet, beached slope
    Soil eroded by channel
    Degraded channel bottom

(d) Potentiometric surface
    Channel bottom

(e)
widening of the stream top width. Soil bioengineering techniques may be effective in protecting the toe of these failed slopes from erosion, preventing subsequent failures.

Piping/sapping problems are likely in a stream system that is degrading more quickly than the ground water level can lower by drainage where the bank soils are susceptible. If the streambank soils were able to drain at the same time the stream bottom degraded, the problem would not occur, because the streambanks would remain in a moist, rather than saturated condition.

Piping/sapping failures may be initiated or accelerated by high rainfall, which recharges the water table in the streambank soils adjacent to the stream. Ground water flow may also be affected by nearby larger bodies of water. Ponding of water at the top of the streambank, especially where levees have been constructed, can also contribute to seepage pressures. Surface flow should be diverted and outletted into the stream away from a streambank area that is susceptible to this problem.

Soil bioengineering methods alone are ineffective in addressing the seepage flow in permeable sand deposits because the flow quantities probably exceed the evapotranspiration ability of plants. This is one situation that will require granular filters in combination with a gravel/rock face to outlet the seepage and prevent piping of the bank and bed soils. Soil bioengineering may be integrated with granular filters to stabilize the upper banks and to reinforce the granular filter layers.

Piping/sapping and sloughing may also occur when a stream has been full of water during a prolonged flood stage, and later the water in the stream lowers rapidly. The water in the stream stored during the higher stage may saturate sands and silts in the streambanks, and the saturated soils may fail by sloughing following the drawdown of the stream water. The likelihood of drawdown slope failures in a cohesionless soil horizon depends on how long the water is stored in the stream to saturate the slopes, how quickly the stream storage is emptied, and the permeability of the soil horizons in question.

**Recognizing the problem**

Recognizing a situation where piping/sapping has occurred or is occurring is easy if it is occurring at the time of the inspection. The shape of the streambanks and the appearance of soils in the banks are clues. The saturated zone at the toe of the streambank will be much flatter than the overlying horizons. Free water is visible as it exits the slope. Figure TS14A–17 illustrates the typical appearance of a streambank where sloughing has occurred. At the point where seepage emerges from the bank, the low plasticity soils that are saturated are on a very flat slope, usually in the range of 3H:1V to 4.5H:1V. The streambank is in a temporarily stable condition. The bank will likely remain stable at this condition, until the toe of the slope is again eroded, ground water levels rise, or the stream bottom degrades.

If a piping/sapping failure is not active when it is inspected, recognizing the problem may be more difficult. Vegetation may have become established at the toe of the slope on the soils that have previously sloughed, obscuring clues of the earlier failure. For more information on diagnosis, see the articles by Hagerty (1991). Geologic and geotechnical investigations that determine whether these soil types occur in the streambanks, the elevations of seasonal high water tables, and other factors are important to recognize these potential problems.

**Traditional solutions**

The most common method for solving piping/sapping problems is to use a layer of graded filter sand or sand/gravel mixture placed on the saturated zone of soil in the streambank. The filter material is designed to be more permeable than the bank soils, but fine enough to filter particles and prevent them from moving through the sand. The filter material is often protected from the erosive forces in the flowing stream with a layer of riprap, manufactured paving blocks, or other suitable methods. Usually, a geotextile separator is used between the filter layer and the overlying cover of riprap or paving blocks. This is described in more detail in NEH654 TS14K.

Some designers use a geotextile placed directly on the bank, covered by riprap or other erosion resistant material, without placing a filter layer. This type of de-
sign is usually less expensive, but may not be suitable for very fine silty soils. In a few situations, interceptor drains are installed away from the streambanks, parallel to the stream, to intercept ground water flow and prevent it from exiting at the slope face. Soil bioengineering methods may be effective in stabilizing a failed site if the conditions that caused the failure will most likely not recur with the same severity at that site.

Soil bioengineering solutions to piping/sapping problems

Soil bioengineering techniques are most useful in protecting lower silt/sand slopes after they have achieved a stable angle of repose. Soil bioengineering may also be effective in protecting upper cohesive soil horizons after the slopes have been shaped to a stable configuration. Soil bioengineering measures alone will not prevent a sapping/piping failure from occurring where conditions change rapidly to cause the failures. Soil bioengineering may be useful in transpiring excess water from streambanks, but vegetation will probably not be able to transpire the quantities of water available from high ground water conditions in permeable soils or from the saturation of banks, which occurs during flood staging of the watercourse.

Shallow slope failures in blocky structured (fissured) highly plastic clays

Highly plastic clays with a fissured or blocky structure can also cause severe stability problems in streambanks. The blocky structure of these soils results from desiccation that occurred after the soils were originally deposited. Repeated drying and wetting cycles cause a structure that is sometimes termed a slickensided structure.

Figure TS14A–19 shows photographs of highly plastic clays with slickensides and blocky structure. Shallow surface slides commonly occur in these soils where streams have been modified or the stream system is not in equilibrium and bed degradation is occurring.
Figure TS14A–20 shows a slope failure on a stream-bank 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.

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–20 Stream slope that failed when the stream was deepened by about 4 feet to increase the flow capacity

Figure TS14A–21 Progression of events that can result in bank instability

(a) Stable 4H:1V slope

(b) Original slope

(c) Failed soil mass
Terzaghi and Peck (1967) describe slope failures in excavated slopes in highly plastic clay soils as follows. Note that this description refers to excavations, but the same principles apply to a slope that is deepened or oversteepened either by human activities or degradation of the streambed.

Almost every stiff clay is weakened by a network of hair cracks or slickensides. If the surface of weakness subdivides the clay into small fragments 1 inch or less in size, a slope may become unstable during construction or shortly thereafter. On the other hand, if the spacing of the joints is greater, failure may not occur until many years after the cut is made...

If the spacing of the joints in a clay is greater than several inches, slopes may remain stable for many years or even decades after the cut is made. The lapse of time between the excavation of the cut and the failure of the slope indicates a gradual loss of the strength of the soil.

Before excavation, the clay is very rigid, and the fissures are completely closed. The reduction of stress during excavation causes an expansion of the clay, and some of the fissures open. Water then enters and softens the clay adjoining these fissures. Unequal swelling produces new fissures until the larger chunks disintegrate, and the mass is transformed into a soft matrix containing hard cores. A slide occurs as soon as the shearing resistance of the weakened clay becomes too small to counteract the forces of gravity.

Most slides of this type occur along toe circles involving a relatively shallow body of soil, because the shearing resistance of the clay increases rapidly with increasing distance below the exposed surface. The water seems to cause only the deterioration of the clay structure; seepage pressures appear to be of no consequence.

After a slide occurs, the material underlying the newly exposed slide begins to soften, and the process continues until another slide occurs. The process does not stop until the slope angle becomes compatible with the softest consistency the clay can acquire... Thus the slopes become gentler...

In summary, shallow slope failures on streambanks composed of plastic clay soils are attributed to a network of fissures and blocky structures that develop due to alternating drying and wetting cycles. The desiccation of the clays may be recent, or it may have occurred long ago when the clays were originally deposited in an alluvial flood plain. Blocky structured clays may also occur very deep in older alluvial profiles and in glaciated areas. Instability is common after heavy rains or from elevated ground water. Water stored in the stream in a storm may be another source. The climatic regime in which these sites exist affects the severity of and time required to develop the fissured structure.

Predicting stable slopes in plastic clays

A reliable analytical method for predicting a stable slope for highly plastic clays is not available. The most reliable method may be empirical examination of stable natural slopes in the same materials near the site. Nearby natural slopes that have not been significantly modified for at least 30 years should be studied. At one site studied by NRCS engineers, the cotangent of natural slopes was from 4H:1V to 7H:1V where the highly plastic clays were found. Failures occurred when the slopes were shaped to 3H:1V during enlargement of the stream cross section.

The strength of highly plastic fissured soils for first-time failures may be modeled using the fully softened condition as recommended by some authors including Stark and Hisham (1997). The fully softened strength of these soils depends primarily on the plasticity of the soils. These problems are most severe for CH soils with plasticity index (PI) values above 40. Soils with PIs greater than 80 are stable only on very flat slopes. Available laboratory testing techniques are inadequate to model the fissured structure in these soils. A conservative design assumes that the effective cohesion value in these blocky structured soils is zero, and the design is based only on the fully softened phi angle. The fully softened phi angle is independent of the original strength of the clay and such factors as water content and liquidity index (fig. TS14A–22). It seems to depend only on the size, shape, and mineralogical composition of the constituent particles and the effective normal stress (Stark and Hisham 1997). Fully softened phi angles are generally assumed to be in the range of 14 to 19 degrees. On this basis, clay soils
with PI values of about 30 to 40 are stable on slopes of about 3H:1V, while clays with high plasticity indices (greater than about 80) may require slopes as flat as 6.5H:1V for long-term stability.

**Analyses where the effective cohesion is significant**

In silty, clayey, coarse-grained soils, a significant cohesion property may be present in effective stress parameters. For this assumption, computerized slope stability analyses can be used to calculate factors of safety by various methods such as the method of slices. These types of computer analyses are not appropriate for zero-cohesion soils because the critical failure surfaces are very shallow on the slope face with factors of safety approaching those obtained using the infinite slope equations.

The equation for slightly cohesive sands with seepage parallel to the slope (eq. TS14A–1) is shown in figure TS14A–22, after an equation presented in Lambe and Whitman (1969):

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

**Recognizing the problem**

This problem is identifiable by visual examination and textural evaluation of the soils in the streambanks. These failures occur usually in highly plastic soils classifying as CH, or fat clays, in the USCS. In the soil survey system of the NRCS, these soils are often classified as vertisols and are identified in the soil survey as having high shrink-swell potential. The most problematic soils have liquid limit values greater than 60 and PI values greater than 40. The severity of the problem is directly proportional to the liquid limit and plasticity index of the soils.

The soils have an observable strongly blocky structure in an exposed face. The blocks of clay may be from one quarter to three-quarter inch in dimension. Figure TS14A–23 shows an exposure in a blocky clay soil. Other photographs show slickensides that are present in many such deposits. Slickensides develop from repeated wetting and drying cycles in clays.
The failures usually occur in a progressive manner. The first evidence of a failure may be a small roll of soil at the toe of the slope. The final failure configuration is circular arc-shaped with a flat radius, and it encompasses at least two-thirds of the slope length. The slides are usually no more than 3 or 4 feet deep, measured normal to the slope. Some slides may be deeper if previous sliding has occurred at the site, with depths normal to the slope of up to 8 feet. Slides are often triggered by a high-intensity rainfall event that closely follows a prolonged droughty period. Flood storage in the stream can also provide water to fill the cracks in the soil, causing failures when the stream empties.

Streambanks seek their stable angle of repose. In a stable system, slopes on natural streams in these soil types reach a slope of from 4H:1V to 7H:1V, depending on the clay content and plasticity of the bank soils. Failures with highly plastic clay soils most often occur in streams modified by man or where streambed degradation has occurred, and the oversteepened slopes fail. These failures may not occur until many years following oversteepening of the streambanks.

Soil bioengineering solutions to shallow slope failures in blocky clays

Soil bioengineering techniques that develop a root system capable of reinforcing the streambanks about 4 feet normal to the outer slope may provide added protection against these types of failures. A disadvantage is that transpiration from vegetation may aggravate the drying and shrinkage crack development of the soils. Litter and shade provided by vegetation may deter the drying of the clays from direct sunlight. The large forces that result from the weight of a saturated clay mass are difficult to overcome solely with root mass reinforcement.

Some flattening of the slopes or replacement of the plastic clays with less plastic soils is required, in addition to soil bioengineering techniques, for these worst case situations. Vegetation may be effective in stabilizing the toe area of a failed slope, preventing it from being removed by subsequent erosion from flowing water in the stream. This allows stabilization of the upper slopes to be effective.

Soil bioengineering techniques, such as supplementing cribwalls with vegetation, brush layering, and similar measures, can be useful, but will probably only be long lasting if the more plastic clay soils are replaced behind the walls, and soils are shaped to a flat slope on the streambank above the reinforced structure.

Traditional solutions

As with most problems related to slope instability, stabilizing the toe of the slope and ensuring that the stream grade will not degrade further are essential to the long-term success of any treatment of the problem. The most common methods for treating stream slopes in highly plastic clays that have failed may be summarized as follows:

- Flattening the failed slope to a predicted stable slope is perhaps the most positive solution. This option requires considerable right-of-way if the slopes must be flattened significantly. For example, consider a 15-foot-deep stream that is on a slope of 3H:1V. If the slope needs to be flattened to about 5.5H:1V to be stable, the top width is increased by 37.5 feet.
- Installing gravel trenches in the slope has been an effective treatment. The USACE used this type of repair successfully on a slope failure on the Sunflower River in Mississippi. Sills and Fleming (1992) describe a similar repair. This method is relatively economical, but has not been used widely, and designers may not be as confident in the results.
- Another method of repairing these slides is covering the highly plastic soils in the slide area with soils having more favorable properties such as silty sands, gravels, or riprap.
- Highly plastic clays may be modified by incorporating either hydrated or quick lime. About 5 percent by dry weight is added to the soils to treat them. This alternative is expensive because the lime is costly, and the construction procedures to apply and mix the lime into the streambanks are expensive.
Dispersive clays

Dispersive clays are different in their chemical composition than ordinary, erosion-resistant clays. Dispersive clay soils are far more erodible than ordinary clays because the interparticle attraction is much reduced by imbalanced electro-chemical bonds. Streamside slopes in dispersive clay deposits often develop a highly rilled appearance, also showing a phenomenon referred to as jugging. Dispersive clays are described in detail in Soil Mechanics Note 13, Dispersive Clays.

Recognizing the problem

Dispersive clay slopes often are severely rilled. These rills often develop in a short time. Jugholes are an ideal diagnostic tool for dispersive clays. These features develop when a drying crack in the exposed soil provides an entrance for precipitation that can then erode the sides of the wall. This internal erosion of the crack results in subterranean cavities often termed jugholes. Figure TS14A–24 illustrates the special appearance of dispersive clays.

Figure TS14A–24 Photographs of exposure of dispersive clays in streamside slopes in TX and OK

(a)  
(b)  
(c)
Another diagnostic tool is that vegetation has little effect on the severity of the erosion of dispersive clays. Vegetation is less effective in reducing erosion on dispersive clays because the particles that are eroding are colloidal in size. They are much too fine to filter with vegetation, and they can go into suspension in essentially standing water.

An excellent field test for dispersion is the crumb test. ASTM Standard Test Method D6572 covers methods for performing the test. The test requires a minimum of equipment and is excellent for screening purposes to determine if dispersive clays are likely present.

**Crumb test reactions**

A common field test for dispersive clays is the crumb test, a test for dispersive soils (fig TS14A–25). A small clump of the soil (about a half-inch cube) is placed in a cocktail glass that has about an inch of distilled water, and observed for at least an hour. A rapid formation of a cloud around the soil indicates that it is dispersive. The observed reaction is typically given a rating from the criteria listed.

1—No reaction, water in glass remains clear. Ignore any slaking of clod; examine only for turbidity.

2—Cloud immediately around clod. A hint of a cloud occurs very near the clod. However, it does not spread significantly away from the clod.

3—A colloidal cloud spreads a considerable distance from the clod. However, it does not spread completely to meet the opposite side of the glass.

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

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

**Soil bioengineering solutions to dispersive clay problems**

Soil bioengineering techniques are not likely to be effective in preventing or curing dispersive clay erosion problems on stream slopes. Severe erosion has been observed on the best vegetated sites.

**Traditional solutions**

Several repair or preventive design measures have successfully been applied to dispersive clay sites. In the first method, the dispersive clays are covered with an insulating blanket to prevent the cracking of the clays that leads to more severe erosion. Blankets of sand/gravel/silts have been used with success. For silty sand or gravel blankets, the fines in the sand or gravel should be checked to assure they are not dispersive.
Another method for treating dispersive clays is to chemically alter the soils. Dispersive clays are predominated by sodium, and replacing the sodium ions with calcium, magnesium, or aluminum ions improves the interparticle attraction of the clays. Chemicals that have been successfully used include hydrated lime (fig. TS14A–26), quick lime, fly ash, alum, and gypsum. Refer to Soil Mechanics Note 13 for more detailed information.

**Low plasticity sands and silts**

The problems of bank sloughing in low plasticity sands and silts with a high ground water table were described previously in this chapter. This section provides additional description of the problem and analytical tools.

**Analysis of slope stability in low plasticity sands and silts**

A slope, such as the banks of a stream composed of sandy or silty soils with a low clay content, may slough if it is too steep. The slope that is stable for a given soil is studied with several equations termed infinite slope equations. The equations will be addressed in detail in the following sections. The stability of a stream slope in susceptible soil types is studied by computing a factor of safety. Designers always prefer that the factor of safety for this type of analysis be greater than 1.1.

Three equations are used to compute the factor of safety of the slope for three different assumed ground water flow conditions. The factor of safety of a slope depends on the following three factors: unit weights of soil and water, effective friction angle (phi angle), and the direction of seepage forces.

**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:

\[ \gamma_{moist} = \gamma_{dry} \left( 1 + \frac{w(\%)}{100} \right) \]  

(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.

**Figure TS14A–26**

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

**Table TS14A–4**

Estimated values for dry unit weight of sands

<table>
<thead>
<tr>
<th>Relative density of sand</th>
<th>( \gamma_d ) – Dry density (assumed), lb/ft(^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose</td>
<td>90</td>
</tr>
<tr>
<td>Medium</td>
<td>105</td>
</tr>
<tr>
<td>Dense</td>
<td>115</td>
</tr>
</tbody>
</table>
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).

\[
\gamma_{\text{moist}} = \gamma_{\text{dry}} \times \left(1 + \frac{w(\%)}{100}\right)
\]

\[
= 90.0\ \text{lb/ft}^3 \times \left(1 + \frac{12}{100}\right) \quad \text{(eq. TS14A–3)}
\]

\[
= 90\ \text{lb/ft}^3 \times 1.12
\]

\[
= 100.8\ \text{lb/ft}^3
\]

\[
\gamma_{\text{buoyant}} = \frac{G_s - 1}{G_s} \times \gamma_{\text{dry}} \quad \text{(eq. TS14A–4)}
\]

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.

\[
\gamma_{\text{buoyant}} = \frac{G_s - 1}{G_s} \times \gamma_{\text{dry}}
\]

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

<table>
<thead>
<tr>
<th>Relative density description</th>
<th>Numerical value of relative density</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose</td>
<td>&lt;15</td>
</tr>
<tr>
<td>Loose</td>
<td>15–35</td>
</tr>
<tr>
<td>Medium</td>
<td>35–65</td>
</tr>
<tr>
<td>Dense</td>
<td>65–85</td>
</tr>
<tr>
<td>Very dense</td>
<td>&gt;85</td>
</tr>
</tbody>
</table>
Figure TS14A–27  An example of a chart used to estimate effective friction angles for different types of soils

![Chart showing Angle of internal friction vs. density for coarse-grained soils](chart.png)

- **Angle of internal friction** $\phi'$ (degrees)
- **Relative density**
  - 100%
  - 75%
  - 50%
- **Material type**
  - GP
  - SW
  - GW
- **Porosity (n)**
- **Void ratio (e)**
- **Dry unit weight** ($\gamma_d$, lb/ft$^3$)

$\phi'$ Obtained from effective stress failure envelopes
Approximate correlation is for cohesionless material without plastic fines.
Equations for analyzing infinite slope stability

A separate equation is used for each of the three following seepage conditions:

- no seepage
- seepage flowing generally parallel to slope
- seepage generally following horizontal flow paths

The equation for no seepage is used to study the stability of soils above the water table. The condition where seepage generally follows horizontal flow paths is probably the one to use for most alluvial deposits because the layering of the alluvial soil profile creates this type of preferential flow path. Assume parallel flow paths for soils without much layering.

Because the assumption for this analysis is that the soils have zero cohesion, the height of the slope is not a factor. The same factor of safety is calculated for any height of slope. A factor of safety of 1.1 is commonly regarded as acceptable for this condition because the failures are shallow sloughing types of failures and not usually disastrous in nature.

Moist slope equation

If no seepage is exiting the slope face being examined, the factor of safety for that slope is simply stated as:

$$ FS = m \times \tan(\phi') $$  
(eq. TS14A–6)

where:

- $m$ = slope cotangent
- $\phi'$ = internal friction angle of cohesionless slope soil

Consider the example for 3H:1V permeability side slopes and soils with an effective $\phi'$ angle of 26 degrees, the calculated factor of safety is:

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

Seepage parallel to slope

When seepage exits the slope face and the direction of the flow is parallel to the slope face, equations TS14A–7 and TS14A–8 are applicable. This pattern of seepage occurs in very homogeneous soils with little or no horizontal layering. This assumption is typically not used to represent soil that has been compacted in layers or in alluvial soils. The equations are:

$$ FS = \frac{\gamma_b \times \tan(\phi')}{\gamma_{sat} \times \tan(\theta)} $$  
(eq. TS14A–7)

or

$$ FS = m \times \frac{\gamma_b}{\gamma_{sat}} \times \tan(\phi') $$  
(eq. TS14A–8)

where:

- $\theta$ = slope angle
- $m$ = slope cotangent = cot ($\theta$)
- $\gamma_b$ = buoyant unit weight
- $\gamma_{sat}$ = saturated unit weight
- $\phi'$ = effective friction angle

Horizontal seepage

Equations TS14A–9 and TS14A–10 assume that the seepage forces acting on the soils in the slope are due to flow along horizontal planes.

$$ FS = \frac{(\gamma_{sat} \times \cos^2(\theta) - \gamma_w) \times \tan(\phi')}{\gamma_{sat} \times \sin(\theta) \times \cos(\theta)} $$  
(eq. TS14A–9)

or

$$ FS = \frac{(\gamma_b \times m^2 - \gamma_w) \times \tan(\phi')}{m \times \gamma_{sat}} $$  
(eq. TS14A–10)

where:

- $\theta$ = slope angle
- $m$ = slope cotangent = cot ($\theta$)
- $\gamma_b$ = buoyant unit weight
- $\gamma_{sat}$ = saturated unit weight
- $\phi'$ = effective friction angle
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.

(210–VI–NEH, August 2007)
## Scour Calculations

### Technical Supplement 14B

<table>
<thead>
<tr>
<th>Contents</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose</td>
<td>TS14B–1</td>
</tr>
<tr>
<td>Introduction</td>
<td>TS14B–1</td>
</tr>
<tr>
<td>Processes</td>
<td>TS14B–2</td>
</tr>
<tr>
<td>Effects of stream types</td>
<td>TS14B–4</td>
</tr>
<tr>
<td>Scour computations for design</td>
<td>TS14B–5</td>
</tr>
<tr>
<td>Long-term bed elevation change</td>
<td>TS14B–6</td>
</tr>
<tr>
<td>Armoring</td>
<td>TS14B–6</td>
</tr>
<tr>
<td>Equilibrium slope</td>
<td>TS14B–7</td>
</tr>
<tr>
<td>Cohesive beds</td>
<td>TS14B–9</td>
</tr>
<tr>
<td>Sand and fine gravel—no bed-material sediment supplied from upstream</td>
<td>TS14B–10</td>
</tr>
<tr>
<td>Sand and fine gravel—reduced sediment supply from upstream</td>
<td>TS14B–10</td>
</tr>
<tr>
<td>Beds coarser than sand—no sediment supplied from upstream</td>
<td>TS14B–11</td>
</tr>
<tr>
<td>Sediment continuity analysis</td>
<td>TS14B–12</td>
</tr>
<tr>
<td>More complex approaches for long-term degradation</td>
<td>TS14B–13</td>
</tr>
<tr>
<td>General scour</td>
<td>TS14B–13</td>
</tr>
<tr>
<td>Process description</td>
<td>TS14B–13</td>
</tr>
<tr>
<td>Contraction scour</td>
<td>TS14B–14</td>
</tr>
<tr>
<td>Live-bed contraction scour</td>
<td>TS14B–15</td>
</tr>
<tr>
<td>Clear-water contraction scour</td>
<td>TS14B–17</td>
</tr>
<tr>
<td>Bridge scour</td>
<td>TS14B–17</td>
</tr>
<tr>
<td>Bend scour</td>
<td>TS14B–17</td>
</tr>
<tr>
<td>Bedform scour</td>
<td>TS14B–19</td>
</tr>
<tr>
<td>Process description</td>
<td>TS14B–19</td>
</tr>
<tr>
<td>Bedform predictors</td>
<td>TS14B–19</td>
</tr>
<tr>
<td>Scour associated with structures</td>
<td>TS14B–21</td>
</tr>
<tr>
<td>Structures that span the full width of the channel</td>
<td>TS14B–21</td>
</tr>
<tr>
<td>Sills</td>
<td>TS14B–22</td>
</tr>
<tr>
<td>Step-pool structures</td>
<td>TS14B–23</td>
</tr>
<tr>
<td>Grade control structures and weirs</td>
<td>TS14B–23</td>
</tr>
<tr>
<td>Structures that partially span the channel</td>
<td>TS14B–26</td>
</tr>
<tr>
<td>Example computations</td>
<td>TS14B–29</td>
</tr>
<tr>
<td>Sand-bed reach</td>
<td>TS14B–29</td>
</tr>
<tr>
<td>Gravel-bed reach</td>
<td>TS14B–32</td>
</tr>
<tr>
<td>Design features and measures to address scour</td>
<td>TS14B–35</td>
</tr>
<tr>
<td>List of symbols</td>
<td>TS14B–37</td>
</tr>
</tbody>
</table>

(210–VI–NEH, August 2007)
### Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table TS14B–1</td>
<td>Examples of bridge failures associated with scour</td>
<td>TS14B–2</td>
</tr>
<tr>
<td>Table TS14B–2</td>
<td>Streambed erosion and deposition processes</td>
<td>TS14B–3</td>
</tr>
<tr>
<td>Table TS14B–3</td>
<td>Types of scour analyses</td>
<td>TS14B–5</td>
</tr>
<tr>
<td>Table TS14B–4</td>
<td>Constants for computation of minimum armor particle size</td>
<td>TS14B–7</td>
</tr>
<tr>
<td>Table TS14B–5</td>
<td>Approaches for determining equilibrium slope</td>
<td>TS14B–8</td>
</tr>
<tr>
<td>Table TS14B–6</td>
<td>Ranges for data set underlying the Yang sediment transport relation</td>
<td>TS14B–11</td>
</tr>
<tr>
<td>Table TS14B–7</td>
<td>Constants for equilibrium slope formulas for coarse-bed channels with little or no sediment load input</td>
<td>TS14B–12</td>
</tr>
<tr>
<td>Table TS14B–8</td>
<td>Constants for Lacey and Blench relations, U.S. units (D_{50} in mm)</td>
<td>TS14B–14</td>
</tr>
<tr>
<td>Table TS14B–9</td>
<td>Constant K for Lacey and Blench relations, SI units (D_{50} in mm)</td>
<td>TS14B–14</td>
</tr>
<tr>
<td>Table TS14B–10</td>
<td>Exponent a for contraction scour relation</td>
<td>TS14B–15</td>
</tr>
<tr>
<td>Table TS14B–11</td>
<td>Summary of scour analyses and applicability to various bed types</td>
<td>TS14B–28</td>
</tr>
</tbody>
</table>

### Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS14B–1</td>
<td>Scour observations from typical reaches of alluvial rivers</td>
<td>TS14B–1</td>
</tr>
<tr>
<td>Figure TS14B–2</td>
<td>Examples of local scour</td>
<td>TS14B–3</td>
</tr>
<tr>
<td>Figure TS14B–3</td>
<td>Examples of general scour</td>
<td>TS14B–3</td>
</tr>
<tr>
<td>Figure TS14B–4</td>
<td>Conceptual representation of the relationship between long-term average vertical stability and sediment transport</td>
<td>TS14B–6</td>
</tr>
<tr>
<td>Figure TS14B–5</td>
<td>Definition of terms for armor limited scour</td>
<td>TS14B–6</td>
</tr>
<tr>
<td>Figure TS14B–6</td>
<td>Definition of equilibrium slope, S_{eq}</td>
<td>TS14B–7</td>
</tr>
<tr>
<td>Figure TS14B–7</td>
<td>Headcut migrating upstream through cohesive streambed toward bridge in north central MS</td>
<td>TS14B–9</td>
</tr>
<tr>
<td>Figure TS14B–8</td>
<td>Empirical equilibrium slope—drainage area relationship for Yalobusha River watershed in northern MS</td>
<td>TS14B–9</td>
</tr>
<tr>
<td>Figure TS14B–9</td>
<td>Critical shear stress for channels with boundaries of noncohesive material</td>
<td>TS14B–10</td>
</tr>
<tr>
<td>Figure TS14B–10</td>
<td>Fall velocity for sand-sized particles with a specific gravity of 2.65</td>
<td>TS14B–16</td>
</tr>
<tr>
<td>Figure TS14B–11</td>
<td>Downstream face of Horse Island Chute bridge near Chester, IL, as viewed from left (north) embankment</td>
<td>TS14B–17</td>
</tr>
<tr>
<td>Figure TS14B–12</td>
<td>Definition of recommended length for protection downstream from a bend apex, ( L_p )</td>
<td>TS14B–19</td>
</tr>
<tr>
<td>Figure TS14B–13</td>
<td>Recommended length of protection divided by hydraulic radius, ( L_p/R ), as a function of Manning’s roughness coefficient for the bend, ( n ), and hydraulic radius, ( R )</td>
<td>TS14B–19</td>
</tr>
<tr>
<td>Figure TS14B–14</td>
<td>Relative relationships between progression of alluvial bedforms and flow intensity</td>
<td>TS14B–20</td>
</tr>
<tr>
<td>Figure TS14B–15</td>
<td>Scour associated with low stone weir</td>
<td>TS14B–21</td>
</tr>
<tr>
<td>Figure TS14B–16</td>
<td>Definition sketch for computing scour associated with sills</td>
<td>TS14B–22</td>
</tr>
<tr>
<td>Figure TS14B–17</td>
<td>Definition sketch for computing scour associated with step-pool structures</td>
<td>TS14B–23</td>
</tr>
<tr>
<td>Figure TS14B–18</td>
<td>(a) Low- and (b) high-drop grade control structures</td>
<td>TS14B–24</td>
</tr>
<tr>
<td>Figure TS14B–19</td>
<td>Definition sketch for computing scour associated with grade control structures</td>
<td>TS14B–25</td>
</tr>
<tr>
<td>Figure TS14B–20</td>
<td>Scour associated with stone spur dike</td>
<td>TS14B–26</td>
</tr>
<tr>
<td>Figure TS14B–21</td>
<td>Definition sketch showing crest length, ( L_c ), and side slope angle, ( \theta ), for spur dikes</td>
<td>TS14B–26</td>
</tr>
<tr>
<td>Figure TS14B–22</td>
<td>Four methods for designing stone structures to resist failure due to bed scour</td>
<td>TS14B–35</td>
</tr>
</tbody>
</table>
Technical Supplement 14B

Scour Calculations

Purpose

Scour is one of the major causes of failure for stream and river projects. It is important to adequately assess and predict scour for any stream or river design. Designers of treatments such as bars, revetments, or weirs (that are placed on or adjacent to streambeds) must estimate the probable maximum scour during the design life of the structure to ensure that it can adjust for this potential change. This technical supplement provides guidance useful in performing scour depth computations.

Introduction

Streams continually mold and remold their streambeds by eroding and depositing sediments. Scour and fill of alluvial channels not undergoing long-term aggradation or degradation occur as fluctuations about some average condition. Blodgett (1986) presented information regarding bed elevation fluctuations from 21 sites on streams with a range of bed material sizes. Monthly or annual measurements were made at the same location within generally straight reaches, free of features like bedrock, bridge piers, or large boulders that might cause local scour. Mean and maximum scour depths are plotted in figure TS14B–1 (Blodgett 1986) as a function of median bed-material size. In this figure, the scour depth is defined as the depth of scour below a reference plane, which was set at the highest thalweg elevation measured during the period of observation. Clearly, scour depths can be quite significant.

Scour is perhaps the primary cause of failure of riverine hydraulic structures, and failure to adequately assess and predict scour hazard represents a major design flaw. For example, most failures of continuous bank protection projects like revetments are due to toe scour. The most spectacular examples of structural failure due to scour involve bridges, such as those

---

**Figure TS14B–1**  Scour observations from typical reaches of alluvial rivers
summarized in table TS14B–1 (Lagasse and Richardson 2001). Less well known, but also important, scour problems account for high failure rates sometimes reported for stream habitat structures and modified or realigned stream channels (Frissell and Nawa 1992). A survey of U.S. Army Corps of Engineers (USACE) flood control channel projects found that most reported problems were linked to some form of bed or bank instability, including local scour and vertical instability (McCarley, Ingram, and Brown 1990).

An analysis of potential scour is required for all types of streambank protection and stabilization projects. In addition, scour analysis should be a part of the design of any hard structure placed within the channel. Scour and deposition, of course, are processes that affect any movable bed channel design as described in NEH654.09. An analytical approach is needed because many streams tend to scour during high flows and fill during hydrograph recession. Therefore, severe scour can occur during periods when the bed is obscured, only to refill and appear completely different at baseflow.

Although the term scour includes all erosive action of running water in streams, including bed and bank erosion, the emphasis in this technical supplement is on erosion that acts mainly downward or vertically, such as bed erosion at the toe of a revetment or adjacent to a bank barb. Designers of objects placed on or adjacent to streambeds such as bridge piers, revetments, spurs, barbs, deflectors, weirs, sills, or grade control structures must estimate the probable maximum scour during the design life of the structure and ensure that the structure extends below maximum scour depth. This technical supplement provides guidance on scour depth computations.

### Processes

Scour occurs due to several related processes, and estimated maximum scour is typically computed by summing the scour due to each individual process. Terms used to describe bed erosion processes include degradation, local scour, contraction scour, bend scour and others, and these are related as shown in table TS14B–2. Aggradation and degradation refer to an increase or decrease, respectively, in bed elevation over a long reach, through sediment deposition or erosion. Aggradation and degradation are major adjustments of a fluvial system due to watershed changes. In contrast, scour is erosion of the streambed that, except locally, does not influence the longitudinal profile or gradient of the stream. Scour may also be of a temporary, cyclic nature, with significant local erosion occurring during high flows, and refilling during the receding portion of the flow.

All types of scour are loosely categorized as either general or local scour (Brice et al. 1978). Local scour refers to erosion of the streambed that is immediately adjacent to (and apparently caused by) some obstruction to flow (fig. TS14B–2) (Brice et al. 1978)). General scour commonly affects the entire channel cross section, but it may affect one side or reach more than another (fig. TS14B–3) (Brice et al. 1978)). Both types

<table>
<thead>
<tr>
<th>Date</th>
<th>Event</th>
<th>Stream</th>
<th>Conditions at time of event</th>
<th>Consequences</th>
<th>Cause</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 1987</td>
<td>NY State Thruway bridge collapses</td>
<td>Schoharie Creek, NY</td>
<td>Near-record flood</td>
<td>10 deaths</td>
<td>Cumulative effect of local scour over 10 yr</td>
</tr>
<tr>
<td>1989</td>
<td>U.S. 51 bridge collapses</td>
<td>Hatchie River, TN</td>
<td>—</td>
<td>8 deaths</td>
<td>Northward migration of the main river channel</td>
</tr>
<tr>
<td>March 10, 1995</td>
<td>Interstate 5 bridges collapse</td>
<td>Los Gatos Creek, CA</td>
<td>Large flood</td>
<td>7 deaths</td>
<td>Stream channel degradation combined with local scour</td>
</tr>
</tbody>
</table>
### Table TS14B–2  Streambed erosion and deposition processes

<table>
<thead>
<tr>
<th>General process</th>
<th>Specific process</th>
<th>Description and subtypes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggradation or degradation</td>
<td></td>
<td>An increase or decrease in bed elevation over a long reach through sediment deposition or erosion</td>
</tr>
<tr>
<td>Scour</td>
<td>General scour</td>
<td>Longitudinally local erosion that affects the entire channel cross section</td>
</tr>
<tr>
<td></td>
<td>Bedform scour</td>
<td>Formation of troughs between crests of bedforms, usually in sand-bed streams</td>
</tr>
<tr>
<td></td>
<td>Local scour</td>
<td>Erosion of the streambed that is immediately adjacent to (and apparently caused by) some obstruction to flow</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bridge pier and abutment scour</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Scour at structures that span the channel, such as weirs and sills</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Scour at structures that do not fully span the channel</td>
</tr>
</tbody>
</table>

**Figure TS14B–2** Examples of local scour

- Scour at spur dike
- Exposures of footing and pile
- Erosion of abutment fill
- Exposures of footing

**Figure TS14B–3** Examples of general scour

- Scour at pile
- Erosion of abutment fill
- Scour at pile
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 pre-flood 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—Non-cohesive 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
Scour computations for design

The total scour depth needed for design of key-in or toe-down elevations may be computed by summing all the components of vertical bed change:

\[ z_t = FS \left( z_{ad} + z_c + z_b + z_{ad} + z_s \right) \]  

(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>
<thead>
<tr>
<th>Type of scour or process</th>
<th>Symbol</th>
<th>Type of analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Long-term bed elevation change</td>
<td>( z_{ad} )</td>
<td>Armoring, equilibrium slope, or sediment continuity</td>
</tr>
<tr>
<td>Total general scour</td>
<td>( z_{ad} )</td>
<td>Empirical equations or regime equations</td>
</tr>
<tr>
<td>Contraction scour</td>
<td>( z_c )</td>
<td>Live-bed or clear-water contraction scour</td>
</tr>
<tr>
<td>Bend scour</td>
<td>( z_b )</td>
<td>Bend scour formulas, most include all types of scour</td>
</tr>
<tr>
<td>Bridge scour</td>
<td>Not treated herein</td>
<td>Guidance provided by Richardson and Davis (2001)</td>
</tr>
<tr>
<td>Bedform scour</td>
<td>( z_{bf} )</td>
<td>Formulas for dunes or antidunes. Select type of bedform using bedform predictor equation.</td>
</tr>
<tr>
<td>Local scour</td>
<td>( z_s )</td>
<td>Empirical relations for each major type of structure</td>
</tr>
</tbody>
</table>
Long-term bed elevation change

Although any given point in a stream is constantly changing, a stable stream maintains the same average vertical position for its bed when viewed over a long reach (>20 channel widths) over a long period of time (several decades to a few centuries). Such stable streams are rare, however, because disturbances in the form of human activities or natural events (rare flood events, volcanic eruptions, earthquakes, landslides, fires) are common. Human activities are the most common cause of vertical instability, and among these are urbanization, dam construction, channelization, streambed mining, and land use changes. The effect of each of these activities on fluvial systems is described in NEH654.01 and in USACE (1994a). In general, these activities change the supply of sediment or water to a reach (for example, a dam) or increase the sediment transport capacity of a reach (for example, channel straightening, which increases channel slope). Vertical (and often lateral) instability occurs as the channel degrades or aggrades in response to the imbalance between supply and transport (fig. TS14B–4).

For long-term scour estimation, the designer must compute the long-term bed elevation change required to produce an equilibrium slope. If coarse materials are present in the bed and are not mobilized by a design event, the designer should also compute the depth of scour needed to produce an armor layer. The correct scour depth to use in design (eq. TS14B–1) will be the lesser of the two depths. In general, armoring limits degradation in beds with gravels and cobbles, while beds of finer material degrade until they reach an equilibrium slope.

Armoring

Streambeds often feature a layer of coarse particles at the surface that overlies a heterogeneous mixture containing a wide range of sediment sizes. This layer, which is usually only one or two particle diameters thick, is referred to as the armor layer. Formation and destruction of armor layers on streambeds is described in NEH654.07. When a streambed contains at least some sediment that is too large to be transported by the imposed hydraulic conditions, finer particles are selectively removed. The layer of coarser materials left behind forms an armor layer that limits further scour unless and until higher levels of shear stress destroy the armor layer. For example, armor layer formation is often observed downstream from dams. Borah (1989) proposed the following relationship to compute the scour depth below a dam in a channel with a well-mixed bed comprised of particles with the same specific gravity (fig. TS14B–5):

![Figure TS14B–4](image1.png)  
**Figure TS14B–4**  
Conceptual representation of the relationship between long-term average vertical stability and sediment transport

![Figure TS14B–5](image2.png)  
**Figure TS14B–5**  
Definition of terms for armor limited scour
\( z_t = T - D_x \)  \hspace{1cm} (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:
\[
T = \frac{D_x}{(1 - e)P_x} \quad (eq. TS14B–3)
\]

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:
\[
D_x = K \left( \frac{yS_e}{\Delta S_g} \right)^a \left( \frac{U_*}{\nu} \right)^b \quad (eq. TS14B–4)
\]

where:
\( y \) = flow depth, ft (m)
\( S_e \) = energy slope
\( \Delta S_g \) = relative submerged density of bed-material sediments \( \cong 1.65 \)
\( U_* \) = shear velocity \( = (gyS_e)^{0.5} \)

where:
\( g \) = acceleration of gravity, 32.2 ft/s\(^2\) (9.81 m/s\(^2\))
\( \nu \) = 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.

\[
D_{50} = \text{median grain size of sediment mixture in ft (m). The bed porosity } e, D_{50} \text{, and } P_a \text{ are all estimated from analyses of bed-material samples. Bed porosity may also be estimated using a formula (Komura and Simons 1967):}
\]
\[
e = 0.245 + \frac{0.0864}{(0.1D_{50})^{0.21}} \quad (eq. TS14B–5)
\]

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

**Equilibrium slope**

When sediment transport capacity exceeds sediment supply, channel bed degradation occurs until an armor layer forms that limits further degradation or until the channel bed slope is reduced so much that the boundary shear stress is less than a critical level needed to entrain the bed material. This new, lower slope may be called the equilibrium slope, \( S_{eq} \). Slope adjustment in a sediment deficient reach occurs by degradation, proceeding from the upstream end to the downstream, and the downstream extent of degradation is often limited by a base level control. Figure TS14B–6 illustrates the relationship between existing slope, \( S_{ex} \), equilibrium slope and ultimate general scour due to bed degradation, \( Z_{ad} \) (for a reach of length \( L \) with base level control).

For example, the reach downstream from a reservoir without major tributaries may degrade first just below the dam, and a wave of bed degradation will proceed downstream, gradually tapering off as a base level control (a culvert or a downstream reservoir) is reached. Without downstream control, degradation will continue until halted by channel bed armoring, or until the
entire profile reaches equilibrium slope. The amount of ultimate degradation at a given location upstream from the base level control may be estimated by:

\[ z_{ad} = L(S_{eq} - S_{eq}) \]  

(eq. TS14B–6)

where:

\[ L = \text{distance upstream of the base level control} \]

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>
<thead>
<tr>
<th>Bed type</th>
<th>Sediment supply from upstream</th>
<th>Approach for equilibrium slope</th>
<th>Equation(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesive silt or clay</td>
<td>Any</td>
<td>Watershed-specific regression</td>
<td>n/a</td>
</tr>
<tr>
<td>Sand to fine gravel</td>
<td>Drastically reduced or none</td>
<td>Back calculation based on critical shear stress</td>
<td>TS14B–7</td>
</tr>
<tr>
<td>0.1 ≤ D₅₀ ≤ 5.0 mm</td>
<td>Reduced</td>
<td>Back calculation based on sediment supply</td>
<td>TS14B–12 or TS14B–13</td>
</tr>
<tr>
<td>Coarser than sand</td>
<td>Drastically reduced or none</td>
<td>Manning and Shields</td>
<td>TS14B–14</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Meyer-Peter and Müller</td>
<td>TS14B–15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Schoklitsch</td>
<td>TS14B–16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Henderson</td>
<td>TS14B–17</td>
</tr>
<tr>
<td>Sand or gravel</td>
<td>Any</td>
<td>Sediment continuity</td>
<td>TS14B–18</td>
</tr>
</tbody>
</table>
Cohesive beds

Nickpoint (or knickpoint) migration is a dramatic form of vertical instability that occurs in cohesive soils (fig. TS14B–7). A nickpoint (or headcut) is an abrupt change or inflection in the longitudinal profile of a cohesive streambed. In noncohesive materials, analogous features are manifest as short, steep reaches known as nickzones (or knickzones). Both types of features tend to migrate upstream, particularly during high flows. Bed degradation in the immediate vicinity of a migrating nickpoint can be dramatic, as the bed may be lowered or degraded up to several meters in a single flow event.

The sequence of changes that typically occurs in channels when a headcut passes through have been described in a conceptual model known as the channel evolution model (CEM), or incised channel evolution model (ICEM) (Simon 1989) as described in NEH654.01, NEH654.03, and NEH654.13. Due to the complexities of cohesive bed erosion, it is difficult to predict the rate of nickpoint migration, even given the hydrograph. However, the ultimate amount of degradation may be estimated by extending a thalweg profile from a fixed downstream base level upstream at a slope equal to the equilibrium slope, $S_{eq}$, determined as described (fig. TS14B–6).

Some investigators have developed watershed-specific regressions for predicting $S_{eq}$ for watersheds with beds of sand and consolidated cohesive outcrops. These formulas may be used to predict $S_{eq}$ from the contributing drainage area. The regressions are based on bed slope and drainage area for reaches that have undergone enough degradation to attain equilibrium (fig. TS14B–8 (Simon and Thomas 2002)). This approach may be sufficient for estimation purposes, but it ignores the unsteady nature of sediment supply and resultant complex response. The scatter in predicted values is large (fig. TS14B–8). A similar alternative approach involves fitting an exponential function to plots of thalweg elevation at a given cross section versus time to predict future bed elevations (Simon 1992).

![Figure TS14B–7](image_url)  
**Figure TS14B–7** Headcut migrating upstream through cohesive streambed toward bridge in north-central MS. Headcut was triggered by downstream channelization.

![Figure TS14B–8](image_url)  
**Figure TS14B–8** Empirical equilibrium slope–drainage area relationship for Yalobusha River watershed in northern MS.

\[
S = 0.0028 A^{0.33}  
\]

\[
r^2 = 0.63  
\]
Sand and fine gravel—no bed-material sediment supplied from upstream

For channels with bed material coarser than sand, armoring and slope reduction processes may occur simultaneously. Both types of analyses must be performed to determine which will provide the limiting factor. Pemberton and Lara (1984) also suggest that stable slopes may be computed for channels with noncohesive beds with sediment sizes between 0.1 millimeter and 5 millimeters by obtaining a critical shear stress value from the graphical compilation published by Lane (1952) (fig. TS14B–9).

\[
S_\text{eq} = \left( \frac{\tau_c}{\gamma y} \right) \quad \text{(eq. TS14B–7)}
\]

where:
- \(\tau_c\) = critical shear stress from the curve in figure TS14B–9, lb/ft\(^2\) (N/m\(^2\))
- \(y\) = mean flow depth, ft (m)

Sand and fine gravel—reduced sediment supply from upstream

It is not uncommon to have the sediment supply reduced to a stream reach. This occurs when a watershed is reforested, in later stages of urbanization, bed material is mined, diversions are constructed, or when reservoirs are placed in one or more subwatersheds. The concept of equilibrium slope remains valid for these conditions. Use observed bed-material sediment discharge data to fit a regression function of the form:

\[
q_s = a u^b y^c \quad \text{(eq. TS14B–8)}
\]

where:
- \(q_s\) = sediment transport capacity in dimensions of volume per unit width per unit time, ft\(^3\)/s (m\(^3\)/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\(^3\)/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):

\[
a = 0.025n^{(2.39-0.6 \log D_{50})} \left(D_{50} - 0.07\right)^{-0.14} \quad \text{(eq. TS14B–9)}
\]

\[
b = 4.93 - 0.74 \log D_{50} \quad \text{(eq. TS14B–10)}
\]

\[
c = -0.46 + 0.65 \log D_{50} \quad \text{(eq. TS14B–11)}
\]

\(D_{50}\) 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 ($0.1 \text{ mm} < D_{50} < 5.0 \text{ mm}$) are provided by Richardson, Simons, and Lågasse (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:

$$S_{eq} = \left( \frac{a}{q_s} \right)^{10 \left( \frac{n}{K} \right)^2} q^{\frac{2(b+c)}{n}} \left( \frac{n}{K} \right)^2$$

(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:

$$S_{eq} = S_{ex} \left( \frac{Q_s(\text{future})}{Q_s(\text{existing})} \right)^{10 \left( \frac{n}{K} \right)^2}$$

(eq. TS14B–13)

where:

- $S_{ex} =$ existing channel slope
- $Q_s =$ sediment supply, $\text{ft}^3/\text{s}$ ($\text{m}^3/\text{s}$)

### Table TS14B–6

Ranges for data set underlying the Yang sediment transport relation (eqs. TS14B–8 through TS14B–11)

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Range (SI units)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{50}$, mm</td>
<td>0.1–2.0</td>
</tr>
<tr>
<td>$u$, velocity, ft/s (m/s)</td>
<td>2.0–8.0 (0.6–2.4)</td>
</tr>
<tr>
<td>$y$, depth, ft (m)</td>
<td>2.0–25 (0.6–7.6)</td>
</tr>
<tr>
<td>$S$, slope</td>
<td>0.00005–0.002</td>
</tr>
<tr>
<td>Manning $n$</td>
<td>0.015–0.045</td>
</tr>
<tr>
<td>Froude No.</td>
<td>0.07–0.70</td>
</tr>
<tr>
<td>$q$, unit discharge of water, ft$^3$/s ($\text{m}^3$/s)</td>
<td>4.0–200 (0.37–18.6)</td>
</tr>
</tbody>
</table>

The sediment supply for existing conditions may be measured or computed, while the supply for future conditions must be computed using an appropriate sediment transport relation. In both cases, the sediment transport rate must correspond to the channel-forming discharge.

### Beds coarser than sand—no sediment supplied from upstream

When a reservoir with long storage time is placed on a river or stream, bed-material sediment supply to downstream reaches is drastically reduced and can be cut off entirely. A similar reduction occurs in the latter stages of urbanization when construction sites and other disturbed areas are covered with impervious surfaces or vegetation. Four equations are presented for $S_{eq}$, the equilibrium slope, in conditions where sediment transport rates are negligibly small. Variable definitions follow the fourth equation. Equilibrium slope may be selected as an average of that provided by the four relations, or the most applicable relationship may be selected for use based on a study of the original references.

- Simultaneous solution of the Manning and Shields equations (for $D_{50} > 6$ mm):

$$S_{eq} = \left[ \theta D_{50} A S_{g} \right]^{10} \frac{K}{\left( \frac{q^n}{q^n} \right)^2}$$

(eq. TS14B–14)

- Based on Meyer-Peter and Müller sediment transport relationship for material coarser than sand:

$$S_{eq} = K \left( \frac{D_{50}}{q} \right)^{7} q^{\frac{9}{6}}$$

(eq. TS14B–15)

- Based on the Schoklitsch equation for coarse sand or gravel:

$$S_{eq} = K \left( \frac{D_{50}}{q} \right)^{3} q^{\frac{9}{2}}$$

(eq. TS14B–16)

- Based on the Henderson formula for materials larger than 6 mm:

$$S_{eq} = K Q_s^{0.46} D_{50}^{1.5}$$

(eq. TS14B–17)
where:

\[ K = \text{constants given as shown in table TS14B–7} \]

\[ S_{\text{eq}} = \text{equilibrium channel slope at which sediment particles of size } D_c \text{ and larger will no longer move} \]

\[ \Delta S_g = \text{relative submerged density of bed-material sediments} \cong 1.65 \]

\[ q = \text{channel-forming discharge per unit width, } \text{ft}^3/\text{s}/\text{ft} (\text{m}^3/\text{s}/\text{m}) \]

\[ n = \text{Manning's roughness coefficient} \]

\[ D_{90} = \text{sediment size for which } 90\% \text{ by weight of bed material is finer, m (ft)} \]

\[ D_{50} = \text{median sediment size, ft (m) (Note units)} \]

\[ D = \text{mean bed-material particle size, mm} \]

\[ Q_d = \text{design discharge, ft}^3/\text{s} (\text{m}^3/\text{s}) \]

\[ Q_b = \text{discharge over bed of channel, ft}^3/\text{s} (\text{m}^3/\text{s}). \]

\[ \text{Normally } Q_d/Q_b = 1 \text{ for wide channels} \]

\[ y = \text{mean flow depth, ft (m)} \]

\[ n_b = \text{Manning’s roughness coefficient for streambed} \]

### Sediment continuity analysis

In theory, sediment continuity analysis may be used for channels with any type of bed material. In practice, the lack of reliable sediment transport relations for channels with bed material finer than sand or coarser than cobbles makes such analysis difficult. In continuity analysis, the volume of sediment deposited in or eroded from a reach during a given period of time is computed as the difference between the volumes of sediment entering and leaving the reach:

\[ \Delta V = V_{s_{\text{in}}} - V_{s_{\text{out}}} \]  

(eq. TS14B–18)

where:

\[ \Delta V = \text{volume of bed-material sediment stored or eroded, } \text{ft}^3 (\text{m}^3) \]

\[ V_{s_{\text{in}}} = \text{volume of bed-material sediment supplied to the reach, } \text{ft}^3 (\text{m}^3) \]

\[ V_{s_{\text{out}}} = \text{volume of bed-material sediment transported out of the reach, } \text{ft}^3 (\text{m}^3) \]

From equation TS14B–18, the average amount of bed level change may be computed by:

\[ z_{\text{ad}} = \frac{\Delta V}{W_c L_r} \]  

(eq. TS14B–19)

where:

\[ W_c = \text{average channel width, ft (m)} \]

\[ L_r = \text{reach length, ft (m)} \]

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 \( \Delta t \) 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 \( \Delta V \). 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

<table>
<thead>
<tr>
<th>Relationship</th>
<th>U.S. units</th>
<th>SI units</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Manning and Shields</td>
<td>1.486</td>
<td>1.0</td>
<td>Lagasse, Schall, and Richardson (2001)</td>
</tr>
<tr>
<td>Meyer-Peter and Müller</td>
<td>60.1</td>
<td>28.0</td>
<td>Lagasse, Schall, and Richardson (2001)</td>
</tr>
<tr>
<td>Schoklitsch</td>
<td>0.00174</td>
<td>0.000293</td>
<td>Pemberton and Lara (1984)</td>
</tr>
<tr>
<td>Henderson</td>
<td>0.44 ((D_{90}) in ft)</td>
<td>0.33 ((D_{90}) in m)</td>
<td>Henderson (1966)</td>
</tr>
</tbody>
</table>
in discrete time intervals, and $\Delta V$ values computed for the average discharge occurring during each interval. Numerical integration is then used to obtain the total $\Delta V$ for the event:

$$
\Delta V = \sum_{i=1}^{n} \left( V_{s_{i-1}} - V_{s_{i+1}} \right) \Delta t_i \quad (eq. \text{TS14B–20})
$$

This type of analysis may be laborious if several events are simulated, and changes in reach hydraulics due to changes in bed-material gradation may be hard to track. More sophisticated methods are described in the following section.

More complex approaches for long-term aggradation or degradation

Detailed assessment of scour and deposition in a channel reach under natural (unsteady) inputs of water and sediment require numerical (computer) simulation modeling. Since flow records are input as hydrographs, it is not necessary to select a single design flow. Primary types of simulation models include one-dimensional models, which simulate changes in bed elevation with streamwise distance, but ignore variations from one side of the channel to the other, and two-dimensional models, which represent the channel bed as a mosaic of rectangular areas, but do not simulate variations in velocity in the vertical direction. One-dimensional models have limited capability to simulate local scour.

The models route sediment down a channel and adjust the channel geometry (usually bed elevation, but not bank position) to reflect imbalances in sediment supply and transport capacity. The BRI–STARS (Molinas 1990) and HEC–6 (USACE 1993c) models are examples of sediment transport models that can be used for single event or long-term degradation and aggradation estimates. HEC–6 is introduced in NEH654.13, and simulates only changes in bed elevation, while BRI–STARS has an option for predicting width adjustment. The USDA ARS model CONCEPTS includes a full suite of routines for assessing the geotechnical stability of channel banks and erosion of bank material through both hydraulic and geotechnical processes, as well as one-dimensional flow modeling (Langendoen 2000). The information needed to run these models includes (Lagasse, Schall, and Richardson 2001):

- channel and flood plain geometry
- structure geometry
- hydraulic roughness
- geologic or structural vertical controls
- downstream water surface relationship
- event or long-term inflow hydrographs
- tributary inflow hydrographs
- bed-material gradations
- upstream sediment supply
- tributary sediment supply
- selection of appropriate sediment transport relationship
- depth of alluvium

CONCEPTS also requires data describing geotechnical properties of bank soils. None of the models can predict the formation of nickpoints or their migration rates. Modeling movable-bed channels requires specialized training and experience. A description of how models should be used is presented by USACE (1993c).

General scour

Process description

General scour refers to all types of scour that are not local (fig. TS14B–3). General scour commonly, but not necessarily, occurs over the entire cross section, and may involve reaches of varying length depending on the type of scour and site-specific conditions. General scour includes contraction scour and bend scour. Presumably, most of the scour measured at the 21 sites observed by Blodgett (1986) was general scour. He noted that:

\[
 z_i (\text{mean}) = K D_{50}^{-0.11} \quad (eq. \text{TS14B–21})
\]

and

\[
 z_i (\text{max}) = K D_{50}^{-0.11} \quad (eq. \text{TS14B–22})
\]
where:

\[ z_t \text{ (mean)} = \text{best fit curve (fig. TS14B–1) for observed scour depth, ft} \]

\[ z_t \text{ (max)} = \text{enveloping curve (fig. TS14B–1) for maximum scour depth, ft} \]

\[ K = \text{coefficient = 1.42 and 6.5 for } z_t \text{ mean and } z_t \text{ max, respectively (0.84 and 3.8 for SI units), and } D_{50} \text{ is the median size of the bed material, ft (mm)} \]

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:

\[ z_t = KQ_d^a W_f^b D_{50}^c \]  

(eq. TS14B–23)

where:

\[ z_t = \text{maximum scour depth at the cross section or reach in question, ft (m)} \]

\[ K = \text{coefficient (table TS14B–8)} \]

\[ Q_d = \text{design discharge, ft}^3/\text{s (m}^3/\text{s)} \]

\[ W_f = \text{flow width at design discharge, ft (m)} \]

\[ D_{50} = \text{median size of bed material (mm)} \]

\[ a, b, c = \text{exponents (table TS14B–8)} \]

Values for the coefficient and exponents are as shown in table TS14B–8 when U.S. units are used for \( Q_d \) and \( W_f \) and \( D_{50} \) is in millimeters.

Values for the exponents, \( a, b, \) and \( c \) are unchanged when SI units are used, but values for the coefficient \( K \) when SI units are used for \( Q \) and \( W_f \) and \( D_{50} \) is in millimeters (table TS14B–9).

**Contraction scour**

Contraction scour occurs when the flow cross section is reduced by natural features, such as stone outcrops, ice jams, or debris accumulations, or by constructed features such as bridge abutments. Contraction scour is most often observed when bridge approaches force flood plain flow back into the main channel to pass under the bridge. According to the law of continuity, a decrease in flow area requires an increase in the mean velocity component normal to the area, which produces an attendant increase in boundary shear stresses and bed-material transport, assuming the boundary is erodible. As erosion progresses, area increases and velocity decreases, leading to an equilibrium condition in which the rate of bed material transported into the contracted reach is equivalent to the rate of transport out of the contracted reach. Contraction scour is a

**Table TS14B–8**  
Constants for Lacey and Blench relations, U.S. units (\( D_{50} \) in mm)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Lacey</th>
<th>Blench</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( K )</td>
<td>( a )</td>
</tr>
<tr>
<td>Straight reach</td>
<td>0.097</td>
<td>1/3</td>
</tr>
<tr>
<td>Moderate bend</td>
<td>0.195</td>
<td>1/3</td>
</tr>
<tr>
<td>Severe bend</td>
<td>0.292</td>
<td>1/3</td>
</tr>
<tr>
<td>Right angle bend</td>
<td>0.389</td>
<td>1/3</td>
</tr>
<tr>
<td>Vertical rock wall</td>
<td>0.487</td>
<td>1/3</td>
</tr>
</tbody>
</table>

**Table TS14B–9**  
Constant \( K \) for Lacey and Blench relations, SI units (\( D_{50} \) in mm)

<table>
<thead>
<tr>
<th>Condition</th>
<th>Lacey</th>
<th>Blench</th>
</tr>
</thead>
<tbody>
<tr>
<td>Straight reach</td>
<td>0.030</td>
<td>0.162</td>
</tr>
<tr>
<td>Moderate bend</td>
<td>0.059</td>
<td>0.162</td>
</tr>
<tr>
<td>Severe bend</td>
<td>0.089</td>
<td>0.162</td>
</tr>
<tr>
<td>Right angle bend</td>
<td>0.119</td>
<td>0.337</td>
</tr>
<tr>
<td>Vertical rock wall</td>
<td>0.148</td>
<td>0.000</td>
</tr>
</tbody>
</table>
form of general scour because material is removed from all, or almost all, of the wetted perimeter of the contracted section.

**Live-bed contraction scour**

Live-bed conditions may be assumed at a site if the mean velocity upstream exceeds the critical velocity for the beginning of motion, \( V_c \), for the median size of bed material available for transport, \( D_{50} \). When the velocity falls below the critical level, clear-water scour dominates. Both types of scour may occur during a given hydrologic event. The critical velocity may be estimated by:

\[
V_c = Ky^{0.6} D_{50}^{0.3} \quad \text{(eq. TS14B–24)}
\]

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:

\[
z_c = y_2 - y_o \quad \text{(eq. TS14B–25)}
\]

and

\[
\frac{y_2}{y_1} = \left( \frac{Q_2}{Q_1} \right)^{0.6} \left( \frac{W_{b2}}{W_{b1}} \right)^{0.8} \quad \text{(eq. TS14B–26)}
\]

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, ft³/s (m³/s)
- \( Q_2 \) = flow rate in the contracted section, ft³/s (m³/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>
<thead>
<tr>
<th>( U^*/\omega )</th>
<th>Exponent ( a )</th>
<th>Mode of bed-material transport</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.50</td>
<td>0.59</td>
<td>Mostly contact bed-material discharge</td>
</tr>
<tr>
<td>0.50 to 2.0</td>
<td>0.64</td>
<td>Some suspended bed-material discharge</td>
</tr>
<tr>
<td>&gt;2.0</td>
<td>0.69</td>
<td>Mostly suspended bed-material discharge</td>
</tr>
</tbody>
</table>

\[
U_*=\left( \frac{\tau_o}{\rho} \right)^{0.5} = (gy_iS_e)^{0.5}
\]

where:
- \( U_* \) = \((\tau_o/\rho)^{0.5} = (gy_iS_e)^{0.5}\), shear velocity in the upstream section, ft/s (m/s)
- \( g \) = acceleration of gravity, 32.2 ft/s², (9.81 m/s²)
- \( S_e \) = slope of energy grade line of main channel, ft/ft (m/m)
- \( \tau_o \) = average bed shear stress in the upstream section, lb/ft² (N/m²), given by:

\[
\tau_o = \gamma_w RS_e
\]

where:
- \( R \) = hydraulic radius, ft (m)
- \( S_e \) = energy slope
- \( \rho_w \) = density of water, 1.94 slugs/ft³ (1,000 kg/m³)
- \( \omega \) = 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).

\[ \omega = \frac{K_1 \Delta S_g g D_s^2}{v} + K_2 \sqrt{\Delta S_g g D_s} \]  
(eq. TS14B–27)

where:

\[ K_1 = 0.055 \tanh \left[ 12A^{-0.50} \exp \left( -0.0004A \right) \right] \]  
(eq. TS14B–28)

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

\[ \Delta S_g = \text{relative submerged density of bed-material sediments} \approx 1.65 \]

\[ g = \text{acceleration of gravity,} \ 32.2 \text{ ft/s}^2 \ (9.81 \text{ m/s}^2) \]

\[ v = \text{kinematic viscosity of water,} \ \text{ft}^2/\text{s} \ (\text{m}^2/\text{s}) \]

\[ D_s = \text{a characteristic sediment diameter, ft} \ (\text{m}) \]

\[ A = \frac{\Delta S_g g D_s^2}{v^2} \]  
(eq. TS14B–30)

If bottom width is not easily defined, it is permissible to use top width for \( W_{b1} \) and \( W_{b2} \) but it is important to use a consistent definition of width for both quantities. In sand-bed streams, a contraction scour zone is often formed during high flows and refilled during falling stages. In such a case, \( y_o \) may be approximated by \( y_1 \).

Live-bed scour depths are sometimes limited by coarse sediments within the sediment mixture that form an armor layer. When gravel or larger sized material is present, it is recommended that scour depths be calculated using both live-bed and clear-water approaches, and that the smaller of the two scour depths be used.

The procedure is a simplified version of one described in greater detail in Petersen (1986). An alternative approach for gravel-bed contraction scour is presented by Wallerstein (2003) that is based on sediment continuity.

**Figure TS14B–10**  
Fall velocity for sand-sized particles with a specific gravity of 2.65
Clear-water contraction scour

Clear-water scour occurs when there is insignificant transport of bed-material sediment from the upstream into the contracted section. In some cases, a channel constriction creates enough of a backwater condition to induce sediment deposition upstream. Scour in the contracted section normally increases as the flow velocity increases. Live-bed scour becomes clear-water scour in the contracted section. The magnitude of clear-water contraction scour may be computed as follows (Richardson and Davis 2001):

\[ z_c = y_2 - y_o \]  
(eq. TS14B–31)

and

\[ y_2 = \frac{KQ_2^2}{D_s^2 W_{b2}^2} \]  
(eq. TS14B–32)

where:

- \( K = 0.0077 \) for U.S. units and 0.025 for SI units
- \( Q_2 \) = discharge through the contracted section, ft³/s (m³/s)
- \( D_s \) = diameter of the smallest nontransportable particle in the bed material. Assumed = 1.25D₅₀, ft (m), (note units)
- \( W_{b2} \) = bottom width of the contracted section, ft (m)

The assumption that \( D_m = 1.25D_{50} \) implies that some armoring takes place as scour occurs. If the bed material is stratified, the ultimate scour depth may be determined by using the clear-water scour equation sequentially with successive \( D_s \) of the bed-material layers.

Bridge scour

Flow under bridges often produces local scour around bridge piers. Contraction of the floodway by bridge abutments and approaches also causes contraction scour across the cross section. Due to the economic and safety implications of bridge scour, it has received more study than any other type of scour, with extensive analyses of the effects of pier and abutment geometry, flow regime, sediment load, and bedforms. Scour at bridges is intensified when debris becomes trapped against the upstream side of piers (fig. TS14B–11 (Huizinga and Rydlund 2001). When the water surface upstream from a bridge opening is higher than downstream, a special condition known as pressure flow occurs. Pressure flow scour may be two to three times as great as normal contraction scour.

NEH654 TS14Q provides guidance for the analysis and design of small bridge abutments. The reader is also referred to Richardson and Davis (2001) for further design guidance.

Bend scour

Flow through channel meander bends results in water moving in a corkscrew or helical pattern that moves sediment from the outside (concave bank) to the inside of the bend, which is often a point bar. Velocity components not in the streamwise direction are referred to as secondary currents, and the secondary currents that occur in meander bends, though often quite complex, generally have the effect of eroding outer banks. The bank toe is often the locus of maximum shear and erosion, particularly if the bank is armored or otherwise resistant to erosion. Empirical relationships between the maximum depth of scour in a bend and the average depth in a bend have been developed using much of the field data as described in

Figure TS14B–11: Downstream face of Horse Island Chute bridge near Chester, IL, as viewed from left (north) embankment. Note debris trapped on upstream face of bents.
NEH654.09. Briefly, the field data lead to a conservative formula for bend scour, $z_b = y_{\text{mean}} - y_{\text{max}}$

$$z_b = y \left( \frac{y_{\text{max}}}{y} - 1 \right) \quad \text{(eq. TS14B–33)}$$

where:
- $y$ = average flow depth in the bend, ft (m)
- $y_{\text{max}}$ = maximum flow depth in the bend, ft (m)

$y_{\text{max}}/y = 1.5 + 4.5 \left( \frac{W_i}{R_c} \right) \quad \text{(eq. TS14B–34)}$

where:
- $W_i$ = channel width at bend inflection point, ft (m)
- $R_c$ = bend radius of curvature, ft (m)

This equation is an asymptotic relationship with a theoretical minimum $y_{\text{max}}/y_{\text{mean}}$ of 1.5 representing pool scour depths expected in a straight channel with a pool-riffle bed topography. From this upper bound relationship, $y_{\text{max}}/y_{\text{mean}}$ ranges from 4 to 3 for $W_i/R_c$ between 0.33 and 0.56. For channels with $W_i/R_c > 0.56$, $y_{\text{max}}$ is independent of bend curvature, and it is recommended that a value of 4 be used for $y_{\text{max}}/y_{\text{mean}}$. 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):

$$\frac{L_p}{R} = 0.0604 \left( R^3 \right)^{1/2} \frac{1}{n} \quad \text{(eq. TS14B–35)}$$

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):

$$\frac{y_{\text{max}}}{y_c} = \text{FS} \left[ 1.8 - 0.051 \left( \frac{R_c}{W_i} \right) + 0.0084 \left( \frac{W_i}{y_c} \right) \right] \quad \text{(eq. TS14B–36)}$$

where:
- $y_{\text{max}}$ = 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 $(1.5 < R_c/W_i < 10)$ and $(20 < W_i/y_c < 125)$. 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 $y_{\text{max}}/y_c$ were underpredicted by more than 5 percent. When FS = 1.19, only 2 percent of the observed values of $y_{\text{max}}/y_c$ 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 $y_{\text{max}}/y_c$ predicted by these relationships vary by less than 25 percent for FS = 1.19 and $5 < R_c/W_i < 10$. 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.
Bedform scour

Process description

Mobile riverbeds deform to produce ripples, dunes, and antidunes at specific levels of shear stress for a given sediment size (fig. TS14B–14). Most textbooks also recognize large bars (forms having length equal to the channel width or greater) as a type of bedform, but reliable predictors for bars have not been developed. In practice, bedforms other than bars are uncommon in channels dominated by sediments coarser than sand. Dunes and antidunes in sand beds can result in additional scour, since they migrate by a systematic process of erosion and deposition (ripples are too small to be significant), controlled by flow velocities. The passage of a large dune may increase local scour depths as much as 30 percent.

Many attempts have been made to develop relationships to predict the type and dimensions of bedforms based on the bed sediment gradation and the imposed flow. In general, scour analysis involves the use of a bedform predictor that is related to bedform type and amplitude. Half of the amplitude is then assumed to contribute to total scour. Some types of bedforms, however, often occur side-by-side in a cross section or reach of a natural stream. Nonetheless, scour computations normally focus on either dunes or antidunes, which have the greatest amplitude.

Bedform predictors

Water flowing over an erodible bed can produce a variety of configurations. Van Rijn (1984) suggested that dunes would form when:

$$D_s = D_{50} \left( \frac{\Delta S g}{v^2} \right)^{\frac{1}{3}} > 10$$  \hspace{1cm} (eq. TS14B–37)

and

$$3 < T_{ts} < 15$$  \hspace{1cm} (eq. TS14B–38)

where:

$$T_{ts} = \frac{\tau_s}{\tau_c}$$  \hspace{1cm} (eq. TS14B–39)
Dₘ = dimensionless sediment size
D₅₀ = median grain size, ft (m) (note units)
g = acceleration of gravity, 32.2 ft/s² (9.81 m/s²)
ν = kinematic viscosity of water, ft²/s (m²/s)
Tₛₜ = dimensionless transport-stage parameter
τₛ* = bed shear stress due to skin or grain friction, lb/ft² (N/m²), which may be computed by

\[ \tau_s^* = \frac{\rho g u^2}{18 \log \left( \frac{12 R}{3 D_{50}} \right)^2} \]  (eq. TS14B–40)

where:

where:

\[ \theta = \frac{0.24}{D_m} + 0.055 \left[ 1 - \exp \left( -0.02 D_m \right) \right] \]  (eq. TS14B–41)

\[ \tau_c^* = \frac{103.0 \theta D_{50}}{D_{90}} \] for D₅₀ in ft and \( \tau_c^* \) in lb/ft²

\[ \tau_c^* = 16.187 \theta D_{50} \] for D₅₀ in m and \( \tau_c^* \) in N/m²

where:

\[ D_m = D_{50} \left( \frac{1.65 g}{\nu^2} \right)^{\frac{1}{3}} \]  (eq. TS14B–42)

\[ \theta_c = 0.02 \] 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):

\[ \theta_c = \frac{0.24}{D_m} + 0.055 \left[ 1 - \exp \left( -0.02 D_m \right) \right] \]

**Figure TS14B–14** Relative relationships between progression of alluvial bedforms and flow intensity

<table>
<thead>
<tr>
<th>Bedform</th>
<th>Plane bed</th>
<th>Ripples</th>
<th>Dunes</th>
<th>Transition</th>
<th>Plane bed</th>
<th>Standing waves and antidunes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water surface</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bed</td>
<td></td>
<td></td>
<td></td>
<td>Transition</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Resistance to flow (Manning’s roughness coefficient)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower regime</td>
<td></td>
<td></td>
<td></td>
<td>Upper regime</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transition</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper flow regime</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Stream power

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:

$$\Delta = 0.11 D_{0.5} y^{0.7} \left(1 - \exp\left(-0.5 T_{ts}\right)\right) \left(25 - T_{ts}\right)$$

(eq. TS14B–43)

where:

- $\Delta$ = 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:

$$z_{st} = \frac{\Delta}{2}$$

(eq. TS14B–44)

Some empirical formulas (eq. TS14B–35) that are based on data sets from sand-bed streams implicitly include bedform scour. Usually bedform scour is much smaller in magnitude than other types of scour in sand-bed rivers.

Consult reviews by Garcia (in press) and Yang (1996) for more information on bedforms.

### Scour associated with structures

**Structures that span the full width of the channel**

Structures that span the full width of the channel include sills, grade control structures, and structures comprised of boulders. The latter are intended to create step-pool morphology in steep, coarse-bed streams. Sills may be thought of as very low weirs, and grade control structures are higher weirs with associated appurtenances. These are used for bed erosion control and pool habitat development (fig. TS14B–15). Figure TS14B–15(a) shows a weir immediately after construction in a sand- and gravel-bed stream. The view shown is facing upstream. Central notch was constructed with invert at existing streambed elevation, and figure TS14B–15(b) is facing downstream from the notch about 6 months later.
Predicting scour depths downstream from weirs and grade control structures is too complex for theoretical calculations. Empirical formulas are used and are based on laboratory flume tests and field data. The scour equations are intended to allow prediction of scour depths in unprotected noncohesive alluvial beds. Commonly, grade control structures are built with preformed, stone-protected downstream scour holes (also called stilling basins). In some cases, these basins are sized using scour prediction equations. Since the equations provided below are empirical formulas, the engineer should become familiar with the original references and apply the formulas with care if the project falls outside the ranges of parameters used to generate them. A more comprehensive treatment of this topic is provided by Simons and Sentürk (1992).

**Sills**

Series of relatively low weirs (sills) are often used to develop pool habitats and to prevent mild to moderate bed degradation. Often these structures are installed by excavating a trench in the bed perpendicular to the flow and placing the structure into the trench so that the initial crest elevation is at bed elevation. Subsequent erosion produces a pool-and-riffle profile. Lenzi et al. (2002) reviewed work by others and conducted a series of flume experiments, resulting in different formulas for low gradient (slope ≤ 0.02) and high gradient (S ≥ 0.08) mountain streams.

For low gradient streams, the scour depth, zₘ (fig. TS14B–16), is given by:

\[
\frac{z_s}{H_s} = 0.180 \frac{a_1 \Delta S}{\Delta s D_{50}} + 0.369
\]  
(eq. TS14B–45)

and the length of the scour pool is given by:

\[
\frac{L_p}{H_s} = 4.479 + 0.023 \left( \frac{a_1}{H_s} \right)^{1.908} + 2.524 \left( \frac{a_1}{\Delta S D_{50}} \right)^{1.129}
\]  
(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:
  \[H_s = 1.5 \times \sqrt{\frac{q^2}{g}}\]
  (eq. TS14B–49)

where:
- \(q\) = flow per unit width over the sill at design discharge, \(\text{ft}^3/\text{s} \times \text{ft}^2\) (m³/s/m)
- \(g\) = acceleration of gravity, 32.2 ft/s² (9.81 m/s²)
- \(a_1\) = the “morphological jump” = \(a_1 = (S_o - S_{eq})L_s\)

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:
  \[S_{eq} = \frac{\theta_c \Delta S D_{50}}{y}\]
  (eq. TS14B–50)

**Figure TS14B–16**  
Definition sketch for computing scour associated with sills
Grade control structures and weirs

Weirs such as grade control structures (fig. TS14B–18) differ from sills in that they are built with crest elevations well above the existing bed. They normally produce backwater effects at baseflow. Several empirical approaches are available for computing the depth of scour in unprotected, noncohesive beds downstream from a vertical weir. Most of these equations were originally developed to compute the depth of scour downstream from dams. The Veronese (1937) equation yields an estimate of erosion measured from the tailwater surface to the bottom of the scour hole:

\[ y + z_s = K h_d^{0.225} q^{0.54} \]  

(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, ft³/s/ft (m³/s/m)

Step-pool structures

Thomas et al. (2000) studied natural step-pools in eight coarse grained mountain streams in Colorado and developed regression equations for design of step-pool structures in steep, boulder-bed streams (fig. TS14B–17):

\[ \frac{z_s}{W} = -0.0118 + 1.394 \left( \frac{h_d}{W} \right) + 5.514 \left( \frac{S_{q_{25}}}{W^2 \sqrt{g}} \right) \]  

(eq. TS14B–51)

and

\[ \frac{l_p}{W} = 0.409 + 4.211 \left( \frac{h_d}{W} \right) + 87.341 \left( \frac{S_{q_{25}}}{W^2 \sqrt{g}} \right) \]  

(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), ft³/s/ft (m³/s/m)
- \( g \) = acceleration of gravity, 32.2 ft/s² (9.81 m/s²)
- \( l_p \) = length of scour pool, ft (m)
The Veronese (1937) equation was modified by Yildiz and Üzücek (1994) to include the angle of the weir overfall jet, $\alpha$

$$y + z_s = Kh_d^{0.225}q^{0.54}\cos \alpha$$  \hspace{1cm} (eq. TS14B–54)

where:

$\alpha$ = angle the incident jet makes with the vertical.

A vertical overfall of water from a cantilevered pipe or sharp-crested weir would have $\alpha=0$.

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:

$$z_s = K \frac{q^{a}h_d^{b}y_t^{c}}{g^{d}}$$  \hspace{1cm} (eq. TS14B–55)

where:

$z_s$ = depth of scour (m)

$K = 6.42 - 3.2h_d^{0.10}$  \hspace{1cm} (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)

$a = 0.6 - h_d/300$  \hspace{1cm} (eq. TS14B–57)

$b = 0.15 h_d/200$  \hspace{1cm} (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**  \hspace{1cm} (a) Low- and (b) high-drop grade control structures

(a) ![Low-drop structure](image1)

(b) ![High-drop structure](image2)
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):

\[
\frac{z_s}{y_w} = 0.540 \left( \frac{W_w}{y_w} \right)^{0.593} \left( \frac{y_t}{h_d} \right)^{-0.126} A_{50}^{0.544} \left( \frac{D_{50}}{D_{90}} \right)^{-0.856} \left( \frac{W_w}{W} \right)^{-0.751}
\]

(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:

\[
A_{50} = \frac{Q_d}{W_w y_w \sqrt{g D_{50} \Delta S_g}}
\]

(eq. TS14B–60)

where:
- \( \Delta S_g \) = relative submerged density of bed-material sediments ≈ 1.65
- \( Q_d \) = design discharge, ft³/s (m³/s)

The following relationship was recommended for estimating the horizontal distance between the weir and the deepest point in the scour hole:

\[
\frac{L_u}{y_w} = 1.616 \left( \frac{W_w}{y_w} \right)^{0.662} \left( \frac{y_t}{h_d} \right)^{-0.117} A_{90}^{0.455} \left( \frac{W_w}{W} \right)^{-0.478}
\]

(eq. TS14B–61)

The quantity \( A_{90} \) is similar to \( A_{50} \):

\[
A_{90} = \frac{Q}{W_w y_w \sqrt{g D_{90} \Delta S_g}}
\]

(eq. TS14B–62)

Sloping drop structures such as rock ramps or Newbury riffles may be attractive options in some stream restoration projects, particularly from an aesthetic and fish passage standpoint. Laursen, Flick, and Ehlers (1986) ran a limited number of flume experiments with sloping sills with slopes of 4H:1V and produced the following relationship:

\[
\frac{y_2}{y_c} = 4 \left( \frac{y_c}{D_s} \right)^{0.2} - 3 \left( \frac{D_r}{y_c} \right)^{0.1}
\]

(eq. TS14B–63)

where:
- \( y_2 \) = depth of water in downstream channel after scour, ft (m)
- \( y_c \) = critical flow depth for the design unit discharge, ft (m)
- \( D_s \) = characteristic bed sediment size, assumed to be median \( D_{50} \) ft (m) (note units)
- \( D_r \) = characteristic size of rock or riprap used to build the sloping structure, assumed to be median \( D_{50} \) ft (m) (note units).

The depth of scour, \( z_s \), is given by:

\[
z_s = y_2 - y_1
\]

(eq. TS14B–64)

where:
- \( y_1 \) = depth of water in downstream channel before scour, ft (m)

The analysis and design of grade control structures is also described in NEH654 TS14G.
Structures that partially span the channel

Structures that protrude from one bank into the channel include groins (groynes), spur dikes (spurs), deflectors, bank barbs, and bendway weirs. Kuhnle, Alonso, and Shields (1999, 2002) conducted a series of clear-water, steady-flow, movable-bed flume studies using various spur dike geometries and measured the depth and volume of scour adjacent to the spurs. Empirical formulas for scour depth were developed based on earlier work by Melville (1992). The Kuhnle 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).

The Kuhnle formulas are:

$$\frac{z_s}{y} = K_1 \left( \frac{L_c}{y} \right)^a$$  \hspace{1cm} (eq. TS14B–65)

and

$$\frac{V_s}{z_s^3} = K_2 \left( \frac{L_c}{y} \right)^b$$  \hspace{1cm} (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–20** Scour associated with stone spur dike

(a)

(b)

**Figure TS14B–21** Definition sketch showing crest length, $L_c$, and side slope angle, $\theta$, for spur dikes
The exponent $a$ is a dimensionless exponent that varies with $L_c/y$. It has a value of $a = 1$ for $L_c/y < 1$, $a = \frac{1}{2}$ for $1 < L_c/y < 25$, and $a = 0$ for $L_c/y > 25$.

$K_2 = \text{dimensionless coefficient that varies with the angle the spur crest makes with the approach flow}$

$K_2 = 17.106 \text{ for perpendicular spurs, and } K_2 = 12.11 \text{ for spurs that are at a nonperpendicular angle (45° or 135°)}$

$b = \text{dimensionless exponent that varies with spur crest angle. } b = -0.781 \text{ for perpendicular spurs, and } b = 0 \text{ in other cases}$

Rahman and Haque (2004) suggested that $K_1$ be modified to reflect the shape of the spur cross section for shorter spurs ($L_c/y < 10$):

$$K_1 = 0.75 \left( 1 + \frac{2 \tan \phi}{\tan \theta} \right)^{\frac{1}{2}} \quad \text{(eq. TS14B–67)}$$

where:

$\phi = \text{angle of repose of bed sediment}$

$\theta = \text{side slope of spur structure (fig. TS14B–21)}$

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

<table>
<thead>
<tr>
<th>Predominant bed material</th>
<th>Long-term bed elevation change</th>
<th>General scour</th>
<th>Local scour</th>
<th>All types of scour</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Armoring analysis</td>
<td>Equilibrium slope</td>
<td>Contraction scour</td>
<td>Bend scour</td>
</tr>
<tr>
<td>Clay or silt, cohesive</td>
<td>X □</td>
<td>✓ Regional regressions (fig. 8)</td>
<td>O</td>
<td>O</td>
</tr>
<tr>
<td>Fine gravel &lt;6 mm</td>
<td>✓ (2–4)</td>
<td>✓</td>
<td>O</td>
<td>X</td>
</tr>
<tr>
<td>Gravel &gt;6 mm, cobble</td>
<td>✓ (14–17)</td>
<td>✓</td>
<td>O</td>
<td>X</td>
</tr>
<tr>
<td>Boulders</td>
<td>O O</td>
<td>✓</td>
<td>O</td>
<td>X</td>
</tr>
</tbody>
</table>

✓ = 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:**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho$ <em>water</em></td>
<td>1,000 kg/m$^3$</td>
</tr>
<tr>
<td>$\rho$ <em>solids</em></td>
<td>2,630</td>
</tr>
<tr>
<td>relative submerged density, $\Delta S_g$</td>
<td>1.63</td>
</tr>
<tr>
<td>Manning's equation equation</td>
<td>1</td>
</tr>
<tr>
<td>Shields constant $\theta_c$</td>
<td>0.038215</td>
</tr>
<tr>
<td>roughness $k_s$</td>
<td>2.8 mm</td>
</tr>
<tr>
<td>$D_{95}$</td>
<td>1.5 mm</td>
</tr>
<tr>
<td>$D_{90}$</td>
<td>0.96 mm</td>
</tr>
<tr>
<td>$D_{84}$</td>
<td>0.8 mm</td>
</tr>
<tr>
<td>$D_{\text{mean}}$</td>
<td>0.28 mm</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>0.3 mm</td>
</tr>
<tr>
<td>bed sediment internal friction angle</td>
<td>45 deg</td>
</tr>
<tr>
<td>distance to downstream base level control</td>
<td>2,000 m</td>
</tr>
<tr>
<td>Manning's $n$</td>
<td>0.027 s/m$^{1/3}$</td>
</tr>
<tr>
<td>design discharge, $Q_d$</td>
<td>392.2 m$^3$/s</td>
</tr>
<tr>
<td>flow width</td>
<td>60 m</td>
</tr>
<tr>
<td>channel width, $W$</td>
<td>70 m</td>
</tr>
<tr>
<td>mean flow depth, $y$</td>
<td>3.0 m</td>
</tr>
<tr>
<td>hydraulic radius, $R$</td>
<td>3.0 m</td>
</tr>
<tr>
<td>mean streamwise velocity, $u$</td>
<td>2.2 m/s</td>
</tr>
<tr>
<td>unit discharge, $q$</td>
<td>6.5 m$^3$/s/m</td>
</tr>
<tr>
<td>unit discharge, 25 yr, $q_{25}$</td>
<td>30 m$^3$/s/m</td>
</tr>
<tr>
<td>existing bed slope, $S$</td>
<td>0.0008 m/m</td>
</tr>
<tr>
<td>bend radius of curvature, $R_c$</td>
<td>1,000 m</td>
</tr>
<tr>
<td>$L$, length of spur crest</td>
<td>20 m</td>
</tr>
<tr>
<td>spur side slope</td>
<td>2H:1V</td>
</tr>
<tr>
<td>spur crest above water surface?</td>
<td>N</td>
</tr>
</tbody>
</table>
Scour Calculations

**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.

\[
D_x = K \left( \frac{y_S e}{\Delta S_g} \right)^{a} \left( \frac{U_s}{v} \right)^b \\
U_s = (g y S_e)^0.5
\]

Assume \( S_e = S_o \)

\[
U_s = \sqrt{32.2 \times 9.8 \times 0.0008} = 0.50 \text{ ft/s}
\]

Particle Re = \( \frac{U_s D_{so}}{v} = 0.50 \times 0.001 = 48 \times 10^{-5} \)

Therefore, \( K = 27 \), \( a = 0.86 \), \( b = -0.14 \).

\[
D_x = 27 \left( \frac{9.8 \times 0.0008}{1.63} \right)^{0.86} \left( \frac{0.50 \times 1.05 \times 10^{-5}}{1.05 \times 10^{-5}} \right)^{-0.14} = 0.06 \text{ ft} \\
\]

\( = 18 \text{ mm} \)

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.


\[
a = 0.025 n \left[ (2.29 - 0.85 \log D_{so}) \right] (D_{so} - 0.07)^{0.14} \\
= 0.025 (0.027) (2.29 - 0.85 \log D_{so}) (0.3 - 0.07)^{0.14} = 1.21 \times 10^{-6} \\
b = 4.93 - 0.74 \log D_{so} \\
= 4.93 - 0.74 \log (0.3) = 5.32 \\
c = -0.46 + 0.65 \log D_{so} \\
= -0.46 + 0.65 \log (0.3) = -0.80
\]

\[
q_s = a u^b y^c \\
q_s = (1.21 \times 10^{-6}) (7.1)^{5.32} (9.8)^{-0.80} = 0.0066 \text{ ft}^2/\text{s}
\]

b. Compute equilibrium slope (eq. TS14B–12).

\[
S_{eq} = \left( \frac{a}{q_s} \right)^{10/3} g_s^{10/3} \left( \frac{u}{K} \right)^{2/3} \\
S_{eq} = \left( \frac{1.21 \times 10^{-6}}{0.066} \right)^{0.54} \times 0.027 \frac{2}{1.486} = 0.00083
\]

Since the existing channel slope is approximately equal to the equilibrium slope, long-term degradation should be minimal.

**Step 3** Compute contraction scour.


\[
V_c = \frac{1}{K} \frac{1}{D_{so}^{3/2}} \\
V_c = 11.17 \times (9.8)^{1/2} (0.001)^{1/2} = 1.60 \text{ ft/s}
\]

Since \( u = 7.1 \text{ ft/s} > 1.6 \text{ ft/s} \), live bed conditions occur.


\[
A = \frac{\Delta S_g D^3_s}{v^2} \\
A = \frac{1.63 \times 32.2 \times (0.001)^3}{\left(1.05 \times 10^{-5}\right)^2} = 500 \\
K_1 = 0.055 \tanh \left[ 12A^{-0.50} \exp \left( -0.0004A \right) \right] \\
K_1 = 0.055 \tanh \left[ 12(500)^{-0.50} \exp \left( -0.0004(500) \right) \right] = 0.014 \\
K_2 = 1.06 \tanh \left[ 0.016A^{0.50} \exp \left( -\frac{120}{A} \right) \right] \\
K_2 = 1.06 \tanh \left[ 0.016(500)^{0.50} \exp \left( -\frac{120}{500} \right) \right] = 0.291 \\
\omega = K_1 \Delta S_g D^3_s / v + K_2 \sqrt{\Delta S_g D_s}
\]
\[ \omega = \frac{0.014 \times 1.63 \times 32.2 \times (0.001)^2}{(1.05 \times 10^{-5})} + (0.291) \sqrt{1.63 \times 32.2 \times 0.001} = 0.135 \text{ ft/s} \]

c. Compute \( U_*/\omega \).
\[ U_* = \sqrt{32.2 \times 9.8 \times 0.0008} = 0.50 \text{ ft/s} \]
\[ \frac{U_*}{\omega} = 0.50 \times \frac{1}{0.135} = 3.72 \text{ ft/s} \]
d. Using \( U_*/\omega \), look up \( a \to a = 0.69 \) (table TS14B–10).
e. Compute \( y_2 \) with equation TS14B–26, assuming \( y_1 = y_0 = 9.8 \text{ ft} \), and since \( Q_1 = Q_2' \).
\[ \frac{y_2}{y_1} = \left( \frac{Q_2}{Q_1} \right)^{1.36} \left( \frac{W_{bt}}{W_{bt}'} \right)^a = 1.32 \]
\[ y_2 = \frac{1.32 \times 9.8}{40} = 13.0 \text{ ft} \]
\[ z_c = y_2 - y_0 = 13.0 - 9.8 = 3.2 \text{ ft} \]

**Step 4** Compute bedform scour.
For dunes to form (eq. TS14B–37):
\[ D_* = D_{so} \left( \frac{\Delta S \times g}{v^2} \right)^{\frac{1}{3}} > 10 \]
\[ D_* = 0.001 \left( \frac{1.63 \times 32.2}{(1.05 \times 10^{-5})^2} \right)^{\frac{1}{3}} = 7.8 < 10 \]
Dunes should not form.

**Step 5** Compute local scour at spur dikes using equations TS14B–65 and TS14B–67.
\[ K_1 = 0.75 \left( 1 + \frac{2 \tan \phi}{\tan \phi} \right)^{\frac{1}{2}} \]
\[ K_1 = 0.75 \left( 1 + \frac{2 \tan (45)}{\tan (27)} \right)^{\frac{1}{2}} = 0.34 \]
\[ \frac{z_s}{y} = K_1 \left( \frac{L_s}{y} \right)^{1.3} \]
\[ \frac{z_s}{y} = 0.34 \left( \frac{65.6}{9.8} \right)^{0.5} = 0.88 \]
\[ z_s = 0.88y \]
\[ = 0.88 \times 9.8 \]
\[ = 8.6 \text{ ft} \]

**Step 6** Compute total scour (eq. TS14B–1).
\[ z_t = FS [z_{as} + z_c + z_b + z_{et} + z_s] \]
\[ z_t = 1.3[0 + 3.2 + 0 + 0 + 8.6] = 15.4 \text{ ft} \]

\[ z_t = 6.5 (0.001)^{0.115} = 14.4 \text{ ft} \]

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, $\delta S_g = 1.63$ and $1.63$
- Angle of repose of sediment, $\phi = 45$ deg and 0.79 rad
- Constant in Manning’s equation, $C = 1$ and 1.486
- C in Strickler equation, $C_{Strickler} = 0.034$
- Shields constant $\theta_c = 0.056$ and 0.055653
- Grain roughness $k_s = 87.5$ mm and 0.287 ft
- $D_{95} = 175$ mm and 0.574 ft
- $D_{90} = 65$ mm and 0.213 ft
- $D_{84} = 25$ mm and 0.082 ft
- $D_{mean} = 15$ mm and 0.049 ft
- $D_{50} = 13$ mm and 0.043 ft
- Bed sediment internal friction angle, $\phi = 45$ deg and 0.785 rad
- Distance to downstream base level control, $L = 2,000$ m and 6,562 ft
- Manning’s $n$, $n_{Manning} = 0.030$ s/m$^{(1/3)}$ and 0.030 s/ft$^{(1/3)}$
- Design discharge, $Q_d = 40.5$ m$^3$/s and 1,430 ft$^3$/s
- Flow width, $W = 18$ m and 60 ft
- Channel width, $W = 19$ m and 62 ft
- Mean flow depth, $y = 1.2$ m and 3.9 ft
- Hydraulic radius, $R = 1.2$ m and 3.9 ft
- Mean streamwise velocity, $u = 1.8$ m/s and 6.1 ft/s
- Unit discharge, $q = 2.2$ m$^3$/s/m and 24 ft$^3$/s/ft
- Existing bed slope, $S = 0.0024$ m/m and 0.0024 ft/ft
- Bend radius of curvature, $R_c = 1,000$ m and 3,281 ft

- Distance between weirs, $L = 250$ m and 820.3 ft
- Weir height above upstream bed, $y_w = 0.3$ m and 1.0 ft
- Weir width, $W_w = 15$ m and 49.2 ft
- Difference in upstream and downstream water surface, $y_d = 0.5$ m and 1.6 ft
- Water depth above uneroded bed, $y_d = 0.7$ m and 2.3 ft
- Angle of the overfall jet with the vertical, $\theta = 0$ Deg and 0.0 rad
Find: Total predicted scour depth

Step 1  Compute bed elevation change due to reach-scale degradation based on equilibrium slope.


$$D_x = K \left( \frac{yS_e}{\Delta S_g} \right)^a \left( \frac{U_\ast}{\nu} \right)^b$$

$U_\ast = (gyS_e)^{0.5}$. Assume $S_e = S_o$

$$U_\ast = \sqrt{32.2 \times 3.9 \times 0.0024} = 0.55 \text{ ft/s}$$

Particle Re = $\frac{U \cdot D_{so}}{\nu} = \frac{0.55 \times 0.043}{1.05 \times 10^{-5}} = 2,252$

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:

$$e = 0.245 + \frac{0.0864}{(0.1D_{so})^{0.21}} = 0.245 + \frac{0.0864}{(0.1 \times 30)^{0.21}} = 0.327$$

since $D_{so} = 30 \text{ mm}$, $P_a = 0.16$. 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)

$$S_{eq} = \left[ \theta \cdot D_{s} \cdot \Delta S_{g} \right]^{\frac{10}{7}} \left( \frac{K}{qn} \right)^{\frac{6}{7}}$$

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)

$$S_{eq} = K \left( \frac{D_{so}}{q} \right)^{\frac{10}{7}} n^{\frac{9}{7}}$$

$$S_{eq} = 60.1 \left( \frac{0.043}{0.03} \right)^{\frac{10}{7}} \left( \frac{0.065}{24} \right)^{\frac{6}{7}} = 0.0013$$

c. Schoklitsch (eq. TS14B–16)

$$S_{eq} = K \left( \frac{D_{so}}{q} \right)^{\frac{3}{7}}$$

$$S_{eq} = 0.00174 \left( \frac{15}{24} \right)^{\frac{3}{7}} = 0.0012$$

d. Henderson (eq. TS14B–17)

$$S_{eq} = K Q^{0.46} D_{50}^{1.15}$$

$$S_{eq} = 0.44 \left( 1.430 \right)^{0.46} (0.043)^{1.15} = 0.00042$$

e. Compute bed degradation using equation TS14B–6. Use the average of the first three $S_{eq}$ values computed above = 0.0011.

$$z_{ad} = L \left( S_{eq} - S_{eq} \right)$$

$$z_{ad} = (6.562)(0.0024 - 0.0011) = 8.5 \text{ ft}$$

Since the armor layer is formed after 0.81 ft of degradation, armoring controls.
Step 3  Compute scour downstream from weirs.

\[
y + z_s = Kh_d 0.225 q^{0.54}
\]
\[
y + z_s = 1.32(1.6)^{0.225} (23)^{0.54} = 8.2 \text{ ft}
\]
\[
z_s = 8.2 - y
\]
\[
= 8.2 - 3.9
\]
\[
= 4.3 \text{ ft}
\]

\[
z_s = K \frac{q^{0.15} h_d^{0.45}}{g^{0.6} D_m^{0.10}}
\]
\[
z_s = 3.4 \left(\frac{2.2}{9.8}\right)^{0.15} \left(\frac{0.5}{0.25}\right)^{0.15} = 2.7 \text{ m}
\]

\[
A_{50} = \frac{Q_d}{W_w y_w \sqrt{gD_{50}\Delta S_g}}
\]
\[
A_{50} = \frac{1.430}{(49.2)(1.0)\sqrt{32.2 \times 0.043 \times 1.63}} = 19.3
\]
\[
\frac{z_s}{y_w} = 0.540 \left(\frac{W_w}{y_w}\right)^{0.593} \left(\frac{y_t}{h_d}\right)^{-0.126} A_{50}^{0.544} \left(\frac{D_{50}}{D_{50}}\right)^{-0.856} \left(\frac{W_w}{W}\right)^{-0.751}
\]
\[
\frac{z_s}{y_w} = 0.540 \left(\frac{15}{0.3}\right)^{0.593} \left(\frac{0.7}{0.5}\right)^{-0.126} (19.3)^{0.544} \left(\frac{0.213}{0.043}\right)^{-0.856} \left(\frac{49.2}{62}\right)^{-0.751} = 7.96
\]
\[
z_s = 7.96 \times 3.1 = 2.4 \text{ m}
\]

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.

Step 4  Compute total scour for design using equation TS14B–1.
\[
z_t = FS \left[ z_m + z_e + z_b + z_{bf} + z_s \right]
\]
Use a factor of safety of 1.3.
\[
z_t = 1.3 \left[ 0.8 + 0.0 + 0.0 + 0.0 + 8.9 \right] = 12.6 \text{ ft}
\]

\[
z_t (\text{max}) = K D_{50}^{-0.11}
\]
\[
z_t \text{ max} = 6.5 \left(0.043\right)^{0.115} = 9.3 \text{ ft}
\]

Estimate depth of scour pools below weirs as the maximum of the above three results.
Design features and measures to address scour

Structures may be designed to withstand scour in either of two ways (fig. TS14B–22 (USACE 1991b)). They may be extended down into the bed a sufficient distance (dig it in or key it in) to be beneath the projected total scour depth (method A, fig. TS14B–22) or until contact is made with a nonerodible material (method B, fig. TS14B–22). The key-it-in approach (method A) is most often used with armor revetment (Biedenharn, Elliott, and Watson 1997), but is difficult and costly to do in a flowing stream. Conventional excavation is usually not feasible in water depths >10 feet (3 m). Greater water depths usually require dredging or de-watering 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

Method A

Method B

Method C

Method D
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,

$$V_{\text{stone}} = 3.35T_rz_t$$

(eq. TS14B–68)

where:

- $V_{\text{stone}}$ = 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

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a_1)</td>
<td>morphological jump (= (S_0 - S_{eq})L_s) ft (m)</td>
</tr>
<tr>
<td>(\Delta)</td>
<td>dune height, ft (m)</td>
</tr>
<tr>
<td>(D_s)</td>
<td>dimensionless sediment size</td>
</tr>
<tr>
<td>(D_{90})</td>
<td>median bed-material size, mm or ft (m)</td>
</tr>
<tr>
<td>(D_{90})</td>
<td>size larger than 90 percent of the bed material by weight, mm or ft (m)</td>
</tr>
<tr>
<td>(D_{90})</td>
<td>size of bed material larger than 90% of the bed by weight, ft (m) (note units)</td>
</tr>
<tr>
<td>(D_s)</td>
<td>the smallest armor size or the size of the smallest nontransportable particle present in the bed material, ft (m)</td>
</tr>
<tr>
<td>(D_c)</td>
<td>diameter of the sediment particle, mm or ft (m)</td>
</tr>
<tr>
<td>(D_m)</td>
<td>mean bed-material particle size, mm or ft (m)</td>
</tr>
<tr>
<td>(D_a)</td>
<td>a characteristic sediment diameter, ft (m)</td>
</tr>
<tr>
<td>(\Delta S_g)</td>
<td>change in relative submerged density of bed-material sediments (\pm 1.65)</td>
</tr>
<tr>
<td>(\Delta V)</td>
<td>change in volume of bed-material sediment stored or eroded, (ft^3 (m^3))</td>
</tr>
<tr>
<td>(e)</td>
<td>porosity of the bed material</td>
</tr>
<tr>
<td>(\phi)</td>
<td>angle of repose of bed sediment</td>
</tr>
<tr>
<td>(FS)</td>
<td>factor of safety</td>
</tr>
<tr>
<td>(g)</td>
<td>acceleration of gravity, (32.2 \text{ ft/s}^2 ) ((9.81 \text{ m/s}^2))</td>
</tr>
<tr>
<td>(\gamma_s)</td>
<td>specific weight of sediment particles (lb/ft^3 (N/m^3))</td>
</tr>
<tr>
<td>(\gamma_w)</td>
<td>specific weight of water, (lb/ft^3 (N/m^3))</td>
</tr>
<tr>
<td>(h_a)</td>
<td>height of step crest above controlling bed elevation at downstream end of pool, ft (m)</td>
</tr>
<tr>
<td>(h_d)</td>
<td>vertical distance between water surface elevation upstream and downstream from the weir, ft (m)</td>
</tr>
<tr>
<td>(H_s)</td>
<td>specific energy of critical flow over the sill, ft (m)</td>
</tr>
<tr>
<td>(\varphi)</td>
<td>side slope of spur structure</td>
</tr>
<tr>
<td>(L_s)</td>
<td>reach length, ft (m)</td>
</tr>
<tr>
<td>(L_c)</td>
<td>length of spur crest measured perpendicular to flow direction, ft (m)</td>
</tr>
<tr>
<td>(L_p)</td>
<td>recommended length of protection, ft (m)</td>
</tr>
<tr>
<td>(L_s)</td>
<td>horizontal distance between sills, ft (m)</td>
</tr>
<tr>
<td>(l_p)</td>
<td>length of scour pool, ft (m)</td>
</tr>
<tr>
<td>(L_{max})</td>
<td>horizontal distance between weir and deepest point of downstream scour hole, ft (m)</td>
</tr>
<tr>
<td>(n)</td>
<td>Manning's roughness coefficient</td>
</tr>
<tr>
<td>(\nu)</td>
<td>kinematic viscosity of water, (ft^2/s ) ((m^2/s))</td>
</tr>
<tr>
<td>(P_x)</td>
<td>the fraction of bed material comprised of particles size (D_a) or larger</td>
</tr>
<tr>
<td>(q)</td>
<td>channel-forming or design discharge per unit width, (ft^3/s (m^3/s))</td>
</tr>
<tr>
<td>(\theta)</td>
<td>dimensionless Shields stress</td>
</tr>
<tr>
<td>(\theta_c)</td>
<td>critical dimensionless shear stress or Shields constant</td>
</tr>
<tr>
<td>(Q_d)</td>
<td>design discharge, (ft^3/s (m^3/s))</td>
</tr>
<tr>
<td>(Q_s)</td>
<td>sediment supply, (ft^3/s (m^3/s))</td>
</tr>
<tr>
<td>(q_s)</td>
<td>sediment transport capacity in dimensions of volume per unit width per unit time, (ft^2/s ) ((m^2/s))</td>
</tr>
<tr>
<td>(\rho)</td>
<td>density of water, (1.94 \text{ slugs/ft}^3 ) ((1,000 \text{ kg/m}^3))</td>
</tr>
<tr>
<td>(R)</td>
<td>hydraulic radius, ft (m)</td>
</tr>
<tr>
<td>(R_c)</td>
<td>bend radius of curvature, ft (m)</td>
</tr>
<tr>
<td>(S)</td>
<td>average channel bed slope</td>
</tr>
<tr>
<td>(S_e)</td>
<td>energy slope</td>
</tr>
<tr>
<td>(S_{eq})</td>
<td>slope of energy grade line of main channel, (ft/ft ) ((m/m))</td>
</tr>
<tr>
<td>(S_{eq})</td>
<td>equilibrium channel slope at which sediment particles of size (D_a) and larger will no longer move</td>
</tr>
<tr>
<td>(S_{ex})</td>
<td>existing channel slope</td>
</tr>
<tr>
<td>(S_g)</td>
<td>specific gravity of the sediment</td>
</tr>
<tr>
<td>(T)</td>
<td>thickness of the active layer of the bed, ft (m)</td>
</tr>
<tr>
<td>(T_c)</td>
<td>critical boundary shear stress, (lb/ft^2 ) ((N/m^2))</td>
</tr>
<tr>
<td>(T^*)</td>
<td>critical shear stress for motion from the Shields diagram, (lb/ft^2 ) ((N/m^2))</td>
</tr>
<tr>
<td>(\tau_o)</td>
<td>average bed shear stress, (lb/ft^2 ) ((N/m^2))</td>
</tr>
<tr>
<td>(T_r)</td>
<td>thickness of the bank revetment, ft (m)</td>
</tr>
<tr>
<td>(\tau_s)</td>
<td>bed shear stress due to skin or grain friction, (lb/ft^2 ) ((N/m^2))</td>
</tr>
<tr>
<td>(T_{ts})</td>
<td>dimensionless transport-stage parameter</td>
</tr>
<tr>
<td>(u)</td>
<td>mean streamwise velocity, (ft/s ) ((m/s))</td>
</tr>
<tr>
<td>(U_s)</td>
<td>shear velocity (= (\gamma S_e)^{0.5})</td>
</tr>
<tr>
<td>Symbol</td>
<td>Description</td>
</tr>
<tr>
<td>--------</td>
<td>-------------</td>
</tr>
<tr>
<td>$V_c$</td>
<td>critical velocity, ft/s (m/s)</td>
</tr>
<tr>
<td>$V_{\text{stone}}$</td>
<td>additional volume of stone added to toe for launching per unit streamwise length of revetment, ft$^3$/ft (m$^3$/m)</td>
</tr>
<tr>
<td>$V_s$</td>
<td>volume of scour hole, ft$^3$ (m$^3$)</td>
</tr>
<tr>
<td>$V_{S_{\text{m}}}$</td>
<td>volume of bed-material sediment supplied to the reach, ft$^3$ (m$^3$)</td>
</tr>
<tr>
<td>$V_{S_{\text{out}}}$</td>
<td>volume of bed-material sediment transported out of the reach, ft$^3$ (m$^3$)</td>
</tr>
<tr>
<td>$W$</td>
<td>average active channel width, ft (m)</td>
</tr>
<tr>
<td>$\omega$</td>
<td>fall velocity of bed material based on the $D_{50}$, ft/s (m/s)</td>
</tr>
<tr>
<td>$W_{b1}$</td>
<td>bottom width of the upstream channel, ft (m)</td>
</tr>
<tr>
<td>$W_{b2}$</td>
<td>bottom width of the contracted section, ft (m)</td>
</tr>
<tr>
<td>$W_c$</td>
<td>average channel width, ft (m)</td>
</tr>
<tr>
<td>$W_f$</td>
<td>flow width at design discharge, ft (m)</td>
</tr>
<tr>
<td>$W_i$</td>
<td>channel width at bend inflection point, ft (m)</td>
</tr>
<tr>
<td>$W_w$</td>
<td>width of weir crest, ft (m)</td>
</tr>
<tr>
<td>$y$</td>
<td>flow depth, ft (m)</td>
</tr>
<tr>
<td>$y_c$</td>
<td>mean water depth in the crossing upstream from the bend, ft (m)</td>
</tr>
<tr>
<td>$y_{\text{max}}$</td>
<td>maximum flow depth in the bend, ft (m)</td>
</tr>
<tr>
<td>$y_t$</td>
<td>tailwater depth above original ground surface, m</td>
</tr>
<tr>
<td>$y_w$</td>
<td>vertical distance between weir crest and upstream channel bed, ft(m)</td>
</tr>
<tr>
<td>$z_{\text{ad}}$</td>
<td>bed elevation changes due to reach-scale deposition (aggradation) or general scour (degradation), ft (m)</td>
</tr>
<tr>
<td>$z_b$</td>
<td>scour on the outside of bend, ft (m)</td>
</tr>
<tr>
<td>$z_{\text{bf}}$</td>
<td>bedform trough depth, ft (m)</td>
</tr>
<tr>
<td>$z_c$</td>
<td>clear-water contraction scour, ft (m)</td>
</tr>
<tr>
<td>$z_s$</td>
<td>depth of scour downstream from structure, ft (m), measured from the crest of the structure to the lowest point within the scour pool</td>
</tr>
<tr>
<td>$z_{\text{s}}$</td>
<td>local scour depth associated with a structure, ft (m)</td>
</tr>
<tr>
<td>$z_t$</td>
<td>total scour depth, ft (m)</td>
</tr>
</tbody>
</table>
Technical Supplement 14C

Stone Sizing Criteria
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.
Contents

Purpose \( \text{TS14C–1} \)
Introduction \( \text{TS14C–1} \)
Basic concepts \( \text{TS14C–1} \)
Description of forces on a stone \( \text{TS14C–1} \)
Flow conditions \( \text{TS14C–2} \)
Sizing techniques \( \text{TS14C–3} \)
Summary guide of selected techniques \( \text{TS14C–11} \)
Factor of safety \( \text{TS14C–12} \)
Example calculations \( \text{TS14C–12} \)
Conclusion \( \text{TS14C–13} \)

Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS14C–1</td>
<td>High-energy vs. low-energy conditions</td>
<td>TS14C–2</td>
</tr>
<tr>
<td>TS14C–2</td>
<td>Federal Highway Administration techniques</td>
<td>TS14C–11</td>
</tr>
<tr>
<td>TS14C–3</td>
<td>Summary of techniques</td>
<td>TS14C–11</td>
</tr>
</tbody>
</table>

Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS14C–1</td>
<td>Forces on a submerged stone</td>
<td>TS14C–1</td>
</tr>
<tr>
<td>TS14C–2</td>
<td>Riprap used to control a headcut</td>
<td>TS14C–2</td>
</tr>
<tr>
<td>TS14C–3</td>
<td>Riprap used to prevent erosion from flow from a side inlet to a channel</td>
<td>TS14C–2</td>
</tr>
<tr>
<td>TS14C–4</td>
<td>Toe of the slope lined with stone to control erosion</td>
<td>TS14C–2</td>
</tr>
<tr>
<td>TS14C–5</td>
<td>Rock size based on Isbash Curve</td>
<td>TS14C–3</td>
</tr>
<tr>
<td>TS14C–6</td>
<td>Graphical solution for Isbash technique</td>
<td>TS14C–4</td>
</tr>
<tr>
<td>TS14C–7</td>
<td>Rock chute spreadsheet</td>
<td>TS14C–9</td>
</tr>
<tr>
<td>TS14C–8</td>
<td>Lane's method</td>
<td>TS14C–10</td>
</tr>
</tbody>
</table>
Purpose

Many channel protection techniques involve rock or stone as a stand-alone treatment or as a component of an integrated system. Stone used as riprap can also be a component of many streambank soil bioengineering projects. Many Federal and state agencies have developed methods and approaches for sizing riprap, and several of those techniques are briefly described in this document. Stone sizing methods are normally developed for a specific application, so care should be exercised in matching the selected method with the intended use. While many of these were developed for application with stone riprap revetments, they are also applicable for other designs involving rock, as well.

Introduction

When the attacking forces of flowing water exceed the resisting forces of the existing channel material, channel protection is needed as part of a restoration design. Channel protection typically ranges from soil bioengineering treatments to more traditional armor ing methods. Numerous methods have been developed for the design and sizing of riprap. Several common techniques for estimating the required stone size are briefly outlined in this document. The designer is encouraged to review the complete development of a selected method and assess the relevance of the assumptions behind that selected method to their application. In this document, the words rock and stone are used interchangeably.

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

Basic concepts

Description of forces on a stone

A rock will be stable until the lift and drag forces of moving water exceed a critical value or threshold. Therefore, for a given rock size subjected to a given force of moving water, there is some unit discharge where the rock will move and become unstable. Forces on a submerged stone, as indicated in figure TS14C–1, typically consist of the force exerted by the flowing water ($F_p$), 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](image-url)
Flow conditions

The flow conditions associated with a particular application will have a major influence on selecting the right rock sizing method. While it is difficult to select a single criterion that separates rock sizing methods, high energy and low energy are used in this development. For example, a technique developed for the design of a riprap blanket revetment in a low-energy environment would not necessarily be suitable for estimating the minimum stone size in a high-energy environment, where the stone projects into the flow. Such applications, including instream habitat boulders, grade stabilization, and stream barbs, should be addressed with impinging flow design techniques. Table TS14C–1 lists some of the flow descriptors that can be associated with high- and low-energy flow conditions. Photographs of the different energy conditions where stone is applied as part of the solution are shown in figures TS14C–2 through 4. In figure TS14C–2, riprap is used to control a headcut. Riprap chutes can be used to control erosion from a headcut in a channel or in a side inlet to a channel. Riprap for this type of structure would fall in the steep-slope, high-energy design. Figure TS14C–3 shows riprap used to prevent erosion from flow from a side inlet to a channel. This structure also prevents a headcut from moving into the field. As illustrated in figure TS14C–4, if the toe of the slope is eroding, and it cannot be controlled with bioengineering alone, lining the toe of the slope with stone may be a solution. Riprap for this type of structure would fall in the mild slope, low-energy design.

The appropriate rock sizing method must consider the flow energy associated with the particular application. While there are exceptions, most rock sizing methods were developed for either a high- or low-energy flow condition.

<table>
<thead>
<tr>
<th>Table TS14C–1</th>
<th>High-energy vs. low-energy conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>High energy</strong></td>
<td><strong>Low energy</strong></td>
</tr>
<tr>
<td>Supercritical flow</td>
<td>Subcritical flow</td>
</tr>
<tr>
<td>Steep slope</td>
<td>Mild slope</td>
</tr>
<tr>
<td>High turbulence</td>
<td>Low turbulence</td>
</tr>
<tr>
<td>Impinging flow</td>
<td>Parallel flow</td>
</tr>
<tr>
<td>Rapidly varied flow</td>
<td>Uniform or gradually varied flow</td>
</tr>
<tr>
<td>Unsteady flow</td>
<td>Steady flow</td>
</tr>
</tbody>
</table>

Figure TS14C–2 Riprap used to control a headcut

Figure TS14C–3 Riprap used to prevent erosion from flow from a side inlet to a channel

Figure TS14C–4 Toe of the slope lined with stone to control erosion
Sizing techniques

There are many techniques for sizing stone, and each method has advantages and disadvantages. Many techniques were derived under specific conditions and developed for particular applications. While this list is not complete and the description is not exhaustive, several commonly used methods are presented. The designer should review the applicability of a technique before choosing it to size stone for a particular project. Following is a brief description of several rock sizing techniques.

Isbash method

The Isbash formula (Isbash 1936) was developed for the construction of dams by depositing rocks into moving water. The Isbash curve should only be used for quick estimates or for comparisons. A coefficient is provided to target high- and low-turbulence flow conditions, so this method can be a high- or low-energy application. The equation is:

$$V_c = C \times \left(2 \times g \times \frac{\gamma_s - \gamma_w}{\gamma_w} \right)^{0.50} \times \left(D_{50}\right)^{0.50}$$  

(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
- $\gamma_s$ = stone density (lb/ft^3)
- $\gamma_w$ = 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

![Graph showing Isbash method](image-url)
Figure TS14C–6  Graphical solution for Isbash technique

Basic equations:

\[
V = C \left[ 2g \left( \frac{\gamma_s - \gamma_w}{\gamma_w} \right) \right]^{\frac{1}{2}} (D_{50})^{\frac{1}{2}}
\]

\[
D_{50} = \left( \frac{8W_{50}}{\pi \gamma_s} \right)^{\frac{1}{3}}
\]

where:

- \( V \) = Velocity, ft/s
- \( \gamma_s \) = Specific stone weight, lb/ft\(^3\)
- \( \gamma_w \) = 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\(^2\)
Figure TS14C–6  Graphical solution for Isbash technique—Continued

(b)

Basic equations:

\[
V_c = C \left[ 2g \left( \frac{\gamma_s - \gamma_w}{\gamma_w} \right) \right]^{\frac{1}{2}} (D_{50})^{\frac{1}{2}}
\]

\[
D_{50} = \left( \frac{8W_{50}}{\pi\gamma_s} \right)^{\frac{1}{3}}
\]

where:

- \( V_c \) = Velocity, ft/s
- \( \gamma_s \) = Specific stone weight, lb/ft³
- \( \gamma_w \) = Specific weight of water, 62.5 lb/ft³
- \( W_{50} \) = 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/s²

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.

\[ \tau_c = \gamma RS_e \]  
\[ \tau_c = 4D_{50} \]

therefore,

\[ D_{50} = \frac{\gamma RS_e}{4} \]

\( \tau_c \) = critical tractive stress  
\( \gamma \) = 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:

\[ D_{30} = FS\times C_s \times C_v \times C_T \times d \times \left[ \frac{\gamma_w}{\gamma_s - \gamma_w} \right]^{0.5} \times \frac{V}{\sqrt{K_i \times g \times d}} \]

(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)  
\( \gamma_w \) = specific weight of water (lb/ft\(^3\))  
\( \gamma_s \) = specific weight of stone (lb/ft\(^3\))  
\( V \) = local velocity; if unknown use 1.5 \( V_{average} \)  
\( g \) = 32.2 ft/s\(^2\)  
\( K_i \) = side slope correction as computed below

\[ K_i = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \]

(eq. TS14C–6)

where:

\( \theta \) = angle of rock from the horizontal  
\( \phi \) = angle of repose (typically 40º)

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:

\[ C_v = 1.283 - 0.2 \log \left( \frac{R}{W} \right) \]

(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:

\[ 1.8D_{15} < D_{60} < 4.6D_{15} \]

(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:

\[
D_{50} = 0.0122V^{2.06}
\]  

(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:

\[
D_{50} = 0.01V^{3.44}
\]  

(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 \( D_{50} \):

\[
D_{50} = 0.00116\frac{V^3}{\sqrt{d}}
\]  

(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:

\[
D_{90} = \frac{1.95S^{0.55}(Cq)^{2/5}}{g^{1/5}}
\]  

(eq. TS14C–12)

where:

- \( D_{90} \) = 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:

\[
D = \frac{18(\text{depth})S_f}{(SG - 1)}
\]  

(eq. TS14C–13)

where:

- \( D \) = minimum stone size
- \( \text{depth} \) = 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:

\[ D_{50} = \left( q_{\text{design}} \right)^{0.56} \times S^{0.41} \times 5.23 \]  
(eq. TS14C–14)

where:

- \( D_{50} \) = stone size in inches; \( m \) percent finer by weight
- \( q_{\text{design}} \) = unit discharge (ft\(^3\)/s/ft)
- \( S \) = channel slope (ft/ft) and \( S \) between 0.02 and 0.20 ft/ft

\[ \left( q_{\text{design}} \right) = \left( q_{\text{failure}} \right) / 0.74 = 1.35q_{\text{failure}} \]  
(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 \( S < 0.1 \)

\[ D_{50} = 12 \left( 1.923qS^{1.5} \right)^{0.529} \]  
(eq. TS14C–16)

0.10 < \( S < 0.40 \)

\[ D_{50} = 12 \left( 0.233qS^{0.5} \right)^{0.529} \]  
(eq. TS14C–17)

**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:

\[ W = \frac{0.00002}{\left( G_S - 1 \right)^{3/2}} \times VM \times V^2 \times G_S \times \frac{1}{\sin^3 \left( r - a \right)} \]  
(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:

\[ D_{75} = \frac{3.5}{C \times K} \times \gamma_w \times D \times S_f \]  
(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)
- \( \gamma_w \) = 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
# 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  
**Design:** Jim Ville  
**County:** Woodbury  
**Date:** 3/30/2006  
**Checked by:**

## Input Channel Geometry

<table>
<thead>
<tr>
<th>Channel</th>
<th>Inlet Channel</th>
<th>Chute</th>
<th>Outlet Channel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bw</td>
<td>20.0 ft.</td>
<td></td>
<td>20.0 ft.</td>
</tr>
<tr>
<td>Side slopes</td>
<td>4.0 (m:1)</td>
<td>Factor of safety</td>
<td>1.20 ($F_s$)</td>
</tr>
<tr>
<td>n-value</td>
<td>0.035</td>
<td>Side slopes</td>
<td>4.0 (m:1)</td>
</tr>
<tr>
<td>Bed slope</td>
<td>0.0060 ft./ft.</td>
<td>Bed slope</td>
<td>0.200 ft./ft.</td>
</tr>
<tr>
<td>Base flow</td>
<td>0.5 ft.</td>
<td>Outlet apron depth, $d$</td>
<td>1.0 ft.</td>
</tr>
</tbody>
</table>

## Design Storm Data (Table 2. NHCP, NRCS Grade Stabilization Structure No. 410)

- **Drainage area:** 450.0 acres  
- **Rainfall:** 0 - 3 in.  
- **Outlet:** 99.0 ft.  
- **Chute capacity:** 50-year  
- **Minimum capacity (based on a 5-year, 24-hour storm with a 3 - 5 inch rainfall):** $Q_{5y} = 330.0$ cfs, $Q_{low} = 75.0$ cfs  

### Notes:
- The total required capacity is routed through the chute (principal spillway) or in combination with an auxiliary spillway.

## Profile and Cross Section (Output)

### Energy Grade Line

<table>
<thead>
<tr>
<th>Height</th>
<th>Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_p$</td>
<td>$0.38 ft. (0.18 ft.)$</td>
</tr>
<tr>
<td>$H_w$</td>
<td>$2.67 ft.$</td>
</tr>
<tr>
<td>$H_{ch}$</td>
<td>$2.3 ft. (0.93 ft.)$</td>
</tr>
<tr>
<td>$H_{ch}$</td>
<td>$2.51 ft.$</td>
</tr>
<tr>
<td>$Y_{ch}$</td>
<td>$1.8 ft. (0.72 ft.)$</td>
</tr>
<tr>
<td>$Z_1$</td>
<td>$1.07 ft. (0.44 ft.)$</td>
</tr>
<tr>
<td>$H_{ch}$</td>
<td>$5 ft.$</td>
</tr>
<tr>
<td>$H_{ch}$</td>
<td>$2.04 ft. (0.86 ft.)$</td>
</tr>
<tr>
<td>$T_w + d$</td>
<td>$3.04 ft. - Tw o.k. (1.86 ft.) - Tw o.k.$</td>
</tr>
<tr>
<td>$2(D_{ch}) + F_d$</td>
<td>$15(D_{ch})(F_d)$</td>
</tr>
<tr>
<td>$D_o$</td>
<td>$5 ft.$</td>
</tr>
<tr>
<td>$D_{ch}$</td>
<td>$15(D_{ch})(F_d)$</td>
</tr>
<tr>
<td>$D_{ch}$</td>
<td>$2.04 ft. (0.86 ft.)$</td>
</tr>
<tr>
<td>$T_w + d$</td>
<td>$3.04 ft. - Tw o.k.$</td>
</tr>
<tr>
<td>$2(D_{ch}) + F_d$</td>
<td>$15(D_{ch})(F_d)$</td>
</tr>
<tr>
<td>$Z_2$</td>
<td>$2.76 ft. (1.09 ft.)$</td>
</tr>
</tbody>
</table>

### Notes:
1. **Output given as High Flow (Low Flow) values.**
2. Tailwater depth plus $d$ must be at or above the hydraulic jump height for the chute to function.
3. Critical depth occurs $2y_c - 4y_c$, upstream of crest.
4. Use min. 8 oz. non-woven geotextile under rock.

### Profile Along Centerline of Chute

- **$q_s$:** 13.65 cfs/ft.  
- **$F_0$:** 1.20  
- **$z_i$:** 1.07 ft.  
- **n-value:** 0.054  
- **$D_{ch}$** (30 in. - 50% round / 50% angular)  

### Typical Cross Section

- **Use $H_p$ along chute but not less than $z_2$.**

---

*(210–VI–NEH, August 2007)*
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:

\[ \text{Step 1} \quad \text{Enter figure TS14C–8 with energy slope (channel grade) and flow depth.} \]

\[ \text{Step 2} \quad \text{Track right to side slope.} \]

\[ \text{Step 3} \quad \text{Track up to ratio of curve radius to water surface width.} \]

\[ \text{Step 4} \quad \text{Track right to estimate required riprap size.} \]

---

**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.

<table>
<thead>
<tr>
<th>(Rc/W) *</th>
<th>C</th>
<th>Side slope</th>
<th>K</th>
</tr>
</thead>
<tbody>
<tr>
<td>4–6</td>
<td>0.6</td>
<td>1-1/2H:1V</td>
<td>.52</td>
</tr>
<tr>
<td>6–9</td>
<td>0.75</td>
<td>1-3/4H:1V</td>
<td>.63</td>
</tr>
<tr>
<td>9–12</td>
<td>0.90</td>
<td>2H:1V</td>
<td>.72</td>
</tr>
<tr>
<td>straight channel</td>
<td>1.0</td>
<td>2-1/2H:1V</td>
<td>.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3H:1V</td>
<td>.87</td>
</tr>
</tbody>
</table>

*\(Rc = \text{Curve radius}\)*

*\(W = \text{Water surface width}\)*

*\(S = \text{Energy slope or channel grade}\)*

*\(w = 62.4\)*
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

<table>
<thead>
<tr>
<th>Technique</th>
<th>High or low energy</th>
<th>Slopes</th>
<th>Typical application(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isbash</td>
<td>Both</td>
<td>Not specified</td>
<td>Rock revetment, stilling basins, river closures</td>
</tr>
<tr>
<td>10S Report</td>
<td>Both</td>
<td>&lt;10%</td>
<td>Quick assessments for stable stone requirements</td>
</tr>
<tr>
<td>Maynord</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Rock revetment, bank protection, stone toe</td>
</tr>
<tr>
<td>Abt and Johnson</td>
<td>High</td>
<td>2% to 20%</td>
<td>Overtopping, grade protection</td>
</tr>
<tr>
<td>ARS – rock chute</td>
<td>High</td>
<td>2% to 40%</td>
<td>Overtopping, rock chutes, grade protection</td>
</tr>
<tr>
<td>USBR</td>
<td>High</td>
<td>Not specified</td>
<td>Riprap below a stilling basin</td>
</tr>
<tr>
<td>USGS Blodgett</td>
<td>Both</td>
<td>Not specified</td>
<td>Riprap stability</td>
</tr>
<tr>
<td>USACE Steep Slope Riprap</td>
<td>High</td>
<td>2% to 20%</td>
<td>Rock chutes, grade protection</td>
</tr>
<tr>
<td>USACE Habitat Boulder</td>
<td>High</td>
<td>Not specified</td>
<td>Instream boulders for habitat enhancement</td>
</tr>
<tr>
<td>CALTRANS RSP</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Rock revetment, bank protection, stone toe</td>
</tr>
<tr>
<td>Lane’s (FWS)</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Stone bank protection, stream barbs with adjustments</td>
</tr>
</tbody>
</table>

### Table TS14C–3  Summary of techniques

<table>
<thead>
<tr>
<th>Technique</th>
<th>High or low energy</th>
<th>Slopes</th>
<th>Typical application(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isbash</td>
<td>Both</td>
<td>Not specified</td>
<td>Rock revetment, stilling basins, river closures</td>
</tr>
<tr>
<td>10S Report</td>
<td>Both</td>
<td>&lt;10%</td>
<td>Quick assessments for stable stone requirements</td>
</tr>
<tr>
<td>Maynord</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Rock revetment, bank protection, stone toe</td>
</tr>
<tr>
<td>Abt and Johnson</td>
<td>High</td>
<td>2% to 20%</td>
<td>Overtopping, grade protection</td>
</tr>
<tr>
<td>ARS – rock chute</td>
<td>High</td>
<td>2% to 40%</td>
<td>Overtopping, rock chutes, grade protection</td>
</tr>
<tr>
<td>USBR</td>
<td>High</td>
<td>Not specified</td>
<td>Riprap below a stilling basin</td>
</tr>
<tr>
<td>USGS Blodgett</td>
<td>Both</td>
<td>Not specified</td>
<td>Riprap stability</td>
</tr>
<tr>
<td>USACE Steep Slope Riprap</td>
<td>High</td>
<td>2% to 20%</td>
<td>Rock chutes, grade protection</td>
</tr>
<tr>
<td>USACE Habitat Boulder</td>
<td>High</td>
<td>Not specified</td>
<td>Instream boulders for habitat enhancement</td>
</tr>
<tr>
<td>CALTRANS RSP</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Rock revetment, bank protection, stone toe</td>
</tr>
<tr>
<td>Lane’s (FWS)</td>
<td>Low</td>
<td>&lt;2%</td>
<td>Stone bank protection, stream barbs with adjustments</td>
</tr>
</tbody>
</table>
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.

\[ G_s = 2.65 \text{ or } \gamma_s = 165.36 \text{ lb/ft}^3 \]
Width = 40 ft
\[ n = 0.045 \]
Slope = 0.01 ft/ft
Depth = 6 ft

Solution: Solve relevant hydraulic parameters

\[ \text{Vel} = 9.1 \text{ ft/s} \]
\[ Q = 2,200 \text{ ft}^3/\text{s} \]
\[ Y_{\text{crit}} = 4.54 \text{ ft} \]

The riprap size determined from several methods is:

<table>
<thead>
<tr>
<th>Method</th>
<th>( D_{50} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isbash</td>
<td>6.5 in</td>
</tr>
<tr>
<td>Maynord</td>
<td>4.6 in, 5.5 in</td>
</tr>
<tr>
<td>Lane's (FWS)</td>
<td>15 in, 12.7 in</td>
</tr>
<tr>
<td>Abt and Johnson</td>
<td>8.1 in</td>
</tr>
<tr>
<td>ARS rock chute</td>
<td>3.6 in</td>
</tr>
</tbody>
</table>

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.

\[ G_s = 2.65 \text{ or } \gamma_s = 165.36 \text{ lb/ft}^3 \]
Width = 40 ft
\[ n = 0.045 \]
Slope = 0.06 ft/ft
Depth = 3.5 ft

Solution: Solve relevant hydraulic parameters

\[ \text{Vel} = 16.7 \text{ ft/s} \]
\[ Q = 2,340 \text{ ft}^3/\text{s} \]
\[ Y_{\text{crit}} = 4.7 \text{ ft} \]

The riprap size determined from several methods is:

<table>
<thead>
<tr>
<th>Method</th>
<th>( D_{50} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Isbash</td>
<td>1.6 ft</td>
</tr>
<tr>
<td>Maynord</td>
<td>1.6 ft, 1.9 ft</td>
</tr>
<tr>
<td>Lane's (FWS)</td>
<td>3.7 ft, 3.2 ft</td>
</tr>
<tr>
<td>Abt and Johnson</td>
<td>1.3 ft</td>
</tr>
<tr>
<td>ARS rock chute</td>
<td>1.1 ft</td>
</tr>
</tbody>
</table>

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.
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.

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.
## Purpose

TS14D–1

## Introduction

TS14D–1

## Materials

TS14D–1

- Geotextile
- Geogrid
- Geonet
- Geocell
- Rolled erosion control products

## Functions

TS14D–3

- Drainage
- Filtration
- Separation
- Reinforcement
- Erosion control

## Applications

TS14D–4

- Geotextile filter
- Reinforced slopes
- Mechanically stabilized earth (MSE) walls
- Earth retaining structures
- Erosion protection

## Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS14D–1</td>
<td>Example problem soil gradation</td>
<td>9</td>
</tr>
<tr>
<td>TS14D–2</td>
<td>Requirements for woven geotextiles</td>
<td>9</td>
</tr>
<tr>
<td>TS14D–3</td>
<td>Requirements for nonwoven geotextiles</td>
<td>9</td>
</tr>
<tr>
<td>TS14D–4</td>
<td>Default geotextile class and design class for the subsurface drainage, permanent erosion control, separation, and stabilization applications</td>
<td>10</td>
</tr>
<tr>
<td>TS14D–5</td>
<td>Summary of design solutions for example problem</td>
<td>10</td>
</tr>
<tr>
<td>Figures</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>------------------</td>
<td>-------------------------------------------------------</td>
<td>--------</td>
</tr>
<tr>
<td>Figure TS14D–1</td>
<td>Monofilament woven geotextile</td>
<td>TS14D–1</td>
</tr>
<tr>
<td>Figure TS14D–2</td>
<td>Needle-punched nonwoven geotextile</td>
<td>TS14D–1</td>
</tr>
<tr>
<td>Figure TS14D–3</td>
<td>Heat-bonded nonwoven geotextile</td>
<td>TS14D–1</td>
</tr>
<tr>
<td>Figure TS14D–4</td>
<td>Biaxial geogrid</td>
<td>TS14D–2</td>
</tr>
<tr>
<td>Figure TS14D–5</td>
<td>Uniaxial geogrid</td>
<td>TS14D–2</td>
</tr>
<tr>
<td>Figure TS14D–6</td>
<td>Geonet</td>
<td>TS14D–2</td>
</tr>
<tr>
<td>Figure TS14D–7</td>
<td>Geocell</td>
<td>TS14D–2</td>
</tr>
<tr>
<td>Figure TS14D–8</td>
<td>Erosion control blanket</td>
<td>TS14D–3</td>
</tr>
<tr>
<td>Figure TS14D–9</td>
<td>Turf reinforcement mats</td>
<td>TS14D–3</td>
</tr>
<tr>
<td>Figure TS14D–10</td>
<td>Separation and/or filtration beneath erosion protection material</td>
<td>TS14D–5</td>
</tr>
<tr>
<td>Figure TS14D–11</td>
<td>Reinforcement of steep streambank slope</td>
<td>TS14D–5</td>
</tr>
<tr>
<td>Figure TS14D–12</td>
<td>Mechanically stabilized earth wall</td>
<td>TS14D–5</td>
</tr>
<tr>
<td>Figure TS14D–13</td>
<td>Earth retaining structure</td>
<td>TS14D–6</td>
</tr>
<tr>
<td>Figure TS14D–14</td>
<td>Erosion protection</td>
<td>TS14D–6</td>
</tr>
<tr>
<td>Figure TS14D–15</td>
<td>Reinforced soil slope with rock face</td>
<td>TS14D–7</td>
</tr>
<tr>
<td>Figure TS14D–16</td>
<td>MSE wall under construction</td>
<td>TS14D–7</td>
</tr>
<tr>
<td>Figure TS14D–17</td>
<td>Geocell earth retaining structure</td>
<td>TS14D–8</td>
</tr>
</tbody>
</table>
Purpose

A variety of geosynthetic materials may be used for various functions and applications in stream restoration and stabilization projects. This technical supplement is intended to provide field staffs an understanding of some of the basic principles and applications of geosynthetic materials.

Introduction

A geosynthetic material is defined as a planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as part of a manmade project, structure, or system (American Society for Testing and Materials International (ASTM) D4439). Geosynthetics used in stream restoration and stabilization include geotextiles, geogrids, geonets, geocells, and rolled erosion control products.

Materials

Selection of a geosynthetic material appropriate for a project requires an understanding of the different types that are available, as well as their performance criteria and range of applications. Five types of geosynthetic materials are described here.

Geotextile

A geotextile is defined as a permeable geosynthetic comprised solely of textiles (ASTM D4439). A geotextile may be woven or nonwoven and may be composed of monofilament yarns or monofilament plastic (fig. TS14D–1). A nonwoven geotextile may be needle-punched (fig. TS14D–2), heat bonded (fig. TS14D–3), or resin bonded.

Geogrid

A geogrid is defined as a geosynthetic formed by a regular network of integrally connected elements with apertures greater than a fourth inch to allow interlocking with surrounding soil, rock, earth, and other sur-
grounding materials to function primarily as reinforcement (ASTM D4439). A geogrid may be biaxial (fig. TS14D–4), or uniaxial (fig. TS14D–5).

**Geonet**

A geonet is defined as a geosynthetic consisting of integrally connected parallel sets of ribs overlying similar sets at various angles for planar drainage of liquids and gases (ASTM D4439). A typical geonet is shown in figure TS14D–6.

**Geocell**

A geocell is defined as a product composed of polyethylene strips, connected by a series of offset, full-depth welds to form a three-dimensional honeycomb system (ASTM D4439). Geocells (fig. TS14D–7) are available in a variety of depths from 4 inches to 9 inches.

---

**Figure TS14D–4**  Biaxial geogrid

**Figure TS14D–5**  Uniaxial geogrid

**Figure TS14D–6**  Geonet

**Figure TS14D–7**  Geocell
Rolled erosion control products

Rolled erosion control products consist of both erosion control blankets (ECB) used for temporary erosion protection and turf reinforcement mats (TRM) for more permanent erosion protection. An ECB is shown in figure TS14D–8 and two TRMs are shown in figure TS14D–9.

Functions

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

Drainage

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

Filtration

Filtration is the most common use of geosynthetics in stream restoration and stabilization projects. A geosynthetic used for filtration is intended to retain the particles of the filtered (protected) soil, while allowing a fluid to flow through the plane of the fabric. Woven and nonwoven needle-punched geotextiles are used for this function.

Separation

The objective of geosynthetics used for the separation function is to prevent two different materials from mixing and compromising the performance of one or both of these materials. Woven and nonwoven geotextiles may both be used for this function. Heat-bonded nonwoven geotextiles offer an economical geosynthetic separator.
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

![Diagram](image1)

**Figure TS14D–11** Reinforcement of steep streambank slope

![Diagram](image2)

**Figure TS14D–12** Mechanically stabilized earth wall

![Diagram](image3)
**Figure TS14D–13**  Earth retaining structure

![Diagram of an earth retaining structure with vegetation, granular material, and geocell filled with granular material.]

**Figure TS14D–14**  Erosion protection

![Diagram of erosion protection with vegetation, rock riprap, turf reinforcement mat, and staples.]

Vegetation

Granular material

Geocell filled with granular material

Rock riprap

Turf reinforcement mat

Staples
Mechanically stabilized earth walls

A mechanically stabilized earth (MSE) wall must be designed for external, internal, and local stability. The external stability analyses include sliding, overturning, bearing capacity, and settlement. The internal stability analyses include geosynthetic pullout, tensile strength of the geosynthetic, and internal sliding. Local stability analyses include an analysis of the facing connection to the geosynthetic and bulging of the facing. A photograph of an MSE wall that is under construction is shown in figure TS14D–16.

Guidance for MSE wall design is provided by FHWA (FHWA 2001c), Designing with Geosynthetics (Koerner 1998), or National Concrete Masonry Association (NCMA) 1997). A computer program entitled MSEW may also aid in the design.
The global stability of the MSE wall, retained soil, and soil foundation must be analyzed, just as reinforced slope design.

**Earth retaining structures**

An earth retaining structure must be designed for external stability and internal stability. The external stability analyses include an analysis of sliding, overturning, bearing capacity, and settlement. In a geocell wall, the internal stability analysis includes an analysis of the friction between each geocell layer.

The global stability of earth retaining structures must be analyzed just as reinforced slopes. A photograph of a geocell earth retaining structure is shown in figure TS14D–17.

**Erosion protection**

Selection and installation of an ECB or TRM is a function of the hydraulic characteristics of the site, streambank slopes, and expected lift of the product. ECBs are used for temporary erosion protection until adequate vegetation can be established. TRMs are considered permanent erosion protection and are designed to reinforce the soil surface and root system of the vegetation.

---

**Example: Geotextile filter calculations**

**Problem:** A streambank stabilization project will include a rock chute constructed on soil with the gradation in table TS14D–1.

Using design criteria for a *woven geotextile* in Design Note 24, Guide for the Use of Geotextiles (USDA SCS 1991) (table TS14D–2), determine the geotextile filter requirements.

**Solution:** Soil contains 15 to 50 percent finer than the # 200 sieve, so:

- Apparent opening size (AOS) $D_{85}$
- Percent open area (POA) $>4$
- Permeability, $K_{\text{geotextile}} > 10 K_{\text{soil}}$

$D_{85} = 0.150 \text{ mm}$, so AOS $\leq 0.15 \text{ mm}$ (#100 sieve)

Percent open area (POA) $>4$

The soil contains 25 percent finer than the #200 sieve with an estimated $K_{\text{soil}} = 0.004 \text{ cm/s}$, so $K_{\text{geotextile}} > 0.04 \text{ cm/s}$


- $AOS \leq 0.425 \text{ mm}$ (#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 $= 0.2 \text{ s}^{-1}$
- $AOS \leq 0.25 \text{ mm}$ (#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

<table>
<thead>
<tr>
<th>Size</th>
<th>% Finer</th>
</tr>
</thead>
<tbody>
<tr>
<td>#40 (0.42 mm)</td>
<td>100</td>
</tr>
<tr>
<td>#60 (0.25 mm)</td>
<td>98</td>
</tr>
<tr>
<td>#140 (0.105 mm)</td>
<td>60</td>
</tr>
<tr>
<td>#200 (0.074 mm)</td>
<td>25</td>
</tr>
<tr>
<td>0.005 mm</td>
<td>4</td>
</tr>
</tbody>
</table>

### Table TS14D–2 Requirements for woven geotextiles

<table>
<thead>
<tr>
<th>Property</th>
<th>Test method</th>
<th>Class I</th>
<th>Class II and III</th>
<th>Class IV ³/ ³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength (lb) ² ¹</td>
<td>ASTM D4632 Grab test</td>
<td>200 min. in any principal direction</td>
<td>120 min. in any principal direction</td>
<td>180 min. in any principal direction</td>
</tr>
<tr>
<td>Bursting strength (lb/in²) ² ¹</td>
<td>ASTM D3786 Diaphragm tester</td>
<td>400 min.</td>
<td>300 min.</td>
<td>NA</td>
</tr>
<tr>
<td>Elongation at failure (%)</td>
<td>ASTM D4632 Grab test</td>
<td>&lt;50</td>
<td>&lt;50</td>
<td>&lt;50</td>
</tr>
<tr>
<td>Puncture (lb)</td>
<td>ASTM D4833</td>
<td>90 min.</td>
<td>60 min.</td>
<td>60 min.</td>
</tr>
<tr>
<td>Ultraviolet light (percent residual tensile strength)</td>
<td>ASTM D4355 150 hr exposure</td>
<td>70 min.</td>
<td>70 min.</td>
<td>70 min.</td>
</tr>
<tr>
<td>Apparent opening size (AOS)</td>
<td>ASTM D4751</td>
<td>As specified, or a min. #100 ²</td>
<td>As specified, or a min. #100 ²</td>
<td>As specified, or a min. #100 ²</td>
</tr>
<tr>
<td>Percent open area (%)</td>
<td>CWO–02215–86</td>
<td>4.0 min.</td>
<td>4.0 min.</td>
<td>1.0 min.</td>
</tr>
<tr>
<td>Permittivity (1/s)</td>
<td>ASTM D4491</td>
<td>0.10 min.</td>
<td>0.10 min.</td>
<td>0.10 min.</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Property</th>
<th>Test method</th>
<th>Class I</th>
<th>Class II</th>
<th>Class III</th>
<th>Class IV ³/ ³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength (lb) ² ¹</td>
<td>ASTM D4632 Grab test</td>
<td>180 min.</td>
<td>120 min.</td>
<td>90 min.</td>
<td>115 min.</td>
</tr>
<tr>
<td>Bursting strength (lb/in²) ² ¹</td>
<td>ASTM D3786 Diaphragm tester</td>
<td>320 min.</td>
<td>210 min.</td>
<td>180 min.</td>
<td>WA</td>
</tr>
<tr>
<td>Elongation at failure (%)</td>
<td>ASTM D4632 Grab test</td>
<td>&gt;50</td>
<td>&gt;50</td>
<td>&gt;50</td>
<td>&gt;50</td>
</tr>
<tr>
<td>Puncture (lb)</td>
<td>ASTM D4833</td>
<td>80 min.</td>
<td>60 min.</td>
<td>40 min.</td>
<td>40 min.</td>
</tr>
<tr>
<td>Ultraviolet light (percent residual tensile strength)</td>
<td>ASTM D4355 150 hr exposure</td>
<td>70 min.</td>
<td>70 min.</td>
<td>70 min.</td>
<td>70 min.</td>
</tr>
<tr>
<td>Apparent opening size (AOS)</td>
<td>ASTM D4751</td>
<td>As specified, max. #40 ²</td>
<td>As specified, max. #40 ²</td>
<td>As specified, max. #40 ²</td>
<td>As specified, max. #40 ²</td>
</tr>
<tr>
<td>Permittivity (1/s)</td>
<td>ASTM D4491</td>
<td>0.70 min.</td>
<td>0.70 min.</td>
<td>0.70 min.</td>
<td>0.70 min.</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Application class</th>
<th>Default class</th>
<th>Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>Subsurface drainage</td>
<td>Class 2</td>
<td>Class 3</td>
</tr>
<tr>
<td>Permanent erosion control</td>
<td>Class 2 for woven</td>
<td>Class 2</td>
</tr>
<tr>
<td></td>
<td>monofilaments</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Class 1 for all others</td>
<td></td>
</tr>
<tr>
<td>Separation</td>
<td>Class 2</td>
<td>Class 3</td>
</tr>
<tr>
<td>Stabilization</td>
<td>Class 1</td>
<td>Class 2 or 3</td>
</tr>
</tbody>
</table>

Table TS14D–5  Summary of design solutions for example problem

<table>
<thead>
<tr>
<th>Property</th>
<th>NRCS DN 24 – Woven</th>
<th>NRCS DN 24 – Nonwoven</th>
<th>AASHTO M–288</th>
</tr>
</thead>
<tbody>
<tr>
<td>AOS</td>
<td>0.155 mm</td>
<td>0.425 mm</td>
<td>0.25 mm</td>
</tr>
<tr>
<td></td>
<td>(#100 Sieve)</td>
<td>(#40 Sieve)</td>
<td>(#60 Sieve)</td>
</tr>
<tr>
<td>Permeability</td>
<td>0.04 cm/s</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>Permittivity</td>
<td>NA</td>
<td>NA</td>
<td>0.2 s⁻¹</td>
</tr>
</tbody>
</table>
Use and Design of Soil Anchors

(210–VI–NEH, August 2007)
Advisory Note

Techniques and approaches contained in this handbook are not all-inclusive, nor universally applicable. Designing stream restorations requires appropriate training and experience, especially to identify conditions where various approaches, tools, and techniques are most applicable, as well as their limitations for design. Note also that product names are included only to show type and availability and do not constitute endorsement for their specific use.
## Technical Supplement 14E

### The Use and Design of Soil Anchors

<table>
<thead>
<tr>
<th>Contents</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose</td>
<td>TS14E–1</td>
</tr>
<tr>
<td>Introduction</td>
<td>TS14E–1</td>
</tr>
<tr>
<td>Calculating the forces acting on a LWM structure</td>
<td>TS14E–1</td>
</tr>
<tr>
<td>Soil anchor types</td>
<td></td>
</tr>
<tr>
<td>Driven anchors</td>
<td>TS14E–1</td>
</tr>
<tr>
<td>Screw-in anchors</td>
<td>TS14E–4</td>
</tr>
<tr>
<td>Cabling (wire rope) to boulders or bedrock</td>
<td>TS14E–5</td>
</tr>
<tr>
<td>Wire rope</td>
<td>TS14E–5</td>
</tr>
<tr>
<td>Connectors and tensioning</td>
<td>TS14E–6</td>
</tr>
<tr>
<td>Method for calculating forces acting on a LWM structure</td>
<td>TS14E–7</td>
</tr>
<tr>
<td>Drag force</td>
<td>TS14E–7</td>
</tr>
<tr>
<td>Buoyancy force</td>
<td>TS14E–8</td>
</tr>
<tr>
<td>Example calculation</td>
<td>TS14E–8</td>
</tr>
<tr>
<td>Anchor manufacturer data</td>
<td>TS14E–9</td>
</tr>
<tr>
<td>Specific gravity of wood</td>
<td>TS14E–12</td>
</tr>
<tr>
<td>Conclusion</td>
<td>TS14E–12</td>
</tr>
</tbody>
</table>
Tables

Table TS14E–1 Duckbill® specifications TS14E–9

Table TS14E–2 Soil classification TS14E–10

Table TS14E–3 Manta Ray® ultimate holding capacity TS14E–10

Table TS14E–4 Stingray® ultimate holding capacity TS14E–11

Table TS14E–5 Specific gravity values for some commercially important woods grown in the United States TS14E–13

Figures

Figure TS14E–1 Platipus Stealth® anchor TS14E–2

Figure TS14E–2 Drive rod being inserted into Duckbill® anchor prior to installation TS14E–2

Figure TS14E–3 Post driver being used to install soil anchors TS14E–3

Figure TS14E–4 Driving soil anchor with a 30-lb jackhammer TS14E–3

Figure TS14E–5 Hi-Lift® jack TS14E–4

Figure TS14E–6 Hi-Lift® jack being used to load-lock a Duckbill® anchor TS14E–4

Figure TS14E–7 Screw-in anchor TS14E–4

Figure TS14E–8 Boulders serve dual purpose: to stabilize the toe and secure brush revetment TS14E–5

Figure TS14E–9 Eyebolt anchored in boulder with epoxy TS14E–5

Figure TS14E–10 Wire rope anchored in boulder with epoxy TS14E–5

Figure TS14E–11 Ratcheting-type cable clamp—allows tension to be applied between two cables TS14E–6

Figure TS14E–12 Gripple® wire rope grip and tensioning tool being used to tension down a brush spur TS14E–6

Figure TS14E–13 Debris lodged against rootwads TS14E–7

Figure TS14E–14 Example problem, planview TS14E–9
Purpose

The success of a soil bioengineering project that uses large woody material (LWM) structures depends on proper anchoring design. This technical supplement presents three of the more common anchoring methods used: driven soil anchors, screw-in soil anchors, and cabling to boulders or bedrock. Also covered is a method for estimating the pullout capacity required of the anchor and another method for connecting of the anchor to a LWM structure. Selecting the anchoring method and sizing the anchor require information about the expected streamflows and soil characteristics. The required pullout capacity per anchor can be estimated from the streamflow information, and the anchor type and method can be selected from the soil information. Once the anchor has been installed, the LWM structure must be firmly held in place by the anchor. This requires applying tension to the wire rope that connects the anchor to the LWM structure. An effective method for achieving this is described.

Introduction

Anchoring is required to hold LWM structures and brush revetments against streambanks and streambeds. During high flows, material placed in the streambed or on the streambank will be subject to drag forces, buoyancy forces, and, possibly, impact forces. Proper anchoring is required to resist these forces and firmly hold the structure in place. Since impact forces are difficult to predict, the factor of safety used in the calculations is assumed to be sufficient to account for impact forces.

Failure of an anchoring system on a LWM structure could cause damage to the embankment and downstream structures. Undersized anchors and loose connections contribute to the majority of failures. A proper connection is required between an anchor and a LWM structure to firmly hold the structure in place.

Calculating the forces acting on a LWM structure

Before the anchor method and anchor size can be selected, an estimation of the needed pullout capacity per anchor must be calculated. A simplified method for estimating the forces acting on a LWM structure is provided in this technical supplement. This approach uses project-specific information about soil characteristics, stream velocity at a flow that submerges the structure, and debris load. Much of the information used in this approach will be difficult to obtain or approximate. As a result, a factor of safety is used to account for the lack of data. The designer must consider the impact of an anchor failure when determining the factor of safety.

Soil anchor types

Soil anchors are an effective way to anchor LWM structures. The two types described here are driven anchors and screw anchors. Both anchors are available in different configurations and sizes, with various holding capacities. The anchors can be installed manually in certain soil conditions and have pullout capacities of up to 5,000 pounds. Much greater pullout capacities can be obtained with both anchor types, but a mechanical means of installation is required. Estimates of pullout capacities for anchors in different classes of soils are available in tables published by the manufacturers.

Driven anchors

Driven-type soil anchors are available in different configurations and sizes. They are pushed vertically into the soil to the recommended depth and then are locked into a horizontal position.

Information and supply can be obtained from vineyard, landscape, and utility supply companies. Some of the more common trade names are:

- Duckbill®
- Platipus Stealth®
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
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
drive bar, to pry the anchor into the locked position. In soils that are harder to penetrate, a Hi-Lift® jack (fig. TS14E–5) can be used to lock the anchor. Figure TS14E–6 shows a Hi-Lift® jack being used to lock a Duckbill® anchor. Larger anchors require an anchor-locking base with a hydraulic ram system that is made specifically for locking the anchor into position and proof-testing the holding capacity of the anchor. The proof-tested holding capacity should be compared with design values to assure adequate anchorage.

**Screw-in anchors**

Screw-in soil anchors (fig. TS14E–7) are another option for anchoring LWM. Screw-in anchors can be used in loose to medium dense, fine to coarse sand and sandy gravels, and firm to very stiff silts and clays. They can have a single helical disk or multiple disks that, when rotated, will auger itself into the soil. These anchors are available in multiple sizes. Smaller screw-in soil anchors, like the ones that can be purchased at a farm supply store, can be installed in silty clay soils without rocks by manually screwing them in, using a cross bar. These manually installed anchors can achieve pull-out capacities of up to 4,000 pounds. Larger screw-in soil anchors require heavy equipment for installation. Drilling attachments for tractors, backhoes, and boom trucks are commonly used to install large screw-in soil anchors. The anchor must be installed parallel with the direction of pull.
Cabling (wire rope) to boulders or bedrock

Boulders or bedrock, when available, can be used to anchor structures. Boulders may exist onsite or be incorporated into the design for bank toe stabilizing. Whichever the case, it is possible to strategically place the boulders so that they can be used as anchors. Figure TS14E–8 shows boulders being used for bank toe stabilization, as well as anchors for a brush revetment.

Cabling to boulders or bedrock requires drilling a hole in the rock and using epoxy to secure an eyebolt (fig. TS14E–9) or the ends of wire rope (fig. TS14E–10) into the rock. Follow the epoxy manufacturers specifications for hole diameter, depth, and time required for the epoxy to set. The hole must be free of dust and debris, and the eyebolt or wire rope must be free of any dust, dirt, and lubrication to allow a proper bond.

Wire rope

Wire rope is typically used to attach LWM structures to the anchors. It comes in a range of sizes, constructions, and materials. The characteristics that are generally most essential in soil bioengineering projects are the breaking strength, flexibility, and corrosion-resistance. Wire rope must be flexible enough to make a tight wrap around a LWM structure. In soil bioengineering projects, the wire rope will be exposed to the weather with portions of the wire rope at times submerged in water or buried in the soil. Using galvanized or stainless steel wire rope can provide added corrosion resistance.

Figure TS14E–8  Boulders serve dual purpose: to stabilize the toe and secure brush revetment

Figure TS14E–9  Eyebolt anchored in boulder with epoxy

Figure TS14E–10  Wire rope anchored in boulder with epoxy
Once the total force per anchor \( F_{\text{Anchor}} \) has been calculated, the breaking strength required of the wire rope can be obtained by multiplying the force per anchor by a minimum factor of safety (FS) of 2 to determine the minimum breaking strength required from the wire rope. A factor of safety of 2 is used to account for corrosion and wear over time, as well as impact forces. A minimum of 1/8-inch-diameter wire rope should be used. However, the designer should not necessarily select the thickest cable available because too thick of a cable may not be flexible enough to secure tightly for some applications.

**Connectors and tensioning**

Proper tensioning of the wire rope to the LWM is essential. Many problems can result from a loose connection between the anchor and LWM such as oscillating forces resulting in the anchor pulling out, increased erosion of the bank or streambed, or the LWM breaking loose from the wire rope.

An effective method for tensioning wire rope around LWM uses ratcheting type cable clamps (fig. TS14E–11) and a special tensioning tool (fig. TS14E–12). Two pieces of wire rope connected to Duckbill® anchors are connected together with a Gripple® wire rope grip. One such type is manufactured by Gripple®. The ratcheting type cable clamp is used for connecting two pieces of wire rope or a single piece that is looped back through the wire rope grip. The wire rope grip allows the wire rope to pass through the wire rope grip in one direction only. With the use of the tensioning tool the wire rope is pulled through the wire rope grip, applying tension to the wire rope. Wire rope ratcheting type cable clamps can be obtained in different sizes with working load limits up to 4,000 pounds. Wire clamps can be added if the design indicates that the wire rope grip capacity will be exceeded or as an added precaution after the wire rope has been tensioned.
**Method for calculating forces acting on a LWM structure**

This technical supplement provides a simplified method for calculating forces on a LWM structure. A more detailed approach is provided in technical supplement 14J of this handbook. The resulting calculation can be used to select the appropriate soil anchor. It should be noted that this simplification may not be applicable in all situations, and a more involved analysis may be necessary.

The forces acting on a LWM structure include the drag force from the water flow, a buoyancy force, and impact forces from debris. Since impact forces are less predictable, the equation includes potential impact forces by increasing the debris or increasing the factor of safety.

**Drag force**

The following empirical equation, based on Stoke's Law (Stokes 1851), can be used to estimate the drag force ($F_d$) in pounds on the LWM structure:

$$F_d = 0.95(A)(v^2)(D)(K) \quad (eq. \ TS14E-1)$$

where:

- $A$ = surface area ($\text{ft}^2$) 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.
- $v$ = expected stream velocity ($\text{ft/s}$)
- $D$ = estimated 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.

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.

$F_d = 0.95A(v^2)(D)(K)$

The debris increase factor is generally between 1 and 1.5. Estimating this factor requires engineering judgment from observation of the debris load on existing stationary objects within the stream and potential for the addition of debris from the streambanks and tributaries. Take notice of the debris load on bridge columns and/or abutments, fallen trees that extend into the stream or have lodged within the stream, or any other stationary object within the stream that could catch debris. Figure TS14E–13 shows an example of a stream with potential for additional debris load on a LWM structure. From these observations and considering the potential damage if an anchor failed, estimate the percent increase in surface area that is perpendicular to the flow, and use that as the debris increase factor.

$K$ = permeability coefficient

This factor is figured by estimating the percentage of voids in the surface area that are not anticipated to plug/fill with debris. Use conservative judgments when making this estimate. If method 1 is used to calculate the surface area, the permeability coefficient ($K$) is 1.0.

**Figure TS14E–13**   Debris lodged against rootwads
Buoyancy force

The buoyancy force \( F_b \) can be estimated by:

\[
F_b = V \left( \gamma_w - \gamma_{(LWM)} \right) \quad \text{(eq. TS14E–2)}
\]

where:

- \( V \) = volume (ft\(^3\)) of LWM submerged
- \( \gamma_w \) = 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
- \( \omega \) = (1+ moisture content, as a decimal)

The unit density (\( \gamma \)) of the LWM can be calculated from the specific gravity of the wood (\( G_S \)) and the expected moisture content (\( \omega \)). 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 (\( F/\text{Anchor} \)) is calculated using the following equation:

\[
\frac{F}{\text{Anchor}} = \frac{FS (F_d + F_b)}{A_n} \quad \text{(eq. TS14E–3)}
\]

where:

- \( FS \) = factor of safety
- \( A_n \) = number of anchors

The factor of safety used depends on the potential damages that would occur if an anchor were to fail, as well as the level of confidence in the design assumptions such as potential impact loads from debris and extent of soils information available. Factors of safety for LWM structures typically range from 1.5 (when limited impact loads are expected and soil characteristics are known) to 3.0 (when impact loads are unknown, and/or the soil characteristics are unknown).

Example calculation

**Problem:**
Brush spurs made from willow brush are designed for a soil bioengineering project to deflect the water flow away from a streambank toe and facilitate the accumulation of sediment between the spurs. The spurs are 20 feet long, 3 feet high, and 3 feet wide and are placed at a 45-degree angle from the streambank, pointing in the upstream direction (fig. TS14E–14). The stream velocity for flow above the brush spur was measured at 4 feet per second. Estimate the force per anchor during a storm event that completely submerges the brush spurs.

**Solution:**
Estimate the drag force acting on the structure using equation TS14E–1.

Solve for the surface area (A) perpendicular to the flow:

\[
A = \text{length} \times \text{height}
\]

\[
\sin \theta = \frac{\text{opp}}{\text{hyp}} = \frac{b}{c}
\]

\[
b = 0.707 \times 20 \text{ ft} = 14.1 \text{ ft}
\]

\[A = 14.1 \text{ ft} \times 3 \text{ ft (height, given)} = 42.4 \text{ ft}^2 \]

\[v = \text{given as 4 ft/s} \]

\[D = 1.25 \text{ (After observation of debris build up on stationary objects within the stream and its tributaries)} \]

\[F_d = 0.95 \left( 42.4 \text{ ft}^2 \right) \left( 4 \text{ ft/s} \right) ^2 \left( 1.25 \right) \times 1 = 802 \text{ lb} \]

\[K = 1 \text{ (brush spur is well compacted, making it fairly impervious)} \]
Estimate the buoyancy force acting on the structure using equation TS14E–2.

First estimate the density (γ) of the wood using the following equation.
\[
\gamma_{(LWM)} = G_S (\gamma_W)(\omega)
\]
\[
\omega = 12\% = 1.12 \quad (12\% \text{ is the typical air dried moisture content})
\]
\[
G_S = 0.39 \quad \text{(table TS14E–5)}
\]
\[
\gamma_W = 62.4 \text{ lb/ft}^3
\]
\[
\gamma_{(LWM)} = 0.39(62.4 \text{ lb/ft}^3)(1.12) = 27.3 \text{ lb/ft}^3
\]

Estimate the volume (V) by assuming 60 percent of the brush spur is wood:
\[
V = 20 \text{ ft}(3 \text{ ft})(3 \text{ ft})(0.60) = 108 \text{ ft}^3
\]

So, the buoyancy force (\(F_b\)) is:
\[
F_b = 108(62.4 - 27.3) = 3,791 \text{ lb}
\]

Estimate the total force per anchor (\(F_t/\text{anchor}\)) using equation TS14E–3.
\[
\frac{F_t}{\text{anchor}} = \frac{FS(F_d + F_b)}{A_n}
\]
\[
FS = 1.5
\]
\[
A_n = 6 \text{ anchors}
\]
\[
F_t/\text{anchor} = 1.5(802 \text{ lb} + 3,791 \text{ lb}) + 6 = 1,148 \text{ lb/anchor}
\]

Anchor manufacturer data

The anchors in table TS14E–1 (Foresight Products 2001) are rated in an average soil condition (class 5). Soil classes are listed in table TS14E–2 (A.B. Chance Company). A torque probe can be used for quick soil classification in the field. A core sampler could also be used to obtain \textit{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.

<table>
<thead>
<tr>
<th>Duckbill model no.</th>
<th>Rated capacity (lb)</th>
<th>Drive rod diameter (in)</th>
<th>Normal depth of installation</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>300</td>
<td>1/4</td>
<td>20 in</td>
</tr>
<tr>
<td>68</td>
<td>1,100</td>
<td>1/2</td>
<td>2 1/2 ft</td>
</tr>
<tr>
<td>88</td>
<td>3,000</td>
<td>3/4</td>
<td>3 1/2 ft</td>
</tr>
<tr>
<td>138</td>
<td>5,000</td>
<td>1</td>
<td>5 ft</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Class</th>
<th>Description</th>
<th>Probe value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Solid bedrock</td>
<td>—</td>
</tr>
<tr>
<td>2</td>
<td>Dense clay; compact gravel dense fine sand; laminated rock; slate; schist; sand stone</td>
<td>Over 600 in/lb</td>
</tr>
<tr>
<td>3</td>
<td>Shale; broken bedrock; hardpan; compacted gravel clay mixture</td>
<td>500–600 in/lb</td>
</tr>
<tr>
<td>4</td>
<td>Gravel; compacted gravel and sand; claypan</td>
<td>400–500 in/lb</td>
</tr>
<tr>
<td>5</td>
<td>Medium-firm clay; loose standard gravel; compacted coarse sand</td>
<td>300–400 in/lb</td>
</tr>
<tr>
<td>6</td>
<td>Medium-firm clay; loose course sand; clayey silt; compact fine sand</td>
<td>200–300 in/lb</td>
</tr>
<tr>
<td>7</td>
<td>Fill; loose fine sand; wet clays; silt</td>
<td>100–200 in/lb</td>
</tr>
<tr>
<td>8</td>
<td>Swamp; marsh; saturated silt; humus</td>
<td>Under 100 in/lb</td>
</tr>
</tbody>
</table>

Table TS14E–3  Manta Ray® ultimate holding capacity

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Very dense and/or cemented sands; coarse gravel and cobbles</td>
<td>60+</td>
<td>10</td>
<td>16</td>
<td>20</td>
<td>28–40</td>
<td>40</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(1,3)</td>
<td>(1,3)</td>
<td>(1,3)</td>
<td>(1,3,4)</td>
<td>(1,3)</td>
<td>(1,3,5)</td>
<td>(1,3,5)</td>
</tr>
<tr>
<td>Dense, fine, compacted sands; very hard silts or clays</td>
<td>45–60</td>
<td>6–10</td>
<td>9–16</td>
<td>17–20</td>
<td>21–28</td>
<td>36–40</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(2,3,4)</td>
<td>(2,3,4)</td>
<td>(2,3,4)</td>
<td>(2,4)</td>
<td>(1,3,4)</td>
<td>(1,3)</td>
<td>(1,3,5)</td>
</tr>
<tr>
<td>Dense clays, sands and gravels; hard silts and clays</td>
<td>35–50</td>
<td>4–6</td>
<td>6–9</td>
<td>12–18</td>
<td>15–22</td>
<td>24–36</td>
<td>32–40</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(2,4)</td>
<td>(2,3,4)</td>
<td>(1,3)</td>
</tr>
<tr>
<td>Medium-dense, sandy gravel, stiff to hard silts and clays</td>
<td>24–40</td>
<td>3–4</td>
<td>4.5–5.5</td>
<td>9–14</td>
<td>12–18</td>
<td>18–20</td>
<td>24–34</td>
<td>32–40</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(2,4)</td>
<td>(2,4)</td>
<td>(2,3,4)</td>
</tr>
<tr>
<td>Medium-dense, coarse sand and sandy gravel, stiff to very stiff silts and clays</td>
<td>14–25</td>
<td>2–3</td>
<td>3.5–4.5</td>
<td>7–9</td>
<td>9–12</td>
<td>15–20</td>
<td>18–24</td>
<td>24–32</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(2,4)</td>
</tr>
<tr>
<td>Loose to medium-dense, fine to coarse sand; firm to stiff clays and silts</td>
<td>7–14</td>
<td>1.5–2.5</td>
<td>2.5–4</td>
<td>5–8</td>
<td>7–10</td>
<td>10–15</td>
<td>14–18</td>
<td>20–24</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
</tr>
<tr>
<td>Loose fine sand; alluvium; soft clays; fine, saturated, silty sand</td>
<td>4–8</td>
<td>0.9–1.5</td>
<td>1.5–2.5</td>
<td>3–5</td>
<td>5–8</td>
<td>8–12</td>
<td>9–14</td>
<td>13–20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(4,6)</td>
<td>(4,6)</td>
<td>(4,6)</td>
<td>(4,6)</td>
<td>(4,6)</td>
<td>(4,6)</td>
<td>(4,6)</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Description</th>
<th>Blow count (N)</th>
<th>SR–1 ultimate = 100 kips</th>
<th>SR–2 ultimate = 100 kips</th>
<th>SR–3 ultimate = 100 kips</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very dense and/or cemented sands; coarse gravel and cobbles</td>
<td>60+</td>
<td>65–89</td>
<td>89–100</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>(1,3)</td>
<td>(1,3)</td>
<td>(1,3,5)</td>
<td></td>
</tr>
<tr>
<td>Dense, fine, compacted sand; very hard silts and clays</td>
<td>45–60</td>
<td>58–65</td>
<td>79–89</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>(2, 4)</td>
<td>(2, 4)</td>
<td>(2, 3)</td>
<td></td>
</tr>
<tr>
<td>Dense clays, sands and gravel; hard silts and clays</td>
<td>35–50</td>
<td>39–58</td>
<td>62–79</td>
<td>85–100</td>
</tr>
<tr>
<td></td>
<td>(4)</td>
<td>(2, 4)</td>
<td>(2, 3, 4)</td>
<td></td>
</tr>
<tr>
<td>Medium dense sandy gravel; very stiff to hard silts and clays</td>
<td>24–40</td>
<td>29–41</td>
<td>46–66</td>
<td>63–90</td>
</tr>
<tr>
<td></td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td></td>
</tr>
<tr>
<td>Medium dense coarse sand and sandy gravel; stiff to very stiff silts and clays</td>
<td>14–25</td>
<td>24–32</td>
<td>31–48</td>
<td>48–63</td>
</tr>
<tr>
<td></td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td></td>
</tr>
<tr>
<td>Loose to medium-dense, fine to coarse sand; firm to stiff clays and silts</td>
<td>7–14</td>
<td>16–24</td>
<td>27–36</td>
<td>37–48</td>
</tr>
<tr>
<td></td>
<td>(4)</td>
<td>(4)</td>
<td>(4)</td>
<td></td>
</tr>
<tr>
<td>Loose, fine sand; alluvium; soft-firm clays; varied clays; fill</td>
<td>4–8</td>
<td>13–19</td>
<td>19–28</td>
<td>24–37</td>
</tr>
<tr>
<td></td>
<td>(4,6)</td>
<td>(4,6)</td>
<td>(4)</td>
<td></td>
</tr>
</tbody>
</table>

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

Tabs 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

<table>
<thead>
<tr>
<th>Common species</th>
<th>Moisture content</th>
<th>Specific gravity¹</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Alder, red</strong></td>
<td>Green</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.41</td>
</tr>
<tr>
<td><strong>Ash</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.49</td>
</tr>
<tr>
<td>Blue</td>
<td>Green</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.58</td>
</tr>
<tr>
<td>Green</td>
<td>Green</td>
<td>0.53</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.56</td>
</tr>
<tr>
<td>Oregon</td>
<td>Green</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.55</td>
</tr>
<tr>
<td>White</td>
<td>Green</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.6</td>
</tr>
<tr>
<td><strong>Aspen</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bigtooth</td>
<td>Green</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.39</td>
</tr>
<tr>
<td>Quaking</td>
<td>Green</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.38</td>
</tr>
<tr>
<td><strong>Basswood</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>American</td>
<td>Green</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.37</td>
</tr>
<tr>
<td><strong>Beech</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>American</td>
<td>Green</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.64</td>
</tr>
<tr>
<td><strong>Birch</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Paper</td>
<td>Green</td>
<td>0.48</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.55</td>
</tr>
<tr>
<td>Sweet</td>
<td>Green</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.65</td>
</tr>
<tr>
<td>Yellow</td>
<td>Green</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.62</td>
</tr>
<tr>
<td>Butternut</td>
<td>Green</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.38</td>
</tr>
<tr>
<td><strong>Cherry</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.50</td>
</tr>
<tr>
<td><strong>Chestnut</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>American</td>
<td>Green</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.43</td>
</tr>
<tr>
<td><strong>Cottonwood</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Balsam, Poplar</td>
<td>Green</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.34</td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.35</td>
</tr>
<tr>
<td>Eastern</td>
<td>Green</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.40</td>
</tr>
<tr>
<td><strong>Elm</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>American</td>
<td>Green</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.50</td>
</tr>
<tr>
<td>Rock</td>
<td>Green</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.63</td>
</tr>
<tr>
<td>Slippery</td>
<td>Green</td>
<td>0.48</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.53</td>
</tr>
<tr>
<td><strong>Hackberry</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Green</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.53</td>
</tr>
<tr>
<td><strong>Hickory, Pecan</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bitternut</td>
<td>Green</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.66</td>
</tr>
<tr>
<td>Nutmeg</td>
<td>Green</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.60</td>
</tr>
<tr>
<td>Pecan</td>
<td>Green</td>
<td>0.60</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.66</td>
</tr>
<tr>
<td>Water</td>
<td>Green</td>
<td>0.61</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.62</td>
</tr>
</tbody>
</table>

¹ Specific gravity values are given as the ratio of the mass of the wood to the mass of an equal volume of water.
<table>
<thead>
<tr>
<th>Common species</th>
<th>Moisture content</th>
<th>Specific gravity&lt;sup&gt;1&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hickory, True</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mockernut</td>
<td>Green</td>
<td>0.64</td>
</tr>
<tr>
<td>12%</td>
<td>0.72</td>
<td></td>
</tr>
<tr>
<td>Pignut</td>
<td>Green</td>
<td>0.66</td>
</tr>
<tr>
<td>12%</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>Shagbark</td>
<td>Green</td>
<td>0.64</td>
</tr>
<tr>
<td>12%</td>
<td>0.72</td>
<td></td>
</tr>
<tr>
<td>Shellbark</td>
<td>Green</td>
<td>0.62</td>
</tr>
<tr>
<td>12%</td>
<td>0.69</td>
<td></td>
</tr>
<tr>
<td>Honeylocust</td>
<td>Green</td>
<td>0.60</td>
</tr>
<tr>
<td>12%</td>
<td>—</td>
<td></td>
</tr>
<tr>
<td>Locust</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.66</td>
</tr>
<tr>
<td>12%</td>
<td>0.69</td>
<td></td>
</tr>
<tr>
<td>Magnolia</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cucumbertree</td>
<td>Green</td>
<td>0.44</td>
</tr>
<tr>
<td>12%</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>Southern</td>
<td>Green</td>
<td>0.46</td>
</tr>
<tr>
<td>12%</td>
<td>0.50</td>
<td></td>
</tr>
<tr>
<td>Maple</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bigleaf</td>
<td>Green</td>
<td>0.44</td>
</tr>
<tr>
<td>12%</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.52</td>
</tr>
<tr>
<td>12%</td>
<td>0.57</td>
<td></td>
</tr>
<tr>
<td>Red</td>
<td>Green</td>
<td>0.49</td>
</tr>
<tr>
<td>12%</td>
<td>0.54</td>
<td></td>
</tr>
<tr>
<td>Silver</td>
<td>Green</td>
<td>0.44</td>
</tr>
<tr>
<td>12%</td>
<td>0.47</td>
<td></td>
</tr>
<tr>
<td>Sugar</td>
<td>Green</td>
<td>0.56</td>
</tr>
<tr>
<td>12%</td>
<td>0.63</td>
<td></td>
</tr>
<tr>
<td>Oak, Red</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.56</td>
</tr>
<tr>
<td>12%</td>
<td>0.61</td>
<td></td>
</tr>
<tr>
<td>Cherrybark</td>
<td>Green</td>
<td>0.61</td>
</tr>
<tr>
<td>12%</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>Laurel</td>
<td>Green</td>
<td>0.56</td>
</tr>
<tr>
<td>12%</td>
<td>0.63</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Common species</th>
<th>Moisture content</th>
<th>Specific gravity&lt;sup&gt;1&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>Northern Red</td>
<td>Green</td>
<td>0.56</td>
</tr>
<tr>
<td>Pin</td>
<td>Green</td>
<td>0.63</td>
</tr>
<tr>
<td>Scarlet</td>
<td>Green</td>
<td>0.60</td>
</tr>
<tr>
<td>Southern Red</td>
<td>Green</td>
<td>0.52</td>
</tr>
<tr>
<td>Water</td>
<td>Green</td>
<td>0.56</td>
</tr>
<tr>
<td>Willow</td>
<td>Green</td>
<td>0.56</td>
</tr>
<tr>
<td>12%</td>
<td>0.63</td>
<td></td>
</tr>
<tr>
<td>Oak, White</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bur</td>
<td>Green</td>
<td>0.58</td>
</tr>
<tr>
<td>12%</td>
<td>0.64</td>
<td></td>
</tr>
<tr>
<td>Chestnut</td>
<td>Green</td>
<td>0.57</td>
</tr>
<tr>
<td>12%</td>
<td>0.66</td>
<td></td>
</tr>
<tr>
<td>Live</td>
<td>Green</td>
<td>0.80</td>
</tr>
<tr>
<td>12%</td>
<td>0.88</td>
<td></td>
</tr>
<tr>
<td>Overcup</td>
<td>Green</td>
<td>0.57</td>
</tr>
<tr>
<td>12%</td>
<td>0.63</td>
<td></td>
</tr>
<tr>
<td>Post</td>
<td>Green</td>
<td>0.60</td>
</tr>
<tr>
<td>12%</td>
<td>0.67</td>
<td></td>
</tr>
<tr>
<td>Swamp Chestnut</td>
<td>Green</td>
<td>0.60</td>
</tr>
<tr>
<td>12%</td>
<td>0.67</td>
<td></td>
</tr>
<tr>
<td>Swamp White</td>
<td>Green</td>
<td>0.64</td>
</tr>
<tr>
<td>12%</td>
<td>0.72</td>
<td></td>
</tr>
<tr>
<td>White</td>
<td>Green</td>
<td>0.60</td>
</tr>
<tr>
<td>12%</td>
<td>0.68</td>
<td></td>
</tr>
<tr>
<td>Sweetgum</td>
<td>Green</td>
<td>0.46</td>
</tr>
<tr>
<td>12%</td>
<td>0.52</td>
<td></td>
</tr>
<tr>
<td>Sycamore</td>
<td></td>
<td></td>
</tr>
<tr>
<td>American</td>
<td>Green</td>
<td>0.46</td>
</tr>
<tr>
<td>12%</td>
<td>0.49</td>
<td></td>
</tr>
<tr>
<td>Tanoak</td>
<td>Green</td>
<td>0.58</td>
</tr>
<tr>
<td>12%</td>
<td>—</td>
<td></td>
</tr>
</tbody>
</table>

<sup>1</sup> Values are approximate.
Table TS14E–5

<table>
<thead>
<tr>
<th>Common species</th>
<th>Moisture content</th>
<th>Specific gravity¹</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Tupelo</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.50</td>
</tr>
<tr>
<td>Water</td>
<td>Green</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.50</td>
</tr>
<tr>
<td><strong>Walnut</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.55</td>
</tr>
<tr>
<td><strong>Willow</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Black</td>
<td>Green</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.39</td>
</tr>
</tbody>
</table>

---

**Softwood**

<table>
<thead>
<tr>
<th>Common species</th>
<th>Moisture content</th>
<th>Specific gravity¹</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baldcypress</td>
<td>Green</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.46</td>
</tr>
<tr>
<td><strong>Cedar</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Atlantic White</td>
<td>Green</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.32</td>
</tr>
<tr>
<td>Eastern redceder</td>
<td>Green</td>
<td>0.44</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.47</td>
</tr>
<tr>
<td>Incense</td>
<td>Green</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.37</td>
</tr>
<tr>
<td>Northern White</td>
<td>Green</td>
<td>0.29</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.31</td>
</tr>
<tr>
<td>Port-Orford</td>
<td>Green</td>
<td>0.39</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.43</td>
</tr>
<tr>
<td>Western redceder</td>
<td>Green</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.32</td>
</tr>
<tr>
<td>Yellow</td>
<td>Green</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.44</td>
</tr>
<tr>
<td><strong>Douglas-fir²</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coast</td>
<td>Green</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.48</td>
</tr>
<tr>
<td>Interior West</td>
<td>Green</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.50</td>
</tr>
<tr>
<td><strong>Interior North</strong></td>
<td>Green</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.48</td>
</tr>
<tr>
<td><strong>Interior South</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Green</td>
<td>0.43</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.46</td>
</tr>
<tr>
<td><strong>Fir</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Balsam</td>
<td>Green</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.35</td>
</tr>
<tr>
<td>California Red</td>
<td>Green</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.38</td>
</tr>
<tr>
<td>Grand</td>
<td>Green</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.37</td>
</tr>
<tr>
<td>Noble</td>
<td>Green</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.39</td>
</tr>
<tr>
<td>Pacific Silver</td>
<td>Green</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.43</td>
</tr>
<tr>
<td>Subalpine</td>
<td>Green</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.32</td>
</tr>
<tr>
<td>White</td>
<td>Green</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.39</td>
</tr>
<tr>
<td><strong>Hemlock</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Eastern</td>
<td>Green</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.40</td>
</tr>
</tbody>
</table>

¹ Specific gravity values are based on a moisture content of 12%.
² Douglas-fir is a type of coniferous softwood.
<table>
<thead>
<tr>
<th>Common species</th>
<th>Moisture content</th>
<th>Specific gravity 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mountain</td>
<td>Green</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.45</td>
</tr>
<tr>
<td>Western</td>
<td>Green</td>
<td>0.42</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.45</td>
</tr>
<tr>
<td>Virginia</td>
<td>Green</td>
<td>0.45</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.48</td>
</tr>
<tr>
<td>Western White</td>
<td>Green</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.38</td>
</tr>
<tr>
<td>Larch</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Western</td>
<td>Green</td>
<td>0.48</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.52</td>
</tr>
<tr>
<td>Redwood</td>
<td>Old-Growth</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>Young-Growth</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.35</td>
</tr>
<tr>
<td>Pine</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Eastern White</td>
<td>Green</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.35</td>
</tr>
<tr>
<td>Jack</td>
<td>Green</td>
<td>0.40</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.43</td>
</tr>
<tr>
<td>Loblolly</td>
<td>Green</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.51</td>
</tr>
<tr>
<td>Lodgepole</td>
<td>Green</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.41</td>
</tr>
<tr>
<td>Longleaf</td>
<td>Green</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.59</td>
</tr>
<tr>
<td>Pitch</td>
<td>Green</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.52</td>
</tr>
<tr>
<td>Pond</td>
<td>Green</td>
<td>0.51</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.56</td>
</tr>
<tr>
<td>Ponderosa</td>
<td>Green</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.40</td>
</tr>
<tr>
<td>Red</td>
<td>Green</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.46</td>
</tr>
<tr>
<td>Sand</td>
<td>Green</td>
<td>0.46</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.48</td>
</tr>
<tr>
<td>Shortleaf</td>
<td>Green</td>
<td>0.47</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.51</td>
</tr>
<tr>
<td>Slash</td>
<td>Green</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.59</td>
</tr>
<tr>
<td>Spruce</td>
<td>Green</td>
<td>0.41</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.44</td>
</tr>
<tr>
<td>Sugar</td>
<td>Green</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.36</td>
</tr>
<tr>
<td>Sitka</td>
<td>Green</td>
<td>0.37</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.40</td>
</tr>
<tr>
<td>White</td>
<td>Green</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.36</td>
</tr>
<tr>
<td>Tamarack</td>
<td>Green</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>12%</td>
<td>0.53</td>
</tr>
</tbody>
</table>
Technical Supplement 14F

Pile Foundations

(210–VI–NEH, August 2007)
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.

(210–VI–NEH, August 2007)
Contents

Purpose  TS14F–1
Introduction  TS14F–1
Bearing capacity  TS14F–1
Lateral load capacity  TS14F–4

Tables

Table TS14F–1  Prescriptive values for cohesionless soils  TS14F–2
Table TS14F–2  Prescriptive values for cohesive soils  TS14F–3

Figures

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  TS14F–5
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  TS14F–5
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  TS14F–6
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  TS14F–6
Pile Foundations

Purpose

Piles are used to transfer foundation forces through relatively weak soil to stronger strata to minimize settlement. The most likely applications for pile foundations in stream restoration and stabilization projects are as support for bank stabilization structures (retaining wall) and anchors for large woody material (LWM). Piles may be used to support ancillary structures such as culverts, structural channels, bridges, and pumping stations. This technical supplement addresses the analyses required to design pile foundations.

Introduction

Foundation structures may be classified into two categories: shallow and deep. There is no specific rule that defines when a particular structure is considered to be shallow or deep. In general, shallow structures are constructed fairly close to ground surface and are usually constructed upwards from the bottom surface of an excavation.

Traditionally, deep foundations refer to piles that are driven into the ground. However, piles are sometimes set into holes that are prebored or drilled into the ground. A hole bored into the ground and filled with concrete is called a drilled shaft. Usually, reinforcing steel is placed into the drilled hole just prior to placement of the concrete. Other terms for drilled shafts are: drilled piers, drilled caissons, cast-in-place piles, cast-in-drilled-hole piles, and augered piles. Another type of drilled shaft foundation is an auger-cast pile.

Piles are typically installed using specialized pile driving equipment. The motive force that drives the pile into the ground is applied by a pile driving hammer, which is attached to the top of a pile. A crane is used to support the pile driving equipment and handle the individual piles.

Bearing capacity

The allowable bearing or axial capacity of a driven pile may be determined from the following equation:

\[ Q_{\text{Allowable}} = \frac{Q_{\text{total}}}{\text{factor of safety}} \]  
\[ Q_{\text{total}} = Q_{\text{point}} + Q_{\text{friction}} - W_{\text{pile}} \]

where:
- \( Q_{\text{total}} \) = ultimate capacity of pile
- \( Q_{\text{point}} \) = end-bearing capacity of pile
- \( Q_{\text{friction}} \) = capacity due to friction along length of pile
- \( W_{\text{pile}} \) = 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:

\[ Q_{\text{point}} = \gamma' D N_q' A_{\text{point}} \]  
\[ (\text{eq. TS14F–3}) \]

where:
\[ \gamma' = \text{effective unit weight of soil} \]
\[ D = \text{embedded depth of pile} \]
\[ A_{\text{point}} = \text{area of pile tip} \]

\[ N_q' = e^{\tan \phi' \tan ^2 (45^\circ + \frac{\phi'}{2})} \]
\[ (\text{eq. TS14F–3a}) \]

where:
\[ \phi' = \tan^{-1}(0.67 \tan \phi) \]
\[ \phi = \text{angle of internal friction} \]

\[ Q_{\text{friction}} = \tau'_{\text{average}} \Sigma \Omega K_{\text{shape}} D \]
\[ (\text{eq. TS14F–4}) \]

where:
\[ \tau'_{\text{average}} = \gamma' z K_o \tan \phi' \]
\[ (\text{eq. TS14F–4a}) \]

I-beam piles are considered to have a square perimeter with side lengths equivalent to their respective depth and width dimensions.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Angle of internal friction (°)</th>
<th>Angle of friction between soil and pile (°)</th>
<th>Bearing capacity coefficient ( N_q )</th>
<th>Maximum allowable capacity, ( Q )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Friction (tons/ft²)</td>
</tr>
<tr>
<td>Clean sand</td>
<td>35</td>
<td>30</td>
<td>11</td>
<td>1.00</td>
</tr>
<tr>
<td>Silty sand</td>
<td>30</td>
<td>25</td>
<td>7</td>
<td>0.85</td>
</tr>
<tr>
<td>Sandy silt</td>
<td>25</td>
<td>20</td>
<td>5</td>
<td>0.70</td>
</tr>
<tr>
<td>Silt</td>
<td>20</td>
<td>15</td>
<td>4</td>
<td>0.50</td>
</tr>
</tbody>
</table>
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.

\[ Q_{\text{point}} = A_{\text{point}} (7.4c + \gamma D) \]  

(eq. TS14F–5)

where:
- \( A_{\text{point}} \) = area of pile tip 
- \( c \) = shear strength of cohesive soil at pile tip depth 
- \( \gamma \) = effective unit weight of soil 
- \( D \) = depth of pile

\[ Q_{\text{friction}} = c_{\text{average}} \sum_0 K_{\text{shape}} D \]  

(eq. TS14F–6)

where:
- \( c_{\text{average}} \) = average effective shear stress of soil, along with a given length, or segment of the pile 
- \( \sum_0 \) = perimeter of pile 
- \( K_{\text{shape}} \) = pile shape factor 
- \( K_{\text{shape}} \) = 1.000 for round perimeter 
- \( K_{\text{shape}} = 0.785 \) for square perimeter

I-beam piles are considered to have a square perimeter with side lengths equivalent to their respective depth and width dimensions.

- \( K_{\text{shape}} = 0.95 \) for octagon perimeter
- \( K_{\text{shape}} = 0.84 \) for hexagon perimeter
- \( K_{\text{shape}} = 0.60 \) for triangular perimeter

\( D \) = depth of pile or length of pile segment

---

<table>
<thead>
<tr>
<th>Clay</th>
<th>Shear strength</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Normally consolidated</strong></td>
<td>[ c_z = 0.5\gamma'z(1 - K_o) ]</td>
</tr>
<tr>
<td><strong>Underconsolidated</strong></td>
<td>[ c_z = 0.125\gamma'z(1 - K_o) ]</td>
</tr>
<tr>
<td><strong>Overconsolidated by erosion</strong></td>
<td>[ c_z = 600 \text{ lb/ft}^2 + 0.5\gamma'z(1 - K_o) ]</td>
</tr>
<tr>
<td><strong>Overconsolidated by desiccation</strong></td>
<td>[ c_z = 2,000 \text{ lb/ft}^2 \text{ for } z = 0 \text{ ft to 20 ft} ]</td>
</tr>
<tr>
<td></td>
<td>[ c_z = 1,200 \text{ lb/ft}^2 \text{ for } z = 20 \text{ ft to 60 ft} ]</td>
</tr>
<tr>
<td></td>
<td>[ c_z = 3,000 \text{ lb/ft}^2 \text{ for } z = 60 \text{ ft to 160 ft} ]</td>
</tr>
</tbody>
</table>

For highly fissured clays, use the following:

- \( c' = 0 \)
- \( \phi' = 5' \text{ to } 10' \)
- \( \tau'_{\text{average}} = \gamma'zK_o\tan\phi' \)

Notes:
- For piles loaded in axial compression, \( K_o = 0.7 \)
- For piles loaded in axial tension, \( K_o = 0.5 \)
- Effective shear strength, \( c' = 0.67c \)
- \( Z \) = depth along pile from ground surface (ft) \( Z = 0 \) at ground surface
Lateral load capacity

An approximate lateral capacity of a short, rigid pile driven into a cohesive soil can be determined from the following equation (figs. TS14F–1 and TS14F–2):

\[
P_{\text{Allowable}} = \frac{P_{\text{Ultimate}}}{\text{factor of safety}} \quad \text{(eq. TS14F–7)}
\]

where:
- \( P_{\text{Allowable}} \) = allowable lateral load applied to exposed portion of pile
- \( \text{factor of safety} = 3 \)

\[
P_{\text{Ultimate}} = \sigma B \left[ \left( \frac{D}{2} + H \right)^2 + D^2 \right] - 2 \left( \frac{D}{2} + H \right) \quad \text{(eq. TS14F–8)}
\]

where:
- \( \sigma \) = 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:

\[
D = \frac{P_{\text{Ultimate}}}{\frac{2 \sigma B}{\gamma' B K_p}} \left( \frac{H + D}{H} \right) \quad \text{(eq. TS14F–9)}
\]

This equation is solved iteratively until both values for \( D \) are equivalent.

\[
P_{\text{Ultimate}} = \frac{\gamma' B K_p D^2}{2(H + D)} \quad \text{(eq. TS14F–12)}
\]

where:
- \( \gamma' \) = effective unit weight of soil
- \( B \) = width or diameter of pile
- \( K_p \) = passive pressure coefficient

\[
K_p = \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) \quad \text{(eq. TS14F–13)}
\]

\( \phi \) = 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.

For most stream restoration and stabilization applications, a driven pile may be considered to be rigid when:

\[
\frac{H}{B} \leq 12 \quad \text{(eq. TS14F–10)}
\]

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).

\[
P_{\text{Allowable}} = \frac{P_{\text{Ultimate}}}{\text{factor of safety}} \quad \text{(eq. TS14F–11)}
\]

where:
- \( P_{\text{Allowable}} \) = allowable lateral load applied to exposed portion of pile
- \( \text{factor of safety} = 3 \)
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.

\[ \sigma_D = 3\gamma' D K_p \]

\[ \sigma_z = 3\gamma' Z K_p \]
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.
<table>
<thead>
<tr>
<th>Contents</th>
<th>TS14G–1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose</td>
<td>TS14G–1</td>
</tr>
<tr>
<td>Introduction</td>
<td>TS14G–1</td>
</tr>
<tr>
<td>Grade control hydraulics</td>
<td>TS14G–1</td>
</tr>
<tr>
<td>Types of grade control structures</td>
<td>TS14G–4</td>
</tr>
<tr>
<td>Loose rock structures</td>
<td>TS14G–5</td>
</tr>
<tr>
<td>Rock sizing for loose rock structures</td>
<td>TS14G–7</td>
</tr>
<tr>
<td>Rock sizing summary</td>
<td>TS14G–8</td>
</tr>
<tr>
<td>Local scour protection for loose rock structures</td>
<td>TS14G–9</td>
</tr>
<tr>
<td>Channel linings</td>
<td>TS14G–9</td>
</tr>
<tr>
<td>Loose rock structures with water cutoff</td>
<td>TS14G–10</td>
</tr>
<tr>
<td>Structures with preformed scour holes and water cutoff</td>
<td>TS14G–10</td>
</tr>
<tr>
<td>Rigid drop structures</td>
<td>TS14G–13</td>
</tr>
<tr>
<td>Alternative construction materials</td>
<td>TS14G–14</td>
</tr>
<tr>
<td>General design considerations for siting grade controlstructures</td>
<td>TS14G–17</td>
</tr>
<tr>
<td>Hydraulic and sediment transport considerations</td>
<td>TS14G–17</td>
</tr>
<tr>
<td>Geotechnical considerations</td>
<td>TS14G–18</td>
</tr>
<tr>
<td>Flood control impacts</td>
<td>TS14G–19</td>
</tr>
<tr>
<td>Environmental considerations</td>
<td>TS14G–19</td>
</tr>
<tr>
<td>Existing structures</td>
<td>TS14G–20</td>
</tr>
<tr>
<td>Local site conditions</td>
<td>TS14G–21</td>
</tr>
<tr>
<td>Downstream channel response</td>
<td>TS14G–21</td>
</tr>
<tr>
<td>Geologic controls</td>
<td>TS14G–21</td>
</tr>
<tr>
<td>Effects on tributaries</td>
<td>TS14G–21</td>
</tr>
<tr>
<td>Grade control siting summary</td>
<td>TS14G–22</td>
</tr>
<tr>
<td>Conclusion</td>
<td>TS14G–22</td>
</tr>
</tbody>
</table>
Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table TS14G–1</td>
<td>Advantage and disadvantages of selected grade control structures</td>
</tr>
<tr>
<td>Table TS14G–2</td>
<td>Advantage and disadvantages of gabion mattresses when used in an erosion control application</td>
</tr>
</tbody>
</table>

Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS14G–1</td>
<td>Lane's balance for water discharge, slope, bed material load, and median bed-material size</td>
</tr>
<tr>
<td>Figure TS14G–2</td>
<td>An energy diagram for the preconstruction condition</td>
</tr>
<tr>
<td>Figure TS14G–3</td>
<td>The modified energy diagram for a bed control structure</td>
</tr>
<tr>
<td>Figure TS14G–4</td>
<td>The modified energy diagram for a hydraulic control structure</td>
</tr>
<tr>
<td>Figure TS14G–5</td>
<td>Channel stabilization with rock sills</td>
</tr>
<tr>
<td>Figure TS14G–6</td>
<td>Top—Riprap grade control structure: Bottom—subsequent launching of riprap at the grade control structure in response to advancing bed degradation and local</td>
</tr>
<tr>
<td>Figure TS14G–7</td>
<td>Loose rock structures are shown in plan and profile for Mink Creek, Manitoba, CA</td>
</tr>
<tr>
<td>Figure TS14G–8</td>
<td>Cross vane rock grade control structure</td>
</tr>
<tr>
<td>Figure TS14G–9</td>
<td>A J-hook grade control structure</td>
</tr>
<tr>
<td>Figure TS14G–10</td>
<td>Comparison of rock sizing methods for a 1V:20H sloping face structure</td>
</tr>
<tr>
<td>Figure TS14G–11</td>
<td>Rock riprap gradient control structure</td>
</tr>
<tr>
<td>Figure TS14G–12</td>
<td>Built riprap grade control structure with an impervious fill cutoff wall (top); launching of riprap at the grade control structure in response to bed degradation and local scour (bottom)</td>
</tr>
<tr>
<td>Figure TS14G–13</td>
<td>Built riprap grade control structure with a sheet pile cutoff wall (top); launching of riprap at the grade control structure in response to bed degradation and local scour (bottom)</td>
</tr>
<tr>
<td>Figure TS14G–14</td>
<td>ARS-type grade control structure with pre-formed riprap-lined stilling basin and baffle plate</td>
</tr>
<tr>
<td>Figure TS14G–15</td>
<td>Schematic of modified ARS-type grade control structure</td>
</tr>
<tr>
<td>Figure TS14G–16</td>
<td>ARS-type grade control structure with grouted riprap face. Upstream riprap extends below the water; however, sediment filling of the stone as shown is supporting vegetation</td>
</tr>
<tr>
<td>Figure TS14G–17</td>
<td>CTT-type drop structure</td>
</tr>
<tr>
<td>Figure TS14G–18</td>
<td>SAF drop structure</td>
</tr>
<tr>
<td>Figure TS14G–19</td>
<td>Spacing of grade control structure</td>
</tr>
<tr>
<td>Figure TS14G–20</td>
<td>Blue Creek, IL, 1997 thalweg profile and structure locations and elevations</td>
</tr>
<tr>
<td>Figure TS14G–21</td>
<td>Blue Creek, IL, thalweg profile surveyed in 2002</td>
</tr>
<tr>
<td>Figure TS14G–22</td>
<td>Grade control structure 1 month and 18 months after construction (Blue Creek, IL)</td>
</tr>
<tr>
<td>Figure TS14G–23</td>
<td>Grade control design (Blue Creek, IL)</td>
</tr>
<tr>
<td>Figure TS14G–24</td>
<td>Grade control design (Blue Creek, IL)</td>
</tr>
<tr>
<td>Figure TS14G–25</td>
<td>Riprap gradations for B-Stone, R–400, R–650, and $D_{90}$ from the Blue Creek example</td>
</tr>
</tbody>
</table>
Purpose

One of the most challenging problems facing river engineers today is the stabilization of degrading channels. Channel degradation leads to damage of both riparian infrastructure, as well as the environment. Bank protection is generally ineffective over the long term and will probably be a waste of resources if the channel continues to degrade. When systemwide channel degradation exists, a comprehensive treatment plan is usually required. A wide variety of structures has been employed to provide grade control in channel systems. The objectives of this technical supplement are to provide a description of some of the more common types of grade control structures that are frequently used throughout the United States and describe the various design factors that should be considered when selecting and siting grade control structures.

Introduction

Grade control is an essential component to stabilize a degrading stream or one that is subject to conditions that may cause degradation. Channel degradation leads to damage of bridges, culverts, petrochemical transmission lines, power lines, sewer and water lines, and other infrastructure. Channel degradation produces an overheightened and oversteepened condition of the channel banks that often leads to severe mass failures of both streambanks. The resulting channel widening and bank erosion cause severe land loss and damage to riparian infrastructure and habitat.

As channel degradation continues, the ground water table may also be lowered along the stream, affecting riparian vegetation. Sediment eroded from the degrading channels is transported downstream, adversely impacting flood control channels, reservoir areas, and wetland habitat areas. This sediment also carries significant amounts of nutrients, particularly phosphorus, which may degrade water quality and habitat along the stream system. Consequently, channel degradation is not simply a local problem that only affects a few landowners, but rather, produces systemwide consequences that can impact all taxpayers.

When systemwide channel degradation exists, a comprehensive treatment plan is usually required. This treatment plan usually involves the use of one or more grade control structures to arrest the degradation process. In the widest sense, the term grade control can be applied to any alteration in the watershed that provides stability to the streambed. It can include stream realignments. The most common method of establishing grade control is the construction of in-channel grade control structures. A wide variety of grade control structures has been used in channel systems. These treatments range from simple loose rock structures to reinforced concrete weirs and vary in scale from small streams to large rivers. While some stream rehabilitation practitioners suggest that grade control cannot be constructed in incised channels, the authors have routinely participated in the design and long-term monitoring of successful grade control structures in severely incised channels.

The two primary engineering factors that promote channel stability are continuity of water and sediment through the stream reach and geotechnical bank stability. A series of well-designed grade control structures can adjust sediment transport capacity to sediment supply and can improve bank stability by reducing bank height and reducing shear at the bank toe. As with most water resources activities, there are positive and negative environmental impacts associated with grade control structures. The most serious negative environmental impact commonly associated with grade control structures is obstruction to fish passage. On the positive side, grade control structures can improve the channel stability, improve habitat, and reduce the supply of sediment and nutrients to the channel system. Fish passage issues, as well as other challenges, can be accommodated through appropriate engineering design and by close cooperation with biologists on the planning and design team. Fish passage is described further in NEH654 TS14N.

Grade control hydraulics

There are two basic types of grade control structures. A bed control structure is designed to provide a hard point in the streambed that is capable of resisting the erosive forces of the degradational zone. This is somewhat analogous to locally increasing the size of the bed material. The Lane relation (Lane 1955b) (fig. TS14G–1) illustrates the dynamic relationship, $Q^* \propto Q^* \frac{S}{D}$, 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 ($Q_S \propto Q_s D_{50}$). 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 pre-construction 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

<table>
<thead>
<tr>
<th>Structure type</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose rock structures</td>
<td>Economical to design and build</td>
<td>Generally limited to less than about 3 ft drop heights</td>
</tr>
<tr>
<td></td>
<td>Limited environmental impacts</td>
<td>Potential for displacement of rock due to seepage flows</td>
</tr>
<tr>
<td>Channel linings</td>
<td>Provides for energy dissipation through the structure</td>
<td>Significant design effort</td>
</tr>
<tr>
<td></td>
<td>Can be designed to accommodate fish passage</td>
<td>Relatively high cost</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Larger construction footprint due to length of structure</td>
</tr>
<tr>
<td>Loose rock structures with water cutoff</td>
<td>Provides positive water cutoff that eliminates seepage problems and potential for rock displacement</td>
<td>More complex design required</td>
</tr>
<tr>
<td></td>
<td>Higher drop heights (up to about 6 ft)</td>
<td>Higher construction cost than simple loose rock structures</td>
</tr>
<tr>
<td>Structures with preformed scour holes and water cutoffs</td>
<td>Improved energy dissipation</td>
<td>More potential for fish obstruction at higher drop heights</td>
</tr>
<tr>
<td></td>
<td>Scour holes provide stable reproductive habitat</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Higher drop heights (up to about 6 ft)</td>
<td></td>
</tr>
<tr>
<td>Rigid drop structures</td>
<td>Can accommodate drop heights greater than 6 ft</td>
<td>High construction cost</td>
</tr>
<tr>
<td></td>
<td>Provides for energy dissipation</td>
<td>Large construction footprint</td>
</tr>
<tr>
<td></td>
<td>Single structure can influence long reach of stream</td>
<td>Significant potential for obstruction to fish</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Potential for downstream channel degradation due to trapping of sediment</td>
</tr>
<tr>
<td>Alternative construction materials</td>
<td>Economically feasible where stone is costly and local labor force is inexpensive and available</td>
<td>Often lack detailed design guidance</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Increased monitoring and maintenance often required</td>
</tr>
</tbody>
</table>
Loose rock structures

Perhaps the simplest form of a grade control structure consists of placing natural stone or other erosion resistant elements across the channel to form a hard point. Some manufactured concrete products may be used in place of stone. This type of structure includes rock sills, rock sills with impermeable cutoffs, artificial riffles, and sloping rock structures. Various types of loose rock structures are presented herein along with rock sizing procedures and some methods for local scour protection.

Types of loose rock structures
Loose rock structures are generally most effective for drop heights that are less than about 2 to 3 feet. In many applications, a series of loose rock structures are placed relatively close together, effectively providing a greater drop height than a single structure. The series of loose rock structures then provides a degree of conservatism in the design, as one element may reduce stress on the upstream element. Loss of one element may not mean loss of function for the total treatment. The structures must be spaced close enough that channel degradation above one does not undermine the upstream structure. A series of rock sills, each creating a head loss of about 2 feet, was used successfully on the Gering Drain in Nebraska (Stufft 1965). The design concept presented by Whittaker and Jäggi (1986) for stabilizing the streambed with a series of rock sills is shown in figure TS14G–5. These sills are bed control structures that are simply acting as hard points to resist streambed erosion.

Construction of bed sills is sometimes accomplished by placing the rock along the streambed to act as a hard point to resist the erosive forces within the degradational zone. In other situations, a trench may be excavated across the streambed and then filled with rock. A critical component in the design of these structures is ensuring that there is a sufficient volume of erosion resistant material to resist the general bed degradation, as well as any additional local scour at the structure. This is illustrated in figure TS14G–6, which shows a riprap grade control structure designed to resist both the general bed degradation of the approaching nickpoint, as well as any local scour that may be generated at the structure. In this instance, the riprap section must have sufficient mass (layer thickness) to launch into the anticipated scour hole.

A unique type of loose rock structure is used by Newbury and Gaboury (1993). These are often referred to as Newbury riffles. The structures are placed at 5 to 7 channel widths spacing to emulate the spacing of

---

**Figure TS14G–5**  Channel stabilization with rock sills

(a) Original bed

(b) Degradation about fixed point

(c) Block sills

Stable channel slope

Stable reach

---

**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

Flow — Riprap grade control structure

Streambed

Knickpoint

Flow — Launched stone

Original bed

Local scour

Bed degradation
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.
**Rock sizing for loose rock structures**

A common factor in all loose rock structures is determining the proper stone size. While a more comprehensive description of rock sizing can be found in TS14C, six methods are presented:

**Method 1: U.S. Army Corps of Engineers (1994f)**

The U.S. Army Corps of Engineers (USACE) developed criteria for sizing steep slope riprap where unit discharges are low and slopes range from 2 to 20 percent. A typical application would be a rock-lined chute. The stone size equation is:

\[
D_{30} = \frac{1.95 S^{0.55} q^{2/3}}{g^{1/3}}
\]  
(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:

\[ q_{\text{design}} = \frac{q_f}{0.74} = 1.35q_f \] \hspace{0.5cm} \text{(eq. TS14G–2)}

\[ D_{50} = 5.23S^{0.43}q_{\text{design}}^{0.56} \] \hspace{0.5cm} \text{(eq. TS14G–3)}

where:
- \( q_f \) = stone size at failure (in)
- \( q_{\text{design}} \) = design discharge (ft\(^3\)/s/ft)
- \( S \) = slope of the riprap layer

**Method 3: Whittaker and Jäggi (1986)**

\[ \frac{q}{\sqrt{gD_{65}^2(G_s - 1)}} \leq \frac{0.257}{J^2} \] \hspace{0.5cm} \text{(eq. TS14G–4)}

where:
- \( q \) = specific discharge over the ramp (m\(^3\)/s × m)
- \( D_{65} \) = characteristic block diameter of the block mixture (m)
- \( G_s \) = specific gravity of the blocks compared to that of the water (e.g., 2.65)
- \( J \) = ramp gradient
- \( g \) = acceleration due to gravity (m/s\(^2\))

**Method 4: Newbury and Gaboury (1993)**

tractive force (kg/m\(^2\)) = incipient diameter (cm)

\[ (\text{eq. TS14G–5}) \]


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:

\[ q = 0.52 D_{50}^{0.80} S_o^{1.50} \text{ for } S_o < 0.10 \] \hspace{0.5cm} \text{(eq. TS14G–6)}

\[ q = 4.30 D_{50}^{0.80} S_o^{1.50} \text{ for } 0.10 < S_o < 0.40 \] \hspace{0.5cm} \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 2\( D_{50} \). 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:

\[ \text{minimum rock size (m)} = 0.1724 \ln(\text{bankfull shear stress, kg/m}^2) + 0.6349 \] \hspace{0.5cm} \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.

**Rock sizing summary**

Figure TS14G–10 compares the six different procedures using a 5 percent sloping (1V:20H) loose rock structure at a unit discharge varying from 1 to 10 cubic meters per meter of width. It should be noted that the \( D_n \) varied between the methods, so an absolute comparison was not possible. For instance, Chervet and Weiss (1990) specified \( D_{50} \), Abt and Johnson (1991) and Robinson, Kadvey, and Rice (1998) specified \( D_{50} \), USACE (1994a) specified \( D_{30} \), Newbury and Gaboury (1993) did not specify a rock size within the gradation, and the Rosgen (2002) method calculates the minimum rock size. However, comparison of the curves in figure TS14G–10 indicates that, with the exception of the Rosgen method, there is general consistency among the other five methods. It is important to note that the Rosgen (2002) relationship determines the minimum size of rock required and unlike the other methods, does not calculate a stone gradation. Therefore, it is not surprising that Rosgen’s results are not compatible.
with the other methods. If the sloping loose rock structures are to be constructed in a location that will encounter completely submerged conditions, traditional riprap-sizing methods (USACE 1994f; U.S. Department of Transportation Federal Highway Administration (FHWA) 2001a) should be used to check structure stability. An example design procedure for a sloping loose rock drop structure, adapted from Watson and Eom (2003), is provided at the end of this technical supplement.

**Local scour protection for loose rock structures**

Chervet and Weiss (1990) reviewed work by Whittaker and Jäggi (1986) and developed a relationship for predicting local scour at the downstream extent of a loose rock structure, referred to by the authors as a block ramp.

The maximum scour depth \( t \) can be estimated using the following approach (Tschopp-Bisaz, modified in accordance with Whittaker and Jäggi (1986)):

\[
 t + h_U \equiv 0.85q^{0.5} \left( \frac{q}{h_N} \right)^{0.5} - 7.125D_{90}
\]

(eq. TS14G–9)

where:

- \( h_U \) = tailwater depth (m);
- \( h_N \) = normal supercritical discharge depth over the ramp (m), e.g., calculated according to Strickler’s formula, using a coefficient of friction of \( k = 21/D_{65}^{1/6} \) (m\(^{1/3}\)/s);
- \( t \) = predicted scour depth (m)

Local scour depth is directly related to unit discharge, and an inverse relationship is shown for tailwater depth and the \( D_{90} \) of the bed material. Chervet and Weiss (1990) recommend that the downstream extent of the structure should extend below an anticipated local scour depth.

Bitner (2003) reviewed local scour depth, reporting that Castro (1999) defined bed key depth as the local scour depth to which the rock structure should be excavated to prevent undermining. Castro recommended that the scour depth may approach 2.5 times the drop height for gravel or cobble beds, and 3.5 times the drop height for sand beds.

**Channel linings**

Grade control can also be accomplished by lining the streambed with an erosion resistant material. These structures are designed to ensure that the drop is accomplished over a specified stream reach that has been lined with riprap or some other erosion-resistant material. Rock riprap gradient control structures have been used by the U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) (formerly the Soil Conservation Service (SCS) 1976) for several years. These structures are designed to flow in the subcritical regime with a constant specific energy at the design discharge, which is equal to the specific energy of flow immediately upstream of the structure (Myers 1982). Although these structures have generally been successful, some have had local scour problems. This precipitated a series of model studies to correct these problems and to develop a design methodology for these structures (Tate 1988, 1991). Plan and profile drawings of the improved structure are shown in figure TS14G–11 (adapted from Tate 1991).
Loose rock structures with water cutoff

One problem often encountered with channel lining structures is the displacement of rock (or rubble) due to the seepage flow around and beneath the structure. This is particularly a problem when the bed of the stream is composed primarily of pervious material. This problem can be eliminated by constructing a water barrier at the structure. One type of water barrier consists of simply placing a trench of impervious clay fill upstream of the weir crest (fig. TS14G–12). In general, this type of barrier has limited longevity due to susceptibility to erosion. This erosion can be avoided by using a concrete or sheet pile cutoff wall. The conceptual design of a riprap grade control structure with a sheet pile cutoff wall is shown in figure TS14G–13.

Structures with preformed scour holes and water cutoff

A scour hole is a natural occurrence downstream of any overfall. Sizing of the scour hole is a critical element in the design process, which is usually based on model studies or on experience with similar structures in the area.

The stability of rock structures is often jeopardized at low tailwater conditions. One way to ensure the stability of the rock is to design the structure to operate in a submerged condition. Linder (1963) developed a structure that is designed to operate at submerged conditions where the tailwater elevation (T) does not fall below 0.8 of the critical depth (d₂) at the crest section. Subsequent monitoring of the in-place structures confirmed the successful performance in the field (USACE 1981).

Little and Murphey (1982) developed a loose rock structure incorporating a sheet pile cutoff and weir, and a preformed scour basin lined with riprap that acts as an energy dissipation basin. They observed that an undular hydraulic jump occurs when the incoming Froude number is less than 1.7. Consequently, Little and Murphey developed a grade control design that included an energy dissipating baffle to break up these undular waves (fig. TS14G–14). This structure, referred to as the Agricultural Research Service (ARS)-type low-drop structure, has been used successfully in northern Mississippi for drop heights up to about 2 meters by both the USACE and the U.S. Department of Agriculture (USDA) SCS (USACE 1981). A recent modification to the ARS structure was developed following model studies at Colorado State University (Johns et al. 1993; Abt et al. 1994). The modified ARS structure, presented in figure TS14G–15, retains the baffle plate, but adopts a vertical drop at the sheet pile, rather than a sloping rockfill section as recommended by Little and Murphey.

Smith and Wilson (1992) provide guidance for design and construction of the ARS-type grade control structure. The guidance is replete with information, and several specific points follow:

- For selection of the final structure site, the stream should be straight for a distance of 10 stream widths upstream and for a minimum of 200 feet downstream.

- No gullies or lateral drains should occur in the site.

- The base width of the weir should be constricted to ensure that the water surface elevation of the 2-year discharge moves from critical depth near the weir crest to normal depth of flow in a short distance; for example, a few stream widths.

- The resulting flood-control impacts should not violate flood-control requirements.
Figure TS14G–12  Top—built riprap grade control structure with an impervious fill cutoff wall; Bottom—launching of riprap at the grade control structure in response to bed degradation and local scour.

Figure TS14G–13  Top—built riprap grade control structure with a sheet pile cutoff wall (top); Bottom—launching of riprap at the grade control structure in response to bed degradation and local scour.
Figure TS14G–14  ARS-type grade control structure with preformed riprap-lined stilling basin and baffle plate
• The stilling basin dimensions should be based on the smaller of the bankfull discharge or the 100-year discharge.
• Downstream tailwater conditions should be based on normal depth calculations of an estimated future, degraded condition.
• Stilling basin riprap size is based on physical model studies referenced in the guidance. Approach stream protection and exit stream protection are specified.

Recent modifications to the ARS-type grade control structure by the USACE Vicksburg District replaced the vertical face downstream of the weir with a 1V:2H sloping face constructed of grouted riprap (fig. TS14G–16). Upstream riprap extends below the water; however, sediment filling of the stone as shown is supporting vegetation. Other modifications included elimination of the baffle plate and the construction of an impervious fill section at the weir section in lieu of the sheet pile cutoff wall. Annual monitoring of these structures since the early 1990s has revealed no significant negative structural or channel impacts.

Rigid drop structures

In many situations where the discharge and/or drop heights are large, in excess of 2 meters, grade control structures are frequently constructed of concrete or a combination of sheet pile and concrete. There are many different designs for concrete grade control structures. Two described here are the California Institute of Technology (CIT) and the St. Anthony Falls (SAF) structures. Both of these structures were used on the Gering Drain project in Nebraska, where the decision to use one or the other was based on the flow and stream conditions (Stuft 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-
sippi, and is shown in figure TS14G–17 (Murphy 1967). Where the stream was relatively deep and the discharges smaller, the SAF drop structure was used. This design was developed from model studies at the SAF Hydraulic Laboratory for the SCS (Blaisdell 1948). This structure is shown in figure TS14G–18. The SAF structure is capable of functioning in flow conditions where the drop height to critical depth ratio is greater than 1 and can provide effective energy dissipation within a Froude number range of 1.7 to 17. Both the CIT and the SAF drop structures have performed satisfactorily on the Gering Drain for more than 25 years.

The design for a large, rigid structure should include consideration of slope stability including sudden drawdown. Slope stability should also be investigated for the site, approach, and downstream channels. Stability analyses should include sliding stability of the structure, underseepage, and allowance for bearing capacity and settlement. As the hydraulic capacity and drop height of the structure increases, the complexity of design and construction increases.

**Alternative construction materials**

While riprap, sheet pile, and concrete may be the most commonly used construction materials for grade control structures, cost or availability of materials may prompt the engineer to consider other alternatives.
Figure TS14G–18  SAF drop structure

Rectangular stilling basin
Half-plan

Trapezoidal stilling basin
Half-plan

Centerline section

Downstream elevation
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 (Rijkswaterstaat 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

<table>
<thead>
<tr>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ability to span minor pockets of subsidence without failure</td>
<td>Susceptibility of the wire baskets to corrosion, abrasion damage, and vandalism</td>
</tr>
<tr>
<td>Interlock to allow use of smaller, lower quality rock in the baskets</td>
<td>High labor cost associated with fabrication and filling the baskets</td>
</tr>
<tr>
<td>Economically feasible where riprap-sized rock is not readily available</td>
<td>More difficult and expensive repair than standard rock protection</td>
</tr>
</tbody>
</table>
rip rap should not be used with grout and that rock with all dimensions greater than the grout thickness be required and placed on a firm subgrade. Grout is then pumped into the voids and vibrated, filling the voids between rocks. The method results in the appearance of a concrete slab with large stones spaced evenly, protruding through the slab. Toe and lateral drains are included for drainage of the grouted area.

**General design considerations for siting grade control structures**

Design considerations for siting grade control structures include determination of the type, location, and spacing of structures, along with the elevation and dimensions of the structures. Siting grade control structures is often considered a simple optimization of hydraulics and economics. However, these factors alone are usually not sufficient to define the optimum grade control siting conditions. In practice, hydraulic considerations must be integrated with a host of other factors that vary from site to site to determine the final structure plan. Some of the more important factors to be considered when siting grade control structures are described in the following sections. This does not represent an all-inclusive list, since there may be other factors that may be locally important. For example, maintenance requirements, debris passage, ice conditions, or safety considerations may be controlling factors. Consequently, there is no definitive procedure for siting grade control structures. However, consideration of each factor in an analytical and balanced fashion can lead to a more effective design process that will ensure that the plan accomplishes the long-term project goals.

**Hydraulic and sediment transport considerations**

One of the most important steps in the siting of a grade control structure or a series of structures is the drop height determination. This requires some knowledge of the ultimate stream morphology, both upstream and downstream of the structure, which involves assessment of sediment transport and stream morphologic processes.

The hydraulic siting of grade control structures is a critical element of the design process, particularly when a series of structures is planned. The design of each structure is based on the anticipated tailwater or downstream bed elevation, which in turn, is a function of the next structure downstream. Heede and Mulich (1973) suggested optimum spacing of structures, so that the upstream structure does not interfere with the deposition zone of the next downstream structure. Mussetter (1982) showed that the optimum spacing should be the length of the deposition above the structure, which is a function of the deposition slope (fig. TS14G–19 (adapted from Mussetter). Figure TS14G–19 also illustrates the recommendations of Johnson and Minaker (1944), that the most desirable spacing can be determined by extending a line from the top of the first structure, at a slope equal to the maximum equilibrium slope of sediment upstream, until it intersects the original streambed.

Theoretically, the hydraulic siting of grade control structures is straightforward and can be determined by:

\[ H = (S_o - S_f)X \]  (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:

\[ N = \frac{H}{h} \]  (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.

**Geotechnical considerations**

The previous description focused only on the hydraulic aspects of design and siting of grade control structures. In some cases, the geotechnical stability of the reach may be an important or even the primary factor to consider when siting grade control structures. This is often the case where stream degradation has caused, or is anticipated to cause, severe bank instability due to exceedance of the critical bank height (Thorne and Osman 1988). When this occurs, bank instability may be widespread throughout the system, rather than restricted to the concave banks in bends. 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); Coppen 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 flow nets 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 restoration. 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 D30 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 D30 value plots with respect to several commonly used riprap gradations. As shown in figure TS14G–24, the Blue Creek D30 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

![Graph showing thalweg profile](image1.png)

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

![Image of grade control structure](image2.png)
Figure TS14G–23  Grade control design (Blue Creek, IL)

Back fill

Original stream bottom

V-shaped crest

6-in gravel layer

Riprap

Excavate to firm material. (2 ft min)

Note: Completely fill key trench with riprap. Minimum trench depth of $D_{100}$.

Stream bottom approximate width

Note: Quarried stone should be approximately 3 by 3 by 2 ft and should fit together relatively tightly. Do not pick larger stone from specified riprap mixture; this would result in undersized crest stone and improper riprap gradation.

Section B–B

Quarried stone

Section C–C

Stream bottom approximate width

Not to scale

Note: Quarried stone should be keyed into each bank of pre-construction native material a minimum distance of 5 ft.
Figure TS14G–24  Grade control design (Blue Creek, IL)

Plan

Section A–A

Not to scale
Figure TS14G–25  Riprap gradations for B-Stone, R–400, R–650, and $D_{30}$ from the Blue Creek example
Flow Changing Techniques
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.

(210–VI–NEH, August 2007)
## Technical Supplement 14H

### Flow Changing Techniques

<table>
<thead>
<tr>
<th>Contents</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose</td>
<td>TS14H-1</td>
</tr>
<tr>
<td>Introduction</td>
<td>TS14H-1</td>
</tr>
<tr>
<td>Flow-changing techniques</td>
<td>TS14H-1</td>
</tr>
<tr>
<td>Spur dikes</td>
<td>TS14H-3</td>
</tr>
<tr>
<td>Groins</td>
<td>TS14H-3</td>
</tr>
<tr>
<td>Jetties</td>
<td>TS14H-3</td>
</tr>
<tr>
<td>Pin deflectors</td>
<td>TS14H-4</td>
</tr>
<tr>
<td>Bendway weirs</td>
<td>TS14H-4</td>
</tr>
<tr>
<td>Stream barbs</td>
<td>TS14H-4</td>
</tr>
<tr>
<td>Vanes</td>
<td>TS14H-6</td>
</tr>
<tr>
<td>Stream barbs</td>
<td>TS14H-7</td>
</tr>
<tr>
<td>Hydraulic function</td>
<td>TS14H-7</td>
</tr>
<tr>
<td>Design criteria</td>
<td>TS14H-7</td>
</tr>
<tr>
<td>Design worksheet</td>
<td>TS14H-15</td>
</tr>
<tr>
<td>Cost</td>
<td>TS14H-16</td>
</tr>
<tr>
<td>Construction considerations</td>
<td>TS14H-16</td>
</tr>
<tr>
<td>Conclusion</td>
<td>TS14H-16</td>
</tr>
</tbody>
</table>
### Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table TS14H–1</td>
<td>Common flow changing techniques, brief description, structure class, and function</td>
</tr>
</tbody>
</table>

### Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS14H–1</td>
<td>Permeable fence jetty close up and aerial view</td>
</tr>
<tr>
<td>Figure TS14H–2</td>
<td>Bendway weir under construction and completed bendway weir</td>
</tr>
<tr>
<td>Figure TS14H–3</td>
<td>Bendway weir</td>
</tr>
<tr>
<td>Figure TS14H–4</td>
<td>Water velocities on Geffert River Project, Neosho River, Allen County, KS</td>
</tr>
<tr>
<td>Figure TS14H–5</td>
<td>Rock barbs and brush barbs</td>
</tr>
<tr>
<td>Figure TS14H–6</td>
<td>Rock vane</td>
</tr>
<tr>
<td>Figure TS14H–7</td>
<td>Approximate surface velocity measurements at Snake River, WY</td>
</tr>
<tr>
<td>Figure TS14H–8</td>
<td>Typical stream barb design layout</td>
</tr>
<tr>
<td>Figure TS14H–9</td>
<td>Historical meander migration limits</td>
</tr>
<tr>
<td>Figure TS14H–10</td>
<td>Depth of bed key</td>
</tr>
<tr>
<td>Figure TS14H–11</td>
<td>Scour effects at the barb tip</td>
</tr>
<tr>
<td>Figure TS14H–12</td>
<td>Rootwad used in the key of a stream barb</td>
</tr>
<tr>
<td>Figure TS14H–13</td>
<td>Drawing and layout details</td>
</tr>
<tr>
<td>Figure TS14H–14</td>
<td>Typical stream barb construction drawing</td>
</tr>
<tr>
<td>Figure TS14H–15</td>
<td>Detail showing the use of a rootwad incorporated into a stream barb</td>
</tr>
</tbody>
</table>
Purpose

Flow changing devices are a broad category of structures that can be used to divert flows away from eroding banks. They are often used to shield banks from eroding flows, build up the toe of the bank, and direct flows to create a stable alignment. While this technical supplement provides descriptions of a variety of techniques, the primary focus is on the analysis, design, and installation of stream barbs. This supplement draws on recent field evaluations that focused both on projects where these structures have performed satisfactorily, as well as areas where the performance has been less than satisfactory. A design description includes cautions and warnings related to specific design features. Finally, a step-by-step design procedure for stream barbs is also provided.

Introduction

The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) has installed numerous flow-changing techniques in support of both streambank stabilization and stream restoration practices. This supplement primarily addresses stream stabilization techniques that work to decrease flow stresses on an eroding streambank through redirection of flow. While a variety of techniques are described, the primary focus of this supplement is on stream barbs. This supplement also provides current NRCS design recommendations for stream barb design.

Flow-changing techniques

The structures used for stream and bank restoration in NRCS projects can be categorized into one of three general classes. The terms used to identify structure classes are somewhat descriptive of the structure function.

- deflector
- redirective
- retard

A deflector type structure forms a physical barrier that protects the bank and forces the flow to change direction either by direct impact or deflection. These structures tend to be massive and often continuous along the protected reach. When properly designed, deflector structures are stable over a wide range of flow conditions.

Rock riprap, grouted rock, concrete lining, rock jetties, gabions, and spur dikes are examples of deflector structures that have historically been used in streambank protection work. Except for rock jetties and spur dikes, these structures harden the bank and reduce roughness, thereby increasing flow velocity. Common building materials for these structures are graded rock, concrete, earthfill, or combinations of these materials. Some of these techniques are addressed in more detail in NEH654.14.

A redirective type structure is designed to be placed in the stream to minimize direct impact and rely more on the characteristics of fluid mechanics to modify the streamflow direction. These structures tend to be less massive and are submerged at higher stages of flow. Redirective structures are usually discontinuous, independent structures. In many cases, they are more likely to be damaged during major events.

Spurs, rock veins/weirs, stream barbs, and bendway weirs fall into the category of redirective structures. Redirective structures can be contrasted with deflector techniques, such as riprap and gabions, which are more static and harden the bank. Common building materials for these structures typically include large rock, graded rock, and earthfill.

A retard structure increases flow resistance by increasing drag, thereby slowing the velocity in the vicinity of the structure. These structures are more porous with a high percentage of open area. Retard structures are generally used where the channel carries a high sediment load and reducing the velocity will result in sediment deposition. Common building material for these structures can include wood, steel, rock, and live plantings. Fence jetties, Killner jacks, timber piling, live poles, and most bioengineered structures are examples of retard structures. Some of these structures are addressed in more detail in NEH654.14.

It is not uncommon to use all three types on projects initiating and terminating protected reaches with deflector type structures and using redirective and retard structures between the hard points. All of the methods
mentioned can be combined with bioengineering measures to improve stream function and bank stability. A general outline of the different techniques is provided in Table TS14H–1. Some of these techniques are addressed in further detail in this technical supplement, as well as in NEH654.14.

<table>
<thead>
<tr>
<th>Practice</th>
<th>Description</th>
<th>Structure class</th>
<th>Function</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete bank lining</td>
<td>Hard, smooth surface of concrete, gravity, or structural support</td>
<td>Deflector</td>
<td>Flow is physically deflected or trained by physical barrier</td>
</tr>
<tr>
<td>Rock masonry bank lining or wall</td>
<td>Hard, semismooth of rock and mortar, gravity support</td>
<td>Deflector</td>
<td>Flow is physically deflected or trained by physical barrier</td>
</tr>
<tr>
<td>Geocell slope/bank protection</td>
<td>Fine or granular fill retained in cells, semismooth to rough, vegetated option</td>
<td>Deflector</td>
<td>Flow is physically deflected or trained by physical barrier</td>
</tr>
<tr>
<td>Rock riprap</td>
<td>Loose rock on slope, semismooth to rough, full or partial bank</td>
<td>Deflector</td>
<td>Flow is physically deflected or trained by physical barrier</td>
</tr>
<tr>
<td>Groins</td>
<td>Rock dike projecting into stream in downstream direction</td>
<td>Deflector</td>
<td>Full range of flows physically deflect-ed away from bank</td>
</tr>
<tr>
<td>Dike</td>
<td>Earth or rock full bank height</td>
<td>Deflector</td>
<td>Flow is physically deflected or trained by physical barrier</td>
</tr>
<tr>
<td>Stream barbs</td>
<td>Low rock sill projecting into stream</td>
<td>Redirective</td>
<td>Flow direction changed by flow over structure</td>
</tr>
<tr>
<td>Bendway weirs</td>
<td>Low rock sill projecting into stream</td>
<td>Redirective</td>
<td>Flow direction changed by flow over structure</td>
</tr>
<tr>
<td>Rock vein</td>
<td>Instream rock sill</td>
<td>Redirective</td>
<td>Flow direction changed by flow over structure</td>
</tr>
<tr>
<td>Rock “V” weir</td>
<td>Instream rock sill</td>
<td>Redirective</td>
<td>Flow direction changed by flow over structure</td>
</tr>
<tr>
<td>Spur dike</td>
<td>Short rock, timber, or earth dike projecting from bank, porous or impermeable</td>
<td>Deflector/retard</td>
<td>Physical barrier, full bank height</td>
</tr>
<tr>
<td>Jetties (fence)</td>
<td>Parallel lines of spaced posts, porous</td>
<td>Retard</td>
<td>Velocity of flow through structure is reduced by friction</td>
</tr>
<tr>
<td>Live stakes, geogrids, brush layers</td>
<td>Vegetative treatment</td>
<td>Retard/deflector</td>
<td>Velocity of flow through and around vegetation is slowed by friction</td>
</tr>
<tr>
<td>Vegetated slope</td>
<td>Vegetative treatment</td>
<td>Retard/deflector</td>
<td>Velocity of flow through and around vegetation is slowed by friction</td>
</tr>
</tbody>
</table>

Table TS14H–1  Common flow-changing techniques, brief description, structure class, and function
Spur dikes

Spur dikes are short dikes that extend out perpendicular from the bank into the channel along a reach of eroded bank. Spur dikes can be short or long, but generally with a top elevation above flood stage or equal to the bank elevation. Streamflow impacting spur dikes is retarded and diverted away from the bank. Spacing of the spur dikes is important to prevent formation of strong eddies that can result in erosion between the dikes. Spur dikes are generally constructed using earthfill with rock riprap surface protection. However, soil bioengineering practices can also be used in between spurs.

Groins

Historically, groins have been in widespread use for many years and are the precursors to redirective structures. Much of the guidance for redirective structures is based in part on the experience with groins. However, there are important differences that the designer must keep in mind. Groins typically are higher profile and affect all stages of flow. Their crest is typically above the high-flow water surface elevation, and they are seldom completely submerged. They act to deflect flows away from the bank. They have a significantly higher effect on the shape of the streams cross-sectional shape since they are used to narrow the stream. Since they are rarely overtopped, they can be effective when oriented downstream.

Jetties

Jetties are fence-like structures extending from the bank into the stream. They are often installed in pairs or multiple pairs to train flow towards the center of the channel. They can also be installed on one side of a stream channel to direct flow away from that bank. Jetties can be permeable or impermeable and are usually installed diagonally in a downstream direction along the bank.

Figure TS14H–1 shows an example of permeable fence jetties. Permeable jetties are used for streams with high sediment loads. The flow passing through the jetty is slowed, allowing deposition of material between the jetties. Impermeable jetties are seldom used except where the line of flow must be diverted away from a structure or other feature. Permeable jetties can also be constructed out of woody debris, jacks, or a combination of logs and large boulders. In streams where there is a large amount of woody material and debris, permeable deflectors can collect and retain this material and become less permeable with time. Once they become impermeable, the portions that project from the bank may function more in a redirective capacity.

Figure TS14H–1 (a) Permeable fence jetty, close up; (b) Aerial view (Photo courtesy of Lamont Robbins, NRCS)
Pin deflectors

A variation of the permeable jetty is the pin or piling deflector. Pin deflectors are generally used in streams where only a small reduction in velocity is needed. Generally, wood pilings are used for their construction. These pilings are driven to a depth where they can resist the forces of the water, as well as any anticipated drift and debris that they may collect. A rule of thumb is a depth that is at least twice that of the projection above the channel bottom, but this is dependent on channel materials. In some applications, it is specified that the piling be driven to refusal. After being driven to the design depth, the pilings can be trimmed with a chain saw to form the design profile. Pilings can be linked with cross pieces or left as individual elements. When connected, they act together. When unconnected, outer wood pilings may fail without putting the rest of the structure in jeopardy.

Bendway weirs

Bendway weirs were developed by the U.S. Army Corps of Engineers (USACE) to reduce erosion along the Mississippi River, and then adapted for smaller streams. As with stream barbs, the premise behind the function of bendway weirs is that flow over the weir is directed perpendicular to the angle of the weir. Bendway weirs are oriented upstream at an angle that is between 50 to 80 degrees to bank tangent. The length of a bendway weir is typically less than a fourth bankfull width. Often, the design is based on baseflow widths. In this case, their length is typically between a fourth to a half of the baseflow width. In all cases, both the length and angle may vary through the bend of the river to better capture, control, and direct the flows. They are typically wide structures with a flat to slight weir slope up toward bank. They should be keyed into the bank at a length equal to the bank height plus anticipated scour depth. More information on the design and application of bendway weirs is provided in the WES Stream Investigation and Streambank Stabilization Handbook (Biedenharn, Elliott, and Watson 1997). While bendway weirs are often used on large streams and rivers (fig. TS14H–2), an example of a bendway weir on a small stream is shown in figure TS14H–3.

Numerous applications have shown that bendway weirs reduce the velocity near the bank. On the little Blue River in Kansas, Balch (2004) observed a 50-percent reduction in stream velocities within the weir field (fig. TS14H–4).

Stream barbs

Stream barbs are low dikes or sill-like structures that extend from the bank towards the stream in an upstream direction. Stream barbs are similar in structure to bendway weirs, perform a similar function, and were developed about the same time by NRCS for...
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.

### Vanes

Vanes are structures constructed in the stream designed to redirect flow by changing the rotational eddies normally associated with streamflow. They are used extensively as part of natural stream restoration efforts to improve instream habitat. There are quite a few variants on rock vane design. The Rosgen style cross vane and J-hook structures are addressed in NEH654 TS14G and NEH654.11.

Vanes are typically oriented upstream 20 to 30 degrees to the bank tangent. However, the angle may vary as they work around the curve. Design of vanes is based on bankfull depth. The length is typically a third of the bankfull width, and the height at the bank is a third of the bankfull depth. The weir slope is 2 to 7 degrees up towards bank. The required stone size for vanes is often very large. A typical rock vane is shown in figure TS14H–6.
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 ft³/s.

Map of flow over stream barbs
Snake River at Moose, Wyoming
May 24, 2005 discharge=5,770 ft³/s
Top of stream barbs were est. at 2 ft below water surface.
Mapping by:
Ken Worster
Civil Engineer
USDA-NRCS-NDSCMC

**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
be used to change the meander pattern of an entire stream system or to channelize the streamflow.

**Number and spacing**—The number of stream bars 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 bars is necessary to prevent the streamflow from cutting between two bars 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 bars. 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 \( L_c \). A limited stream barb spacing of four to five times \( L_c \) 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:

\[
H_w = \frac{1}{3} D_a \text{ to } \frac{1}{2} D_a
\]  
(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 bars 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

\[ (210–VI–NEH, August 2007) \]
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. 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.

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:

---

**Figure TS14H–10** Depth of bed key

- **Flow**
- **Bed**

$H = h = \text{height of exposed rock relative to bed}$

- $\text{Scour} = 2.5 \times h$ (gravel or cobble bed streams)
- $= 3$ to $3.5 \times h$ (sand bed streams)
The stream barb size is determined as follows:

- \( D_{50} \text{ stream barb} = 2 \times D_{50} \text{ stream-} \)
  - bank riprap

- \( D_{100} \text{ stream barb} = 2 \times D_{50} \text{ stream barb} \)

- \( D_{\text{minimum}} = 0.75 \times D_{50} \text{ as determined for stream-} \)
  - bank 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 bars, as it results in sizes too small for this application.

In general, structures that are constructed with graded material perform better than ones built out of a few large boulders. This may be due to the fact that a structure built with a larger number of smaller stones can be more easily constructed to a specified grade and can adjust better than one made out of a few larger boulders. However, it should be noted that, depending on availability, large rock (generally greater than 3 feet in diameter) can be less expensive by weight and can take less time to install. More information on stone size is provided in NEH654 TS14C and NEH654 TS14G.

**Figure TS14H–11** Scour effects at the barb tip
Woody debris—Rootwads and other woody debris have been incorporated into stream barbs to enhance aesthetics and the habitat benefits of the structure. Details of such structures are provided in figure TS14H–12. Large wood elements have also been incorporated into the weir, as well. Rootwad sections have been incorporated both perpendicular to the weir, as well as longitudinally. In either case, the anchoring requirements of the wood elements must be considered.

If the wood element is not anchored sufficiently, it may break loose, damage the structure, and possibly result in adverse downstream impacts. Anchoring could be accomplished by cabling to rock bolsters, soil anchors, or with the weight of the rocks that make up the barb. Forces of the flows during design conditions, as well as buoyancy should be considered. In addition, the consequences of the woody material catching floating debris should be considered in the design and evaluation of its anchoring requirements. More information related to designing soil anchors is provided in NEH654 TS14E.

Figure TS14H–12  Rootwad used in the key of a stream barb
Finally, the designer should also consider how the placement of woody debris within the structure might also affect its hydraulics. Woody material should not be placed and aligned where it might direct flows into the bank.

**Design worksheet**

This section provides a generalized worksheet for designing a stream barb. The user is cautioned that, as with all stream projects, the design and placement of stream barbs are site specific. These listed steps will likely need to be modified and adjusted for specific projects. Figures TS14H–8, TS14H–10, and TS14H–12 will facilitate these steps.

**Step 1** Investigate site and obtain physical- and geomorphic-based parameters. The designer should determine if site is suitable for stream barbs.

Can *yes* be answered to the following questions:

- Is erosion occurring on the outside of a bend?
- Is the channel bed stable or quasi stable?
- Is the stream thalweg close to the eroding bank toe?
- Is this a natural channel (uncontrolled)?

If the answer is *yes* to all of the above questions, proceed.

**Step 2** Determine bankfull elevation, radius of outer bank, typical section, and hydraulic gradient. Develop a plan drawing of site from aerial photo or from survey information showing outer bank, bankfull line on opposite bank, on the eroding bank if it is significantly different than top of bank, and the thalweg. Locate beginning and ending points of the eroding bank. Using CAD or other methods, approximate the outer bank radius and bankfull width. If the radius varies significantly through eroded section of bend, determine the radius, width, and area at the beginning of erosion and at one or two other points that typify the stream curve.

From field survey and cross-sectional data, determine widths, radius, and area of bankfull discharge.

Radius of bend (R)

\[ R_1 = \ldots \]

\[ R_2 = \ldots \]

Bankfull width (W)

\[ W_1 = \ldots \]

\[ W_2 = \ldots \]

Bankfull area (A)

\[ A_1 = \ldots \]

\[ A_2 = \ldots \]

Determine the average depth

\[ D_a = \frac{\sum_{i} A_i W_i}{\sum_{i} W_i} \]

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 \( \theta \) for stream barb alignment. See the description, and use the guide below.

\[ \frac{R_i}{W_i} \]

- ≥ 3 If <3, consider other treatment
  - If <6, consider reduced angle, \( \theta \leq 30^\circ \)
  - If >6, \( \theta = 30^\circ \) to \( 45^\circ \) 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, \( \theta \) 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:

\[ L_e = L \times \sin \theta = \]

Check length: \[ \frac{W}{4} = \]

Is \( L_e \leq \frac{W}{4} \) ?

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.

\[ H = \frac{1}{3} D_s = \text{height of weir section, outer end} \]

\[ H = \frac{1}{2} D_s = \text{height of weir section, bank end} \]

\[ S = \left( \frac{1}{3} \text{ to } \frac{1}{2} \right) \times 2.5 \times D_s = \text{depth of bed key} \]

Step 10  Determine rock size per the description on rock size (TS14H–16).

Step 11  Prepare construction drawings. See figure TS14H–14, Typical construction drawing. Figure TS14H–15 shows a detail that illustrates one possibility of incorporating a rootwad into a rock stream barb.

Cost

The cost of rock stream barbs can vary considerably given availability of material, construction access, and permitting requirements. Stream barbs are often used in combination with other treatments. In general, their cost is between $2,000 and $5,000 per individual barb. Maintenance may involve replacement of materials. Monitoring should focus particularly on the area immediately below a series of stream barbs and the bank key.

Construction considerations

Instream devices like stream barbs are best constructed during low flow. Achieving a design key in depth may require dewatering, which may be accomplished with a cofferdam. If the designs include soil bioengineering or planting, either as part of the project or to stabilize the root or bank key, then appropriate planting designs also need to be considered. All stream or river design techniques should consider critical spawning and migration periods, as well as other regulatory concerns.

Conclusion

A variety of flow-changing techniques are applicable for use in stream design projects. They can provide valuable stability and habitat benefits. Stream barbs have been well received, and it is apparent these structures will continue to be a valuable tool for streambank restoration projects in NRCS. However, they do not work in all circumstances and must be designed to fit site-specific conditions.
Figure TS14H–13  Drawing and layout details

**Section Normal to Bank**

**Plan**

- **Bank key**
- **Weir section**
- **Natural ground**
- **Instream barb height**
- **Existing channel bottom**
- **Depth of bed key**
- **Bed key**
- **Top of bank**
- **Toe of bank**
- **Flow**
- **Length bank key**
- **Downstream edge of crest**
- **Project line**
- **Tangent line**
- **Tangent line**
- **2nd S.B.**
- **1st S.B.**
- **End of bank erosion**
- **Beginning bank erosion**
- **Design Layout**
- **Refers to step**

(210–VI–NEH, August 2007)
Figure TS14H–14  Typical stream barb construction drawing
Figure TS14H–15  Detail showing the use of a rootwad incorporated into a stream barb
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

<table>
<thead>
<tr>
<th>Purpose</th>
<th>TS14I–1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Introduction</td>
<td>TS14I–1</td>
</tr>
<tr>
<td>Benefits of streambank soil bioengineering</td>
<td>TS14I–2</td>
</tr>
<tr>
<td>Riparian areas</td>
<td>TS14I–3</td>
</tr>
<tr>
<td>Riparian planting zones</td>
<td>TS14I–4</td>
</tr>
<tr>
<td>Defining and managing risks</td>
<td>TS14I–6</td>
</tr>
<tr>
<td>Determining appropriateness of treatments</td>
<td>TS14I–7</td>
</tr>
<tr>
<td>Limiting velocity and shear criterion</td>
<td>TS14I–7</td>
</tr>
<tr>
<td>Plants for soil bioengineering</td>
<td>TS14I–11</td>
</tr>
<tr>
<td>Woody plants</td>
<td>TS14I–11</td>
</tr>
<tr>
<td>Herbaceous plants</td>
<td>TS14I–21</td>
</tr>
<tr>
<td>USDA NRCS Plant Materials Program: Plant development for stream-bank stabilization</td>
<td>TS14I–21</td>
</tr>
<tr>
<td>Purchasing plant materials</td>
<td>TS14I–28</td>
</tr>
<tr>
<td>Containerized plants</td>
<td>TS14I–28</td>
</tr>
<tr>
<td>Streambank soil bioengineering techniques</td>
<td>TS14I–31</td>
</tr>
<tr>
<td>Toe treatments</td>
<td>TS14I–31</td>
</tr>
<tr>
<td>Coir fascines</td>
<td>TS14I–31</td>
</tr>
<tr>
<td>Brush and tree revetments</td>
<td>TS14I–33</td>
</tr>
<tr>
<td>Rootwad revetments</td>
<td>TS14I–35</td>
</tr>
<tr>
<td>Brush spurs</td>
<td>TS14I–36</td>
</tr>
<tr>
<td>Live siltation</td>
<td>TS14I–38</td>
</tr>
<tr>
<td>Cribwall</td>
<td>TS14I–39</td>
</tr>
<tr>
<td>Fascines</td>
<td>TS14I–41</td>
</tr>
<tr>
<td>Bank treatments</td>
<td>TS14I–42</td>
</tr>
<tr>
<td>Live pole cuttings</td>
<td>TS14I–42</td>
</tr>
<tr>
<td>Dormant post planting</td>
<td>TS14I–44</td>
</tr>
<tr>
<td>Contour fascines</td>
<td>TS14I–46</td>
</tr>
<tr>
<td>Joint planting</td>
<td>TS14I–47</td>
</tr>
<tr>
<td>Brush layering</td>
<td>TS14I–48</td>
</tr>
<tr>
<td>Brush mattress</td>
<td>TS14I–50</td>
</tr>
<tr>
<td>Branch packing</td>
<td>TS14I–52</td>
</tr>
<tr>
<td>Vegetated reinforced soil slope</td>
<td>TS14I–53</td>
</tr>
<tr>
<td>Brush wattle fence</td>
<td>TS14I–56</td>
</tr>
<tr>
<td>Top of bank/flood plain treatments</td>
<td>TS14I–57</td>
</tr>
<tr>
<td>Brush trench</td>
<td>TS14I–57</td>
</tr>
<tr>
<td>Other techniques</td>
<td>TS14I–58</td>
</tr>
<tr>
<td>Wattle fence as an erosion stop</td>
<td>TS14I–58</td>
</tr>
<tr>
<td>Crimping and seeding</td>
<td>TS14I–59</td>
</tr>
</tbody>
</table>
Adjunctive measures ................................................................. TS14I–60
Erosion control .............................................................................. TS14I–60

**Integrating soil bioengineering and structural treatments** TS14I–60

**Soil bioengineering techniques for specific climate conditions** TS14I–62
Hot climate issues ........................................................................... TS14I–62
Cold climate issues ........................................................................ TS14I–62
High precipitation issues ............................................................. TS14I–64
Low precipitation issues ............................................................... TS14I–64

**Installation equipment and tips** TS14I–66
Dead blow hammer ........................................................................... TS14I–66
Stinger (metal) .................................................................................. TS14I–66
Waterjet hydrodrill ............................................................................ TS14I–67
Muddying-in ...................................................................................... TS14I–70
Holding ponds ................................................................................... TS14I–70
Sealing or marking paint ................................................................. TS14I–71
Construction scheduling ................................................................. TS14I–71
Plant protection ............................................................................... TS14I–71
Soil compaction .............................................................................. TS14I–73

**Planting plans** TS14I–73

**Conclusion** TS14I–76

---

**Tables**

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table TS14I–1</td>
<td>Design modifications to account for site conditions</td>
<td>TS14I–6</td>
</tr>
<tr>
<td>Table TS14I–2</td>
<td>Relationships between type of streambank stabilization project and type of site</td>
<td>TS14I–7</td>
</tr>
<tr>
<td>Table TS14I–3</td>
<td>Questions to ask before starting a streambank soil bioengineering project</td>
<td>TS14I–8</td>
</tr>
<tr>
<td>Table TS14I–4</td>
<td>Compiled permissible shear stress levels for streambank soil bioengineering practices</td>
<td>TS14I–10</td>
</tr>
<tr>
<td>Table TS14I–5</td>
<td>Woody plants with very good to excellent ability to root from dormant, unrooted cuttings and their soil bioengineering applications</td>
<td>TS14I–12</td>
</tr>
<tr>
<td>Table TS14I–6</td>
<td>Grasses, legumes, and forbs for soil bioengineering systems</td>
<td>TS14I–22</td>
</tr>
<tr>
<td>Table TS14I–7</td>
<td>Recommended spacing of fascines</td>
<td>TS14I–47</td>
</tr>
<tr>
<td>Table TS14I–8</td>
<td>Spacing for brush layers</td>
<td>TS14I–50</td>
</tr>
<tr>
<td>Figures</td>
<td>Description</td>
<td>Page</td>
</tr>
<tr>
<td>------------------</td>
<td>------------------------------------------------------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Figure TS14I–1</td>
<td>Riparian plant zones indicate where different riparian plant species should be planted</td>
<td>TS14I–5</td>
</tr>
<tr>
<td>Figure TS14I–2</td>
<td><em>Salix exigua</em> ssp. <em>exigua</em> (Coyote willow)</td>
<td>TS14I–18</td>
</tr>
<tr>
<td>Figure TS14I–3</td>
<td><em>Salix amygdaloides</em> (Peachtree willow)</td>
<td>TS14I–18</td>
</tr>
<tr>
<td>Figure TS14I–4</td>
<td><em>Cornus sericea</em> (Redosier dogwood)</td>
<td>TS14I–18</td>
</tr>
<tr>
<td>Figure TS14I–5</td>
<td><em>Populus balsamifera</em> ssp. <em>trichocarpa</em> (Black cottonwood)</td>
<td>TS14I–20</td>
</tr>
<tr>
<td>Figure TS14I–6</td>
<td><em>Populus angustifolia</em> (Narrowleaf cottonwood)</td>
<td>TS14I–20</td>
</tr>
<tr>
<td>Figure TS14I–7</td>
<td>USDA plant hardiness zone map and key</td>
<td>TS14I–29</td>
</tr>
<tr>
<td>Figure TS14I–8</td>
<td>Installation of coir fascines</td>
<td>TS14I–32</td>
</tr>
<tr>
<td>Figure TS14I–9</td>
<td>Stacked coir fascines using woody vegetation</td>
<td>TS14I–32</td>
</tr>
<tr>
<td>Figure TS14I–10</td>
<td>Brush/tree revetment over poles and a brush mattress</td>
<td>TS14I–33</td>
</tr>
<tr>
<td>Figure TS14I–11</td>
<td>Rootwads being installed over rock toe and with soil anchors</td>
<td>TS14I–35</td>
</tr>
<tr>
<td>Figure TS14I–12</td>
<td>Rootwad being pushed into the bank</td>
<td>TS14I–35</td>
</tr>
<tr>
<td>Figure TS14I–13</td>
<td>Brush spur being installed</td>
<td>TS14I–37</td>
</tr>
<tr>
<td>Figure TS14I–14</td>
<td>Brush spur after one growing season</td>
<td>TS14I–37</td>
</tr>
<tr>
<td>Figure TS14I–15</td>
<td>Live siltation construction or live brush sills</td>
<td>TS14I–38</td>
</tr>
<tr>
<td>Figure TS14I–16</td>
<td>Live siltation construction or live brush sills with rock</td>
<td>TS14I–38</td>
</tr>
<tr>
<td>Figure TS14I–17</td>
<td>(a) Live cribwall under construction; (b) After first growing season</td>
<td>TS14I–40</td>
</tr>
<tr>
<td>Figure TS14I–18</td>
<td>Assembling fascines</td>
<td>TS14I–41</td>
</tr>
<tr>
<td>Figure TS14I–19</td>
<td>Installation of live fascines combined with erosion control fabric</td>
<td>TS14I–41</td>
</tr>
<tr>
<td>Figure TS14I–20</td>
<td>Preparation of pole cuttings</td>
<td>TS14I–43</td>
</tr>
<tr>
<td>Figure TS14I–21</td>
<td>Iron punch bar being used to create pilot hole</td>
<td>TS14I–43</td>
</tr>
<tr>
<td>Figure TS14I–22</td>
<td>Live pole cuttings after one season</td>
<td>TS14I–43</td>
</tr>
</tbody>
</table>
Figure TS14I–23  Installation of dormant posts with stinger  TS14I–45
Figure TS14I–24  Fascines installed at an angle over a riprap toe  TS14I–46
Figure TS14I–25  (a) Completed installation of joint planting;  (b) Early in first growing season  TS14I–48
Figure TS14I–26  (a) Excavation of the brush layer bench;  (b) Cutting placement  TS14I–49
Figure TS14I–27  (a) Completed installation of brush layers;  (b) Results after two growing seasons  TS14I–49
Figure TS14I–28  (a) Brush mattress being installed; (b) Brush mattress after one growing season  TS14I–51
Figure TS14I–29  (a) Branch packing (using live poles) under construction;  (b) One growing season later  TS14I–53
Figure TS14I–30  Fill placement within VRSS  TS14I–54
Figure TS14I–31  Geogrid wrapping of soil lift  TS14I–54
Figure TS14I–32  Completed VRSS  TS14I–54
Figure TS14I–33  VRSS development after 4 years  TS14I–54
Figure TS14I–34  (a) Wattle fence immediately after construction; (b) 1 year later  TS14I–56
Figure TS14I–35  (a) Brush trench after installation; (b) 1 year later  TS14I–57
Figure TS14I–36  Brush wattle fence to deter erosion in a gully  TS14I–58
Figure TS14I–37  Crimped straw  TS14I–59
Figure TS14I–38  Cutting coir fabric  TS14I–61
Figure TS14I–39  Live cuttings installed in fabric  TS14I–61
Figure TS14I–40  Combining fascines and fabric  TS14I–61
Figure TS14I–41  U.S. annual precipitation map  TS14I–63
Figure TS14I–42  Stinger  TS14I–68
Figure TS14I–43  (a) Water jet nozzle; (b) Stinger  TS14I–69
Figure TS14I–44  (a) Water jet pump; (b) Equipment on trailer  TS14I–69
Figure TS14I–45  Cuttings with basal ends submerged in a pond  TS14I–70
Figure TS14I–46  Soaking willow cuttings at Fox Creek, Driggs, ID
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
Figure TS14I–48  (a) Tree cage built out of 6-foot-high horse fence; (b) Example of a tree protection sleeve
Figure TS14I–49  Illustration of expenditure profiles for soil bio- engineering and inert structures
Purpose

Streambank soil bioengineering is defined as the use of living and nonliving plant materials in combination with natural and synthetic support materials for slope stabilization, erosion reduction, and vegetative establishment. As a result of increased public understanding and greater appreciation of the environment, many Federal, state, and local governments, as well as grassroots organizations, are actively engaged in implementing soil bioengineering treatments to stabilize streambanks. Stabilizing streambanks through the integration of natural vegetation has many advantages over using hard armor linings alone. When compared to streams with little or no vegetation on their banks, streams with well-established perennial vegetation on their banks typically have higher economic value, better water quality, and better fish and wildlife habitats. A variety of vegetative techniques are in widespread use. Many of these include soil bioengineering practices. The value of vegetation in civil engineering and the role of woody vegetation in stabilizing streambanks have gained considerable recognition in recent years (Greenway 1987; Coppin and Richards 1990; Gray and Sotir 1996). However, streambank soil bioengineering is not universally applicable. There are important considerations to take into account for their successful application and long-term sustainability. This technical supplement provides guidance for the analysis, design, installation, and maintenance of some of the most effective and commonly used soil bioengineering techniques.

Introduction

Soil bioengineering is an integrated watershed-based technology that uses sound engineering practices in conjunction with integrated ecological principles to assess, design, construct, and maintain living vegetative systems. This technology can be applied to repair damage caused by erosion and failures in the land and protect or enhance already healthy, functioning systems (Gray and Sotir 1996). Streambank soil bioengineering uses plants as structural components to stabilize and reduce erosion on streambanks. When selecting the best-suited soil bioengineering techniques, it is important to have a clear understanding of the ecological systems of the adjacent areas. Plant selection and the techniques used will play an initial role in site stabilization and, ultimately, serve as the foundation for the ecological restoration of the site. The successful establishment and long-term sustainability of herbaceous and woody plants are extremely important to the physical and biological functions of the streams and the connected watershed system.

Streambank soil bioengineering has a long history with many milestones.

- Tapestries have been found in Chinese emperor’s tombs that depict Chinese peasants using willow bundles for streambank stabilization along the Yellow River in the year 28 B.C.
- In Europe, soil bioengineering techniques were used by Celtic villagers to create walls and fences.
- Romans used wattles and poles for hydro construction.
- The first written record of soil bioengineering was documented by Leonardo Da Vinci (1452–1519), where he recommended using rootable, living willow branches to stabilize agricultural irrigation channels, thus creating living streambanks. In the 16th century, streambank soil bioengineering treatments were used throughout Europe.
- In 1791, Woltmann published a soil bioengineering manual illustrating live stake techniques (Stiles 1991). In about 1800, soil bioengineers in Austria were using brush trenches to trap silt and reshape channels.
- In the 1900s, European soil bioengineers were using many of the treatments in use today (Stiles 1988).
- In 1934, Charles J. Kraebel, U.S. Forest Service, installed willow wattles above a road near Berkeley, California (Kraebel and Pillsbury 1934).
- In the late 1930s, the U.S. Department of Agriculture (USDA) Soil Conservation Service (SCS), now the Natural Resource Conservation Service (NRCS), began working on the Winooski River Watershed in Vermont after a succession of extremely damaging storm events. They used a series of soil bioengineering techniques...
such as fascines, brush dams, brush mattresses, and live stakes along the Winooski River streambanks. In 1995, a detailed study of the project was completed. More than 50 out of 92 demonstration sites are still functioning today. The study found that the most successful measures generally included a mix of vegetation and mechanical treatments at each site (USDA NRCS 1999a).

After World War II, the availability of cheap energy; surplus bulldozers and dump trucks; the high cost of labor; and the advent of cheap, well-designed steel and concrete structures caused hard, inflexible structures to take over from the soil-bioengineered structures as the preferred methods for treating streambank erosion. Over the past few decades, it has become apparent that these hard structures have inherent problems that have caused a breakdown of the riparian ecosystem because of their overuse and, often, inappropriate use. A movement back to flexible and more natural streambank soil bioengineering treatments offering broader functions has come from this realization, and so has begun the modern age of soil bioengineering.

Benefits of streambank soil bioengineering

Streambank soil bioengineering has aesthetic benefits. Streambank soil bioengineering provides improved landscape and habitat values (Lewis 2000). However, most designers are interested in the specific structural benefits provided by the vegetation. Gray (1977), Bailey and Copeland (1961), and Allen (1978) describe five mechanisms through which vegetation can aid erosion control:

- reinforce the soil through the plant roots
- dampen waves or dissipate wave energy
- intercept high-water velocities
- enhance water infiltration
- deplete water in the soil profile by uptake and transpiration

Klingeman and Bradley (1976) point out four specific ways vegetation can protect streambanks.

- The root system helps hold the soil together and increases the overall bank stability by its binding network structure, that is, the ability of roots to hold soil particles together.
- The exposed vegetation (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.
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.
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>
<thead>
<tr>
<th>Issue</th>
<th>Concern</th>
<th>Possible action or design modification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Duration of inundation</td>
<td>Some plant species and soils cannot withstand long flooding duration</td>
<td>• Choose plant materials that can withstand long inundation such as willow, dogwood, or elderberry, which can withstand 1 to 6 months of inundation&lt;br&gt;• Use or combine inert or soil reinforcement material in areas of prolonged inundation</td>
</tr>
<tr>
<td>Susceptibility of plant materials to disease or insects</td>
<td>Loss of plants could endanger the project</td>
<td>• 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&lt;br&gt;• Monitor the installation regularly for the first year or two during the establishment period&lt;br&gt;• Apply a fungicide or insecticide as needed to promote healthier growth</td>
</tr>
<tr>
<td>Excessive velocity</td>
<td>High velocities could damage or destroy the project</td>
<td>• Compare estimated velocity and/or shear thresholds at site to recommendations for limiting velocity and shear when selecting project type and method of repair</td>
</tr>
<tr>
<td>Increased resistance to flood levels</td>
<td>Increased roughness resulting from project may result in more frequent out-of-bank flows</td>
<td>• Choose plant material that remains supple. Avoid plant material that will be tree-like and form an obstruction to the flow&lt;br&gt;• Install the vegetative treatment further up on the bank&lt;br&gt;• Coordinate possible affects with flood plain regulatory authorities&lt;br&gt;• Excavate floodway to account for lost conveyance</td>
</tr>
<tr>
<td>Predation by herbivores</td>
<td>Loss of plants could endanger the project</td>
<td>• Fence the project area&lt;br&gt;• Fence planting areas within the project&lt;br&gt;• Surround the area with vegetation that the expected herbivores do not eat&lt;br&gt;• Choose plant material that they typically do not eat—thorny or otherwise unappetizing to the expected herbivores</td>
</tr>
</tbody>
</table>
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.

### 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>
<thead>
<tr>
<th>Site description</th>
<th>Tolerance for movement</th>
<th>Type of project</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eroding streambank threatening a home or municipal sewage treatment plant</td>
<td>None—streambank must be made static</td>
<td>Relies primarily on hard or inert structures, but may include a vegetative component for adjunctive support, environmental, and aesthetic benefits</td>
</tr>
<tr>
<td>Eroding streambank adjacent to a secondary road</td>
<td>Slight—road must be protected for moderate storms, but some movement is allowed</td>
<td>Rely on streambank soil bioengineering measures that incorporate hard or inert components</td>
</tr>
<tr>
<td>Eroding streambank threatening hiking trails in a park</td>
<td>Moderate—a natural system is desired, but movement should be slowed</td>
<td>May rely entirely on vegetative protection, but more likely on streambank soil bioengineering measures that incorporate some hard or inert components</td>
</tr>
<tr>
<td>Eroding streambank in rangeland</td>
<td>Relatively high—but erosion should be reduced</td>
<td>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</td>
</tr>
<tr>
<td>Erosion on a wild and scenic stream system</td>
<td>High—but erosion should be reduced</td>
<td>Do nothing or rely on plantings and vegetative streambank soil bioengineering measures</td>
</tr>
</tbody>
</table>

Table TS14I–2 Relationships between type of streambank stabilization project and type of site
<table>
<thead>
<tr>
<th>Question</th>
<th>Issue</th>
</tr>
</thead>
<tbody>
<tr>
<td>What is the land use conversion trend for the drainage area?</td>
<td>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.</td>
</tr>
<tr>
<td>Is a management plan in place and being maintained?</td>
<td>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.</td>
</tr>
<tr>
<td>Is the purpose of the streambank soil bioengineering project to protect critical structures such as a home, business, or manufacturing site?</td>
<td>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).</td>
</tr>
<tr>
<td>Are both sides of channel unstable?</td>
<td>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.</td>
</tr>
<tr>
<td>Is the channel grade stable?</td>
<td>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).</td>
</tr>
<tr>
<td>Is local scour on the bends an issue?</td>
<td>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).</td>
</tr>
<tr>
<td>What is the bank height?</td>
<td>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.</td>
</tr>
<tr>
<td>What is the velocity of the stream at design flows?</td>
<td>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.</td>
</tr>
<tr>
<td>What is the depth of the water?</td>
<td>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.</td>
</tr>
</tbody>
</table>
### Table TS14I–3  Questions to ask before starting a streambank soil bioengineering project—Continued

<table>
<thead>
<tr>
<th>Question</th>
<th>Issue</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Is a noncohesive soil layer present in the slope?</strong></td>
<td>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)</td>
</tr>
<tr>
<td><strong>Is bank instability due to piping or ground water sapping?</strong></td>
<td>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)</td>
</tr>
<tr>
<td><strong>Will mature vegetation adversely affect the stream hydraulics?</strong></td>
<td>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</td>
</tr>
<tr>
<td><strong>Is there a stable bank to tie into at each end of the treatment area?</strong></td>
<td>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</td>
</tr>
<tr>
<td><strong>What site conditions may inhibit plant growth?</strong></td>
<td>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</td>
</tr>
<tr>
<td><strong>Is there anything in the stream water or surface runoff that will inhibit plant growth?</strong></td>
<td>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</td>
</tr>
<tr>
<td><strong>Will the site be shaded during the growing season?</strong></td>
<td>If the site will be shaded, choose plant species that tolerate and thrive in shade conditions</td>
</tr>
<tr>
<td><strong>Is there significant surface runoff from above the streambank?</strong></td>
<td>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</td>
</tr>
<tr>
<td><strong>Are beaver, muskrats, moose, elk, or deer present in the area?</strong></td>
<td>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</td>
</tr>
<tr>
<td><strong>Are adequate plant materials available from the natural surrounding area or from local nurseries?</strong></td>
<td>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)</td>
</tr>
<tr>
<td><strong>Are invasive species present in the area?</strong></td>
<td>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</td>
</tr>
</tbody>
</table>
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.

\[ \tau_0 = \gamma R S_f \]  

(eq. TS14I–1)

where:

- \( \tau_0 \) = average boundary shear (lb/ft\(^2\))
- \( \gamma \) = 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>
<thead>
<tr>
<th>Practice</th>
<th>Permissible shear stress (lb/ft(^2))(^a)</th>
<th>Permissible velocity (ft/s)(^a)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Live poles (Depends on the length of the poles and nature of the soil)</td>
<td>Initial: 0.5 to 2 Established: 2 to 5+</td>
<td>Initial: 1 to 2.5 Established: 3 to 10</td>
</tr>
<tr>
<td>Live poles in woven coir TRM (Depends on installation and anchoring of coir)</td>
<td>Initial: 2 to 2.5 Established: 3 to 5+</td>
<td>Initial: 3 to 5 Established: 3 to 10</td>
</tr>
<tr>
<td>Live poles in riprap (joint planting) (Depends on riprap stability)</td>
<td>Initial: 3+ Established: 6 to 8+</td>
<td>Initial: 5 to 10+ Established: 12+</td>
</tr>
<tr>
<td>Live brush sills with rock (Depends on riprap stability)</td>
<td>Initial: 3+ Established: 6+</td>
<td>Initial: 5 to 10+ Established: 12+</td>
</tr>
<tr>
<td>Brush mattress (Depends on soil conditions and anchoring)</td>
<td>Initial: 0.4 to 4.2 Established: 2.8 to 8+</td>
<td>Initial: 3 to 4 Established: 10+</td>
</tr>
<tr>
<td>Live fascine (Very dependent on anchoring)</td>
<td>Initial: 1.2 to 3.1 Established: 1.4 to 3+</td>
<td>Initial: 5 to 8 Established: 8 to 10+</td>
</tr>
<tr>
<td>Brush layer/branch packing (Depends on soil conditions)</td>
<td>Initial: 0.2 to 1 Established: 2.9 to 6+</td>
<td>Initial: 2 to 4 Established: 10+</td>
</tr>
<tr>
<td>Live cribwall (Depends on nature of the fill (rock or earth), compaction and anchoring)</td>
<td>Initial: 2 to 4+ Established: 5 to 6+</td>
<td>Initial: 3 to 6 Established: 10 to 12</td>
</tr>
<tr>
<td>Vegetated reinforced soil slopes (VRSS) (Depends on soil conditions and anchoring)</td>
<td>Initial: 3 to 5 Established: 7+</td>
<td>Initial: 4 to 9 Established: 10+</td>
</tr>
<tr>
<td>Grass turf—bermudagrass, excellent stand (Depends on vegetation type and condition)</td>
<td>Established: 3.2</td>
<td>Established: 3 to 8</td>
</tr>
<tr>
<td>Live brush wattle fence (Depends on soil conditions and depth of stakes)</td>
<td>Initial: 0.2 to 2 Established: 1.0 to 5+</td>
<td>Initial: 1 to 2.5 Established: 3 to 10</td>
</tr>
<tr>
<td>Vertical bundles (Depends on bank conditions, anchoring, and vegetation)</td>
<td>Initial: 1.2 to 3 Established: 1.4 to 3+</td>
<td>Initial: 5 to 8 Established: 6 to 10+</td>
</tr>
</tbody>
</table>

\(^a\) (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)
Recommendations 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 leaved, and silky (fig. TS14I–4). All are multistemmed shrubs that are valu-
<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name and cultivars*</th>
<th>Procure from</th>
<th>Region of adaptation**</th>
<th>Rooting ability</th>
<th>Soil bioengineering technique</th>
</tr>
</thead>
<tbody>
<tr>
<td>Species with very good to excellent rooting ability from live hardwood material</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Populus balsamifera</em></td>
<td>Balsam poplar</td>
<td>Local collections</td>
<td>1,2,3,4,5,8,9,0,A</td>
<td>Very good</td>
<td>Live cuttings, poles</td>
</tr>
<tr>
<td><em>Populus deltoids</em></td>
<td>Eastern cottonwood</td>
<td>Local collections</td>
<td>1,2,3,4,5,6,7</td>
<td>Very good</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Populus balsamifera ssp trichocarpa</em></td>
<td>Black cottonwood</td>
<td>Local collections</td>
<td>4,8,9,0,A</td>
<td>Very good</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Salix alaxensis</em></td>
<td>Feltleaf willow</td>
<td>Local collections</td>
<td>A</td>
<td>Very good</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Salix amygdaloides</em></td>
<td>Peachleaf willow</td>
<td>Local collections</td>
<td>1,2,3,4,5,6,7,8,9</td>
<td>Very good</td>
<td>Poles, posts, live cuttings</td>
</tr>
<tr>
<td><em>Salix barclayi</em></td>
<td>Barclay's willow</td>
<td>Local collections</td>
<td>A</td>
<td>Very good</td>
<td>Poles, posts, live cuttings</td>
</tr>
<tr>
<td><em>Salix brachycarpa</em></td>
<td>Barren Ground willow</td>
<td>Local collections</td>
<td>A</td>
<td>Very good</td>
<td>Poles, posts, live cuttings</td>
</tr>
<tr>
<td><em>Salix boothii</em></td>
<td>Booth's willow</td>
<td>Local collections</td>
<td>7,8,9,0</td>
<td>Excellent</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix cottetii</em></td>
<td>Bankers' Dwarf willow (cultivar)</td>
<td>Nursery</td>
<td>Introduced 1,2,3</td>
<td>Very good</td>
<td>Fascines, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix discolor</em></td>
<td>Pussy willow</td>
<td>Local collections</td>
<td>1,2,3,4,9</td>
<td>Very good</td>
<td>Fascines, poles, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix drummondiana</em></td>
<td>Drummond's willow</td>
<td>Local collections</td>
<td>7,8,9,0</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix interior</em></td>
<td>‘Greenbank’ Sandbar willow (cultivar)</td>
<td>Nursery</td>
<td>1,3,4,5</td>
<td>Excellent</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix interior</em></td>
<td>Sandbar willow</td>
<td>Local collections</td>
<td>1,3,4,5</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix melanopsis</em></td>
<td>Coyote willow (green stem)</td>
<td>Local collections</td>
<td>8,9,0</td>
<td>Excellent</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix eriocephala</em></td>
<td>Missouri River willow</td>
<td>Local collections</td>
<td>7,8,9,0</td>
<td>Very good</td>
<td>Fascines, poles, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix flaviatilis</em></td>
<td>‘Multnomah’ River willow (cultivar)</td>
<td>Nursery</td>
<td>9 (Coast only)</td>
<td>Excellent</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix flaviatilis</em></td>
<td>River willow</td>
<td>Local collections</td>
<td>9</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix geyeriana</em></td>
<td>Geyer willow</td>
<td>Local collections</td>
<td>7,8,9,0</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix gooddingii</em></td>
<td>‘Goodding’s willow</td>
<td>Local collections</td>
<td>6,7,8,0</td>
<td>Very good</td>
<td>Poles, posts, live cuttings</td>
</tr>
<tr>
<td><em>Salix hookeriana</em></td>
<td>Clatsop’ Hooker willow (cultivar)</td>
<td>Nursery</td>
<td>9, 0 (Coast only)</td>
<td>Excellent</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
</tbody>
</table>
### Table TS14I–5
Woody plants with very good to excellent ability to root from dormant, unrooted cuttings and their soil bioengineering applications—Continued

<table>
<thead>
<tr>
<th>Scientific name and cultivars*</th>
<th>Procure from</th>
<th>Region of adaptation**</th>
<th>Rooting ability</th>
<th>Soil bioengineering technique</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Salix hookeriana</em> Hooker willow</td>
<td>Local collections</td>
<td>9,0</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix laevigata</em> Red willow</td>
<td>Local collections</td>
<td>7,8,9,0</td>
<td>Very good</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Salix lasiolepis</em> 'Rogue' Arroyo willow (cultivar)</td>
<td>Nursery</td>
<td>9,0</td>
<td>Excellent</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix lasiolepis</em> Arroyo willow</td>
<td>Local collections</td>
<td>6,7,8,9,0</td>
<td>Very good</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Salix leimonii</em> 'Palouse' Lemmon's willow (cultivar)</td>
<td>Nursery</td>
<td>8,9,0</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix leimonii</em> Lemmon's willow</td>
<td>Local collections</td>
<td>8,9,0</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix eriocephala</em> spp. <em>ligulifolia</em> 'Placer' Erect willow (cultivar)</td>
<td>Nursery</td>
<td>9,0 (Coast only)</td>
<td>Excellent</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix ligulifolia</em> Strapleaf willow</td>
<td>Local collections</td>
<td>8,9,0</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix lucida</em> ssp. <em>lasandra</em> 'Nehalem' Pacific willow (cultivar)</td>
<td>Nursery</td>
<td>9 (Coast only)</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix lucida</em> ssp. <em>lasandra</em> Pacific willow</td>
<td>Local collections</td>
<td>7,8,9,0,A</td>
<td>Excellent</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Salix pentandra</em> 'Aberdeen Selection' Laurel willow (cultivar)</td>
<td>Nursery</td>
<td>Introduced 8,9,0</td>
<td>Excellent</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Salix purpurea</em> 'Streamco' Purpleosier willow (cultivar)</td>
<td>Nursery</td>
<td>Introduced 1,2,3</td>
<td>Excellent</td>
<td>Fascines, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix sericea</em> 'Riverbend Germplasm' Silky willow (cultivar)</td>
<td>Nursery</td>
<td>1,2,3</td>
<td>Excellent</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix sericea</em> Silky willow</td>
<td>Local collections</td>
<td>1,2,3</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix sitchensis</em> 'Plumas' Sitka willow (cultivar)</td>
<td>Nursery</td>
<td>9,0 (Coast only)</td>
<td>Very good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Salix sitchensis</em> Sitka willow</td>
<td>Local collections</td>
<td>9,0,A</td>
<td>Very good</td>
<td>Fascines, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Sambucus nigra</em> ssp. <em>anadensis</em> Common elderberry</td>
<td>Local collections</td>
<td>1,2,3,4,5,6,8,0,A</td>
<td>Very good</td>
<td>Fascines, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td>Scientific name</td>
<td>Common name and cultivars*</td>
<td>Procure from</td>
<td>Region of adaptation**</td>
<td>Rooting ability</td>
</tr>
<tr>
<td>-----------------</td>
<td>---------------------------</td>
<td>--------------</td>
<td>------------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td><em>Baccharis pilularis</em></td>
<td>'Coyote' brush</td>
<td>Local collections</td>
<td>7,9,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Baccharis salicifolia</em></td>
<td>Mule’s Fat</td>
<td>Local collections</td>
<td>6,7,8,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Cephalanthus occidentalis</em></td>
<td>'Keystone' Common Button-bush (cultivar)</td>
<td>Nursery</td>
<td>1,2,3,5,6,7,0</td>
<td>Good</td>
</tr>
<tr>
<td><em>Cephalanthus occidentalis</em></td>
<td>Common buttonbush</td>
<td>Local collections</td>
<td>1,2,3,5,6,7,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Cornus amomum</em></td>
<td>'Indigo' Silky dogwood (cultivar)</td>
<td>Nursery</td>
<td>1,2,3,4,5,6</td>
<td>Good</td>
</tr>
<tr>
<td><em>Cornus amomum</em></td>
<td>Silky dogwood</td>
<td>Local collections</td>
<td>1,2,3,4,5,6</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Cornus sericea</em></td>
<td>'Ruby' Redosier dogwood (cultivar)</td>
<td>Nursery</td>
<td>1,3,4,5,7,8,9,0,A</td>
<td>Good</td>
</tr>
<tr>
<td><em>Cornus sericea</em></td>
<td>Redosier dogwood</td>
<td>Local collections</td>
<td>1,3,4,5,7,8,9,0,A</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Cornus sericea ssp. occidentalis</em></td>
<td>'Mason' Western Redosier dogwood (cultivar)</td>
<td>Nursery</td>
<td>9,0 (Coast only)</td>
<td>Good</td>
</tr>
<tr>
<td><em>Cornus sericea ssp. occidentalis</em></td>
<td>Western Redosier dogwood</td>
<td>Local collections</td>
<td>9,0,A</td>
<td>Good</td>
</tr>
<tr>
<td><em>Lonicera involucrate</em></td>
<td>Black Twinberry</td>
<td>Local collections</td>
<td>3,7,8,9,0,A</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Philadelphus lewisii</em></td>
<td>'Lewis' Mock-orange</td>
<td>Local collections</td>
<td>9,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Physocarpus capitatus</em></td>
<td>Pacific ninebark</td>
<td>Local collections</td>
<td>9 (Coast only)</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Physocarpus opulifolius</em></td>
<td>Common ninebark</td>
<td>Local collections</td>
<td>1,2,3,4,5</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Populus angustifolia</em></td>
<td>Narrowleaf cottonwood</td>
<td>Local collections</td>
<td>4,5,6,7,8,9,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Populus fremontii</em></td>
<td>Fremont cottonwood</td>
<td>Local collections</td>
<td>6,7,8,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Rubus spectabilis</em></td>
<td>Salmonberry</td>
<td>Local collections</td>
<td>8,9,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix alba</em></td>
<td>White willow</td>
<td>Local collections</td>
<td>introduced 1,2,3,4</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix bebbiana</em></td>
<td>Bebb willow</td>
<td>Local collections</td>
<td>1,3,4,5,7,8,9,0,A</td>
<td>Fair</td>
</tr>
<tr>
<td>Scientific name</td>
<td>Common name and cultivars*</td>
<td>Procure from</td>
<td>Region of adaptation**</td>
<td>Rooting ability</td>
</tr>
<tr>
<td>-----------------------</td>
<td>----------------------------</td>
<td>--------------</td>
<td>------------------------</td>
<td>----------------</td>
</tr>
<tr>
<td><em>Salix humilis</em></td>
<td>Prairie willow</td>
<td>Local collections</td>
<td>1,2,3,4,5,6</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix drummondiana</em></td>
<td>‘Curlew’ Drummond’s willow (cultivar)</td>
<td>Nursery</td>
<td>7,8,9,0</td>
<td>Good</td>
</tr>
<tr>
<td><em>Salix drummondiana</em></td>
<td>Drummond’s willow</td>
<td>Local collections</td>
<td>7,8,9,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix exigua</em></td>
<td>‘Silvar’ Coyote willow (cultivar)</td>
<td>Nursery</td>
<td>6,7,8,9,0,A</td>
<td>Good</td>
</tr>
<tr>
<td><em>Salix exigua</em> sp interior</td>
<td>Sandbar willow (grey stem)</td>
<td>Local collections</td>
<td>6,7,8,9</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix lucida</em></td>
<td>Shining willow</td>
<td>Local collections</td>
<td>1,3,4,5,7,8,9,0,A</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix lutea</em></td>
<td>Yellow willow</td>
<td>Local collections</td>
<td>4,5,7,8,9,0</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix nigra</em></td>
<td>Black willow</td>
<td>Local collections</td>
<td>1,2,3,5,6</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix planifolia</em></td>
<td>Plainleaf willow</td>
<td>Local collections</td>
<td>1,3,4,5,7,8,9,0,A</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Salix prolaxa</em></td>
<td>‘Rivar’ Mackenzie’s willow (cultivar)</td>
<td>Nursery</td>
<td>8,9,0,A</td>
<td>Good</td>
</tr>
<tr>
<td><em>Salix scouleriana</em></td>
<td>Mackenzie’s willow</td>
<td>Local collections</td>
<td>8,9,0,A</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Sambucus racemosa</em></td>
<td>Red elderberry</td>
<td>Local collections</td>
<td>9</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Spiraea douglasii</em></td>
<td>‘Bashaw’ Douglas Spirea (cultivar)</td>
<td>Nursery</td>
<td>9 (Coast only)</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Spiraea douglasii</em></td>
<td>Douglas Spirea</td>
<td>Local collections</td>
<td>0,9</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Symphoricarpos albus</em></td>
<td>Common snowberry</td>
<td>Local collections</td>
<td>9 (Coast only)</td>
<td>Fair</td>
</tr>
</tbody>
</table>
### Table TS14I–5
Woody plants with fair to good ability to root from dormant, unrooted cuttings and their soil bioengineering applications—Continued

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name and cultivars*</th>
<th>Procure from</th>
<th>Region of adaptation**</th>
<th>Rooting ability</th>
<th>Soil bioengineering technique</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Caribbean area</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><em>Batis maritima</em></td>
<td>Barilla, Saltwort</td>
<td>Local collections</td>
<td>C,H</td>
<td>Good</td>
<td>Brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Bucida buceras</em></td>
<td>úcar, gregre</td>
<td>Local collections</td>
<td>C</td>
<td>Fair</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Bursera simaruba</em></td>
<td>almácigo, turpentine tree</td>
<td>Local collections</td>
<td>C</td>
<td>Good</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Clusia rosea</em></td>
<td>Cupey</td>
<td>Local collections</td>
<td>C,H</td>
<td>Good</td>
<td>Pole, live cuttings</td>
</tr>
<tr>
<td><em>Commelina ssp.</em></td>
<td>Chifre</td>
<td>Local collections</td>
<td>C</td>
<td>Good</td>
<td>Brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Cordia sebestenea</em></td>
<td>Vomitel, geiger tree</td>
<td>Local collections</td>
<td>C,H</td>
<td>Fair</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Erythrina poeppigiana</em></td>
<td>Bucayo, bucare, mountain immortale</td>
<td>Local collections</td>
<td>C,H</td>
<td>Good</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Glyricidia sepium</em></td>
<td>Mata ratón, Glyricidia</td>
<td>Local collections</td>
<td>C,H</td>
<td>Good</td>
<td>Poles, live cuttings</td>
</tr>
<tr>
<td><em>Hibiscus spp.</em></td>
<td>Hibiscos</td>
<td>Local collections</td>
<td>C,H</td>
<td>Good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Hymenocallis caribaea</em></td>
<td>Lirio blanco, Spyder lily</td>
<td>Local collections</td>
<td>C</td>
<td>Fair</td>
<td>Brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Lagerstroemia indica</em></td>
<td>Astromelia</td>
<td>Local collections</td>
<td>C</td>
<td>Good</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Mangrove species</em></td>
<td>(Rhizophora, Avicenia, Conocarpus)</td>
<td>Local collections</td>
<td>C,H</td>
<td>Good</td>
<td>Pole, live cuttings</td>
</tr>
<tr>
<td><em>Nicolaia elatior</em></td>
<td>Flor de cera, Torch ginger</td>
<td>Local collections</td>
<td>C</td>
<td>Fair</td>
<td>Brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Pictetia acauleata</em></td>
<td>Fustic</td>
<td>Local collections</td>
<td>C</td>
<td>Fair</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Rhoeo spathacea</em></td>
<td>Sanguinaria</td>
<td>Local collections</td>
<td>C,H</td>
<td>Good</td>
<td>Brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Sansevieria hyacinthoides</em></td>
<td>Lengua de chucho, sweet Sansevieria</td>
<td>Local collections</td>
<td>C</td>
<td>Good</td>
<td>Brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Sphagneticola trilobata</em></td>
<td>Margarita, Bay Biscayne creeping oxeye</td>
<td>Local collections</td>
<td>C</td>
<td>Fair</td>
<td>Brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Zingiber spp.</em></td>
<td>Jengibre, Ginger</td>
<td>Local collections</td>
<td>C, H</td>
<td>Fair</td>
<td>Fascines, poles, brush mattress, brush layering, live cuttings</td>
</tr>
<tr>
<td><em>Cordyline terminalis</em></td>
<td>Ti</td>
<td>Local collections</td>
<td>H</td>
<td>Good</td>
<td>Live cuttings, poles</td>
</tr>
<tr>
<td><em>Polyscias guifoylei</em></td>
<td>Panax</td>
<td>Local collections</td>
<td>H</td>
<td>Good</td>
<td>Live cuttings, poles</td>
</tr>
</tbody>
</table>
Table TS14I–5  Woody plants with fair to good ability to root from dormant, unrooted cuttings and their soil bioengineering applications—Continued

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name and cultivars*</th>
<th>Procure from</th>
<th>Region of adaptation**</th>
<th>Rooting ability</th>
<th>Soil bioengineering technique</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Erythrina variegata</em></td>
<td>Tropic Coral’ Tall Erythrina (cultivar)</td>
<td>Nursery</td>
<td>H</td>
<td>Good</td>
<td>Live cuttings, poles</td>
</tr>
</tbody>
</table>

**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 leaved 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. *trichocarpa* (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>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name</th>
<th>Region/wetland status</th>
<th>Soil</th>
<th>Shade tolerance</th>
<th>Drought tolerance</th>
<th>Flood tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Ammophila breviligulata</em></td>
<td>American beachgrass</td>
<td>1, fac 2, upl 3, upl*</td>
<td>Sands</td>
<td>Poor</td>
<td>Fair</td>
<td>Poor</td>
</tr>
<tr>
<td><em>Andropogon gerardii</em></td>
<td>Big bluestem</td>
<td>1, fac 2, fac 3, fac 4, facu 5, fac 6, facu 7, fac 8, facu 9, facu</td>
<td>Loams</td>
<td>Poor</td>
<td>Good</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Beckmannia Syzigachne</em></td>
<td>Sloughgrass</td>
<td>1, obl 2, obl 3, obl 4, obl 5, obl 7, obl</td>
<td>Silt</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td><em>Calamagrostis canadensis</em></td>
<td>Blue-joint reedgrass</td>
<td>1, facw+ 2, obl 3, obl 4, facw+ 5, obl 7, obl 8, obl 9, facw+ 0, facw+</td>
<td>Silt</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td><em>Carex aquatilis</em></td>
<td>Water sedge</td>
<td>1, obl 2, obl 3, obl 4, obl 5, obl 7, obl 8, obl 9, obl 0, obl</td>
<td>Silt</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td><em>Carex nebrascensis</em></td>
<td>Nebraska sedge</td>
<td>3, obl 4, obl 5, obl 7, obl 8, obl 9, obl 0, obl</td>
<td>Silt</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td><em>Carex utriculata</em></td>
<td>Beaked sedge</td>
<td>A, obl</td>
<td>Silt</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
</tbody>
</table>
Table TS14I–6  Grasses, legumes, and forbs for soil bioengineering systems—Continued

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name</th>
<th>Region/wetland status</th>
<th>Soil</th>
<th>Shade tolerance</th>
<th>Drought tolerance</th>
<th>Flood tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Deschampsia caespitosa</em></td>
<td>Tufted hairgrass</td>
<td>1, facw 2, facw 3, facw+ 4, facw 7, facw- 8, facw 9, facw 0, facw A, facw</td>
<td>Loam</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td><em>Distichlis spicata</em> var. stricta*</td>
<td>Inland saltgrass</td>
<td>8, facw* 9, facw+ 0, facw*</td>
<td>Loam</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td><em>Eleocharis palustris</em></td>
<td>Creeping spikerush</td>
<td>1, obl 2, obl 3, obl 4, obl 5, obl 6, obl 7, obl 8, obl 9, obl 0, obl A, obl</td>
<td>Silt</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td><em>Elymus lanceolatus</em></td>
<td>Streambank wheatgrass</td>
<td>3, facu- 4, fac 5, fac 7, fac 8, upl 9, facu-A, upl</td>
<td>Loam</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Elymus virginicus</em></td>
<td>Wildrye</td>
<td>1, facw- 2, fac 3, facw- 4, fac 5, fac 6, fac 7, facw 8, facw 9, facw</td>
<td>Loams</td>
<td>Good</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td><em>Elytrigia elongate</em></td>
<td>Tall wheatgrass</td>
<td>3, fac 4, fac 5, fac 8, fac</td>
<td>Loam</td>
<td>Fair</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td>Scientific name</td>
<td>Common name</td>
<td>Region/wetland status</td>
<td>Soil</td>
<td>Shade tolerance</td>
<td>Drought tolerance</td>
<td>Flood tolerance</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>----------------------</td>
<td>-----------------------</td>
<td>--------</td>
<td>----------------</td>
<td>-------------------</td>
<td>-----------------</td>
</tr>
<tr>
<td>Elytrigia intermedia</td>
<td>Intermediate wheatgrass</td>
<td>2, facu</td>
<td>Loam</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3, facu</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Eragrostis trichodes</td>
<td>Sand lovegrass</td>
<td>1, facu</td>
<td>Sands</td>
<td>Poor</td>
<td>Good</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2, facu+</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>3, face-</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>5, fac</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6, fac*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>7, facw-</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>8, fac</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>9, fac</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0, fac</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>A, fac</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hemarthria altissima</td>
<td>Limpograss</td>
<td>1-9, obl</td>
<td>Silt</td>
<td>Fair</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0, A, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glyceria striata</td>
<td>Mannagrass</td>
<td>1-9, obl</td>
<td>Silt</td>
<td>Fair</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td>Juncus balticus</td>
<td>Baltic rush</td>
<td>1, facw+</td>
<td>Silt</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>4, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>5, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>7, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>8, facw</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>9, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>A, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Juncus mertensianus</td>
<td>Merten's rush</td>
<td>7, obl</td>
<td>Silt</td>
<td>Fair</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8, obl*</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>9, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>A, obl</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Scientific name</td>
<td>Common name</td>
<td>Region/wetland status</td>
<td>Soil</td>
<td>Shade tolerance</td>
<td>Drought tolerance</td>
<td>Flood tolerance</td>
</tr>
<tr>
<td>-----------------</td>
<td>-------------</td>
<td>-----------------------</td>
<td>------</td>
<td>-----------------</td>
<td>------------------</td>
<td>----------------</td>
</tr>
<tr>
<td><em>Juncus tenuis</em></td>
<td>Poverty rush</td>
<td>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+</td>
<td>Silt</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Panicum</em></td>
<td>Coastal</td>
<td>1, facu- 2, fac</td>
<td>Sands to loams</td>
<td>Poor</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td><em>Amarulum</em></td>
<td>Panicgrass</td>
<td>6, facu-</td>
<td>Loams to sands</td>
<td>Poor</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td><em>Panicum virgatum</em></td>
<td>Switchgrass</td>
<td>1, fac 2, fac+ 3, fac+ 4, fac 5, fac 6, facw 7, fac+ 8, fac 9, fac+ H, upl</td>
<td>Loam</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td><em>Pascopyrum smithii</em></td>
<td>Western wheatgrass</td>
<td>1, upl 2, facu 3, facu+ 4, facu 5, facu 6, fac- 7, fac- 8, facu 9, facu 0, fac- A, upl</td>
<td>Loam</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td><em>Paspalum vaginatum</em></td>
<td>Seashore paspalum</td>
<td>2, obl 6, facw* C, obl H, facw+</td>
<td>Sandy</td>
<td>Poor</td>
<td>Good</td>
<td></td>
</tr>
</tbody>
</table>
### Table TS14I–6
Grasses, legumes, and forbs for soil bioengineering systems—Continued

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name</th>
<th>Region/wetland status</th>
<th>Soil</th>
<th>Shade tolerance</th>
<th>Drought tolerance</th>
<th>Flood tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Puccinellia nuttalliana</em></td>
<td>Alkaligrass</td>
<td>1, fac 3, obl 4, obl 5, obl 7, obl 8, obl 9, obl 0, obl A, facw</td>
<td>Loam</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td><em>Schizachyrium scoparium</em></td>
<td>Little bluestem</td>
<td>1, facu- 2, facu 3, facu- 4, facu 5, facu 6, facu+ 7, facu 8, facu 9, facu</td>
<td>Sands to loams</td>
<td>Poor</td>
<td>Good</td>
<td>Poor</td>
</tr>
<tr>
<td><em>Scirpus acutus</em></td>
<td>Hard-stem bulrush</td>
<td>1, obl 2, obl 3, obl 4, obl 5, obl 6, obl 7, obl 8, obl 9, obl 0, obl</td>
<td>Silt</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
<tr>
<td><em>Scirpus maritimus</em></td>
<td>Cosmopolitan bulrush</td>
<td>1, obl 9, obl 0, obl H, obl</td>
<td>Silt</td>
<td>Good</td>
<td>Good</td>
<td>Good</td>
</tr>
<tr>
<td><em>Scirpus pungens</em></td>
<td>Common three-square</td>
<td>1, facw+ 2, obl 4, obl 5, obl 6, obl 7, obl 8, obl 9, obl 0, obl</td>
<td>Silt</td>
<td>Fair</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td><em>Sorghastrum nutans</em></td>
<td>Indiangrass</td>
<td>1, upl 2, facu 3, facu+ 4, facu 5, facu 6, facu 7, upl 8, facw</td>
<td>Sands to loam</td>
<td>Fair</td>
<td>Fair</td>
<td>Poor</td>
</tr>
</tbody>
</table>
### Table TS14I–6
Grasses, legumes, and forbs for soil bioengineering systems—Continued

<table>
<thead>
<tr>
<th>Scientific name</th>
<th>Common name</th>
<th>Region/wetland status</th>
<th>Soil</th>
<th>Shade tolerance</th>
<th>Drought tolerance</th>
<th>Flood tolerance</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>Spartina pectinata</em></td>
<td>Prairie cordgrass</td>
<td>1, obl 2, obl 3, facw+ 4, facw 5, facw 6, facw+ 7, facw 8, obl 9, obl</td>
<td>Silt</td>
<td>Fair</td>
<td>Fair</td>
<td>Good</td>
</tr>
<tr>
<td><em>Verbena hastate</em></td>
<td>Blue vervain</td>
<td>1, facw+ 2, fac 3, facw+ 4, facw 5, facw 6, facw+ 7, facw 8, facw 9, facw+ 0, facw</td>
<td>Loam</td>
<td>Fair</td>
<td>Fair</td>
<td>Fair</td>
</tr>
<tr>
<td><em>Zizaniopsis miliacea</em></td>
<td>Giant cutgrass</td>
<td>1, obl 2, obl 3, obl 6, obl</td>
<td>Loam</td>
<td>Poor</td>
<td>Poor</td>
<td>Good</td>
</tr>
</tbody>
</table>

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 them 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 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
### USDA Plant Hardiness Zone Map

<table>
<thead>
<tr>
<th>USDA zone</th>
<th>Temperature range</th>
<th>Example cities</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Below -50 F (-45.6 C)</td>
<td>Fairbanks, Alaska, Resolute, Northwest Territories (Canada)</td>
</tr>
<tr>
<td>2a</td>
<td>-50 to -45 F (-42.8 to -45.5 C)</td>
<td>Prudhoe Bay, Alaska, Flin Flon, Manitoba (Canada)</td>
</tr>
<tr>
<td>2b</td>
<td>-45 to -40 F (-40.0 to -42.7 C)</td>
<td>Unalakleet, Alaska, Pinecreek, Minnesota</td>
</tr>
<tr>
<td>3a</td>
<td>-40 to -35 F (-37.3 to -39.9 C)</td>
<td>International Falls, Minnesota, St. Michael, Alaska</td>
</tr>
<tr>
<td>3b</td>
<td>-35 to -30 F (-34.5 to -37.2 C)</td>
<td>Tomahawk, Wisconsin, Sidney, Montana</td>
</tr>
<tr>
<td>4a</td>
<td>-30 to -25 F (-31.7 to -34.4 C)</td>
<td>Minneapolis/St.Paul, Minnesota, Lewistown, Montana</td>
</tr>
<tr>
<td>4b</td>
<td>-25 to -20 F (-28.9 to -31.6 C)</td>
<td>Northwood, Iowa, Nebraska</td>
</tr>
<tr>
<td>5a</td>
<td>-20 to -15 F (-26.2 to -28.8 C)</td>
<td>Des Moines, Iowa, Illinois</td>
</tr>
<tr>
<td>5b</td>
<td>-15 to -10 F (-23.4 to -26.1 C)</td>
<td>Columbia, Missouri, Mansfield, Pennsylvania</td>
</tr>
<tr>
<td>6a</td>
<td>-10 to -5 F (-20.6 to -23.3 C)</td>
<td>St. Louis, Missouri, Lebanon, Pennsylvania</td>
</tr>
<tr>
<td>6b</td>
<td>-5 to 0 F (-17.8 to -20.5 C)</td>
<td>McMinnville, Tennessee, Branson, Missouri</td>
</tr>
<tr>
<td>7a</td>
<td>0 to 5 F (-15.0 to -17.7 C)</td>
<td>Oklahoma City, Oklahoma, South Boston, Virginia</td>
</tr>
<tr>
<td>7b</td>
<td>5 to 10 F (-12.3 to -14.9 C)</td>
<td>Little Rock, Arkansas, Griffin, Georgia</td>
</tr>
<tr>
<td>8a</td>
<td>10 to 15 F (-9.5 to -12.2 C)</td>
<td>Tifton, Georgia, Dallas, Texas</td>
</tr>
<tr>
<td>8b</td>
<td>15 to 20 F (-6.7 to -9.4 C)</td>
<td>Austin, Texas, Gainesville, Florida</td>
</tr>
<tr>
<td>9a</td>
<td>20 to 25 F (-3.9 to -6.6 C)</td>
<td>Houston, Texas, St. Augustine, Florida</td>
</tr>
<tr>
<td>9b</td>
<td>25 to 30 F (-1.2 to -3.8 C)</td>
<td>Brownsville, Texas, Fort Pierce, Florida</td>
</tr>
<tr>
<td>10a</td>
<td>30 to 35 F (1.6 to -1.1 C)</td>
<td>Naples, Florida, Barstow, California</td>
</tr>
<tr>
<td>10b</td>
<td>35 to 40 F (4.4 to 1.7 C)</td>
<td>Miami, Florida, Coral Gables, Florida</td>
</tr>
<tr>
<td>11</td>
<td>above 40 F (4.5 C)</td>
<td>Honolulu, Hawaii, Mazatlan, Mexico</td>
</tr>
</tbody>
</table>

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 Robin B. Sotir & Associates, Inc.*

(a) ![Image](image1)

(b) ![Image](image2)

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...
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 matress, 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.

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
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.
• 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.
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
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–18**  
Assembling fascines

**Figure TS14I–19**  
Installation of live fascines combined with erosion control fabric (Photo courtesy of Robbin B. Sotir & Associates, Inc.)
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, non-galvanized 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).

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.

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 semi-arid areas where there is uncertainty in determining the water table.
**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.

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 handle, 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

Materials
• Live cuttings—3/4 to 2 inches in diameter, 5 to 15 feet long

• Ties—cord, braided manila, sisal or pre-stretched 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
- 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 (downtown) 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>
<thead>
<tr>
<th>Bank H:V</th>
<th>Bank distance between trenches in feet</th>
<th>Maximum bank length in feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>1:1 to 1.5:1</td>
<td>3–4</td>
<td>15</td>
</tr>
<tr>
<td>1.5:1 to 2:1</td>
<td>4–5</td>
<td>20</td>
</tr>
<tr>
<td>2:1 to 2.5:1</td>
<td>5–6</td>
<td>30</td>
</tr>
<tr>
<td>2.5:1 to 3:1</td>
<td>6–8</td>
<td>40</td>
</tr>
<tr>
<td>3:1 to 4:1</td>
<td>8–9</td>
<td>50</td>
</tr>
<tr>
<td>4:1 to 5:1</td>
<td>9–10</td>
<td>60</td>
</tr>
</tbody>
</table>

(210–VI–NEH, August 2007)
**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.*)
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 pre-stretched 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

<table>
<thead>
<tr>
<th>Bank H:V</th>
<th>Bank distance (ft) between benches for wet slopes</th>
<th>Bank distance (ft) between benches for dry slopes</th>
<th>Maximum bank length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2:1 to 2.5:1</td>
<td>3</td>
<td>3</td>
<td>15</td>
</tr>
<tr>
<td>2.5:1 to 3.5:1</td>
<td>3</td>
<td>4</td>
<td>15</td>
</tr>
<tr>
<td>3.5:1 to 4:1</td>
<td>4</td>
<td>5</td>
<td>20</td>
</tr>
</tbody>
</table>

Figure TS14I–28  (a) Brush mattress being installed; (b) Brush mattress after one growing season
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
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
• Install additional live cuttings at 1- to 2-foot intervals along the proposed location of the wattle wall. If the wattling materials available do not flex easily, the distance between the live cuttings can be adjusted accordingly.

• Weave the flexible wattling material between the live cuttings in a basket-like fashion. To begin, take a length of wattling material and weave it horizontally to the left of the first live cutting, then to the right of the second live cutting, then to the left of the next. Do this over the entire length of the wattle wall. Gently compress, so this layer makes contact with the ground. Begin the next layer by weaving the wattling material in the opposite direction; begin by weaving the material to the right of the first live cutting, then to the left of the second, and so on. Once the second layer is woven, gently compress so it touches the first layer, ensuring a tight weave. Continue weaving in this manner up to the top of the wattle wall.

• Backfill the trench and tamp the soil. After installation, the area should be watered. Supplemental irrigation may also be required as the plants become established.

Top of bank/flood plain treatments

Brush trench
A brush trench is a row of live cuttings that is inserted into a trench along the top of an eroding streambank parallel to the stream (figs. TS14I–35). The live cuttings form a fence that filters runoff and reduces the likelihood of rilling. The live cuttings eventually root and provide a permanent living structure. Brush trenches are often used to supplement other soil bioengineering treatments.

Materials

• Live cuttings—3/4 to 3 inches in diameter, 2.5 to 5 feet long
• Tools—machete, clippers, shovel, saw, hammer, and excavator
• Fertilizer and other soil amendments

Installation

• Collect and soak the live cuttings for 14 days or install them the day they are harvested. Leave the side branches intact. It is important to select low-growing species that will remain supple.
• Install appropriate bank and toe protection prior to excavating the brush trench.

• If a moderate amount of runoff currently flows over the bank, consider using a low berm along the top of the bank and directing the flow to a stable outfall away from the bank.

• Excavate a trench that is 10 to 12 inches wide and 1 to 2 feet deep. The trench should be no less than 1 foot back from the top of the bank so that it does not weaken the bank.

• Pack the branches tightly with the basal ends down, forming an intertwined mat. Make sure that the basal ends touch the bottom of the trench. Install 8 to 15 live cuttings per linear foot of trench. The branches protruding from the top of the trench should be taller than the height of competing vegetation.

• Avoid gaps in the vegetation.

• Fill in around the live cuttings with soil, then wash in to assure good soil-to-stem contact. All gaps between the plant materials within the trench should be filled with soil.

• Cut off the terminal end or buds to promote root growth. After the installation is completed, water the entire area. Supplemental irrigation may also be required as the vegetation becomes established.

Other techniques

Wattle fence as an erosion stop
A wattle fence can be used to deter erosion in ditches or small dry channel beds to resist the formation of rills and gullies (fig. TS14I–36). Wattle fences are rows of live stakes or poles with live cuttings woven in a basket-like fashion. The live cuttings eventually root and provide a permanent living structure. During planning and selection of wattling material, consider the potential for excessive growth clogging the channel. The planted material should be of a species that will remain supple. This treatment is not typically suitable for areas with high velocities, prolonged inundation, or significant headcuts.

Materials

• Live cuttings—2 to 4 inches in diameter and 2 to 3 feet long

• Wattling materials—flexible branches that are 3/4 to 1 inch in diameter and 4 to 10 feet long

• Tools—machete, shovels, hammer, punch bar, clippers, and saw

• Fertilizer and other soil amendments

Installation

• Collect and soak the live cuttings for 14 days, or install them the day they are harvested. Leave the side branches intact.

• Excavate a trench 1 to 2 feet deep across the dry channel or ditch.

• The trench should be keyed-in or extended 1 to 3 feet into the sides of the channel or ditch and should curve upslope.

• Use a punch bar or stake to create a pilot hole at the base of the trench. The pilot hole should have a minimum depth of 1 foot below the invert of the trench.

• Tap the live cutting into the ground so that the sharpened basal end is inserted first. Approximately two-thirds to three-fourths of the live cutting should be below the top of the trench. In addition, the top of the stakes should not be higher than one-third of the channel depth.

Figure TS14I–36 Brush wattle fence to deter erosion in a gully
- Fill the hole with water and soil slurry. Tamp the ground around the live cutting.
- Install additional live cuttings at 1- to 2-foot intervals along the proposed location of the wattle wall. If the wattling materials available do not flex easily, the distance between the live cuttings can be adjusted accordingly.
- Weave the flexible wattling material between the live cuttings in a basket-like fashion. To begin, take a length of wattling material and weave it horizontally to the left of the first live cutting, then to the right of the second live cutting, then to the left of the next. Do this over the entire length of the wattle wall. Gently compress so this layer makes contact with the ground. Begin the next layer by weaving the wattling material in the opposite direction. Begin by weaving the material to the right of the first live cutting, then to the left of the second, and so on. Once the second layer is woven, gently compress so it touches the first layer, ensuring a tight weave. Continue weaving in this manner up to the top of the wattle wall.
- The center of the wattle should be lower than the sides to reduce the likelihood of bank erosion. The sides of the wattling should be keyed-in 1 to 3 feet into the sides of the ditch or channel.
- Backfill the trench and tamp the soil. After the installation is completed, water the entire area. Supplemental irrigation may also be required as the vegetation becomes established.
- Key stones into the bed of the channel below the wattle structure for a minimum length of two times the height of the structure.

**Crimping and seeding**

Crimping is a surface roughening treatment that secures straw to the surface (fig. TS14I–37). It is a temporary surface treatment that protects and promotes the establishment of permanent grasses and vegetation. It can be accomplished with heavy equipment or by hand.

**Materials**

- Straw—avoid moldy or compacted straw
- Seeds
- Tools—shovels

**Installation**

- Determine approximate contour lines for installation along the slope. The contour lines should be separated by 2 to 3 feet.
- Push the shovel into the ground along the contour lines to a depth of approximately 8 inches. With the shovel still in the ground, push the shovel forward then pull it backwards to make a V-shaped indentation in the soil.
- Distribute straw along the tops of the V-shaped indentations.
- Using the shovel, push the straw into the V-shaped indentations leaving 1 to 3 inches of straw protruding above the ground surface.
- Tamp the ground by foot to close the indentation around the straw.
- Seed the area and water it.
- Place a 2- to 4-inch layer of straw between the contours. Supplemental irrigation may be required.
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, desiccation, 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 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, gullyng 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

(210–VI–NEH, August 2007)
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\textsuperscript{®}. 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 vegetation 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

(210–VI–NEH, August 2007)
Figure TS14I–42  Stinger
Figure TS14I–43  (a) Water jet nozzle; (b) Stinger

(a) Water jet nozzle; (b) Stinger

Figure TS14I–44  (a) Water jet pump; (b) Equipment on trailer

(a) Water jet pump; (b) Equipment on trailer
drilling. The more sediment that is allowed to bubble out, the more sediment that will have to be replaced after the water moves into the surrounding soil. The cuttings should be pushed into the hole immediately after it has been jetted, to avoid having it collapse or fill with sediment. The waterjet does not work well in large gravels and cobbles. It works best in fine-textured soils. It will work in sands if the cutting is pushed into the ground at the same time as the probe.

**Muddying-in**

Good soil-to-stem contact is critical for successful establishment of most dormant unrooted cuttings. This can be achieved by muddying them in. Muddying-in the cuttings means pouring a slurry mix of water and soil into the hole around the cutting stem. The slurry mix will flow around the cutting, completely displacing any air pockets and creating good soil-to-stem contact. As the water percolates into the surround soil, the soil that is in the slurry will settle tightly around the stem, improving rooting success. Using the waterjet can accomplish the same thing.

**Holding ponds**

All dormant cuttings benefit from being soaked prior to installation. Ideally, this will be for 14 days. The soaking process can occur in an existing pond, backwater zone of a river, a small plastic-lined pond, or anything that will hold water. The goal is to fully submerge the dormant cuttings. Soaking the cuttings allows the plant tissues to fully hydrate. It also causes the root buds to start growing. The roots will emerge from the bark in about 14 days. The tender emerged roots will rub off when the cuttings are planted, so remove the cuttings from the water just before the roots emerge. Once planted, the cuttings will root into the soil much faster than they would if they have to absorb water from the surround soil (fig. TS14I–45).

If the entire cutting is soaked in a cold water pond, preferably in shady conditions, it can prolong the dormant period, allowing a project to proceed effectively even when the construction schedule is lagging. The entire cutting should be submerged while soaking especially as the air temperature rises. Lastly, placing the cuttings in a holding pond facilitates inventorying of plant materials to be assembled and safely stored, allowing an efficient and uninterrupted process of collection and installation (NEH654 TS14I–46).
Sealing or marking paint

Once dormant cuttings are collected, it is difficult for most people to accurately identify one species of plant from another and keep track of which end is up on a cutting. One of the simplest measures for addressing this need is to have various colors of marking paint available for coding plants and tagging the top (shoot) end of the cuttings (NEH654 TS14I).

Bundle the cuttings, and tap the tops on the ground to ensure the tops are even. Pour about 1 inch of a mix of 50 percent water and 50 percent latex paint into a flat pan or bucket. Dip the top inch of the bundle briefly into the paint/water mix, and stack the bundles to dry. This treatment does not harm the plants. Its purpose is to prevent desiccation and make it easier to identify the species once they are planted. In addition, it reduces the chance that the live cuttings will be planted upside down.

Painting the tops of the cuttings is handy for a construction inspector who can far more readily spot the cuttings, and it helps the crews keep track of where to plant them. It also is indispensable in monitoring to assess how well the plantings have developed.

Construction scheduling

Many of the soil bioengineering treatments outlined here depend on installation of inert elements, as well as plant materials. To accommodate the inevitable delays in construction project scheduling, it is useful to realize that the structural phase can often be done first, during a season that is not amenable to planting, and the planting can be scheduled immediately thereafter or added the following year.

Plant protection

One benefit of soil bioengineering treatments is their ability to provide habitat by serving as a food source, but too much of a good thing can be destructive to the success of the project.

Newly planted shrubs along the riverbank may be grazed by deer and other herbivores. This becomes an issue in harsh climate, such as cold, dry climates, where vegetation is scarce. Although most stands of plants can tolerate being trimmed down to the ground once a year (assuming they have initially rooted and become established onsite), continual grazing pressure may exceed the ability of plants to maintain the health of their root system and regenerate.

There are a number of different options to protect the cuttings until they have rooted or to protect mature riparian woody plants. Often, fencing that surrounds either the entire planting, or sections of it, is the best solution. After 3 to 5 years, the plants no longer require that protection. Similarly, tree plantings in riparian corridors frequently come under pressure from deer or small mammals that eat their bark at the snow level. Wire rabbit mesh or commercially available plant protection sleeves can prevent this damage (fig. TS14I–48).

Particular care and attention should be exercised when using tall plant protection sleeves in very hot areas because they can act as super hot greenhouses that will cook the tender plants inside. In areas that have moose and elk, 6-foot-high horse fence that is tied into 3- to 4-foot circles and placed around the trees will prevent them from putting their heads over the top of the fencing and eating the apical bud. The circle should be wide enough to prevent wildlife from eating any branches that are close to the sides.

Figure TS14I–46  Soaking willow cuttings at Fox Creek, Driggs, ID
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

Figure TS14I–48  
(a) Tree cage built out of 6-foot-high horse fence; (b) Example of a tree protection sleeve
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-
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.
Use of Large Woody Material for Habitat and Bank Protection
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.
The Use of Large Woody Material for Habitat and Bank Protection

Contents

Introduction TS14J–1

Area of applicability TS14J–1

Environmental and habitat considerations TS14J–1

Design TS14J–4

Types of LWM structures.......................................................... TS14J–4
Selecting a type of structure..................................................... TS14J–4
Dimensions for intermittent LWM structures................................. TS14J–6
Force and moment analysis.......................................................... TS14J–6
Ballast and anchoring.................................................................. TS14J–9
Materials..................................................................................... TS14J–10
Cost.............................................................................................. TS14J–12
Maintenance ................................................................................ TS14J–12

Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table TS14J–1</td>
<td>Limitations on applicability of large wood structures</td>
<td>TS14J–2</td>
</tr>
<tr>
<td>Table TS14J–2</td>
<td>Published values for design variables for LWM structures</td>
<td>TS14J–3</td>
</tr>
<tr>
<td>Table TS14J–3</td>
<td>Classification of large wood instream structures</td>
<td>TS14J–5</td>
</tr>
<tr>
<td>Table TS14J–4</td>
<td>Comparison of desirability of various tree species for stream structures</td>
<td>TS14J–11</td>
</tr>
<tr>
<td>Table TS14J–5</td>
<td>Reported costs for stream stabilization and habitat enhancement structures</td>
<td>TS14J–13</td>
</tr>
</tbody>
</table>

Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS14J–1</td>
<td>Large historical logjams of large woody material, Great Raft, Red River, LA</td>
<td>TS14J–3</td>
</tr>
<tr>
<td>Figure TS14J–2</td>
<td>White River, IN, with large woody debris</td>
<td>TS14J–4</td>
</tr>
<tr>
<td>Figure TS14J–3</td>
<td>Definition sketch for geotechnical forces on buried log</td>
<td>TS14J–8</td>
</tr>
</tbody>
</table>
Introduction

Large woody materials (LWM) have been used for river training and stabilization for centuries. Many of the earliest river training structures built on large rivers in the United States included willow mattresses, brush mattresses, or wooden pilings driven into the bed. More recent efforts include tree revetments and other structures featuring large wood that were placed in the Winooski River, Vermont, in the 1930s, as part of a successful comprehensive watershed stabilization project (Edminster, Atkinson, and McIntyre 1949; U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) 1999a). A wide-ranging federally funded streambank protection research and demonstration program in the 1970s included several field trials of LWM-based protection schemes (U.S. Army Corps of Engineers (USACE) 1981). Most of these installations produced favorable short-term results for erosion control and in terms of costs, although some projects were damaged by ice (Henderson 1986).

In the 1970s, George Palmiter developed a suite of techniques involving repositioning LWM for controlling erosion and high-frequency flooding along low-gradient, medium-sized rivers clogged with debris and sediment. His approach featured use of hand tools and small power equipment (Institute of Environmental Sciences, Miami University, 1982; National Research Council 1992). A 1986 evaluation of 137 log habitat structures in the Northwest revealed high rates of damage and failure (Frissell and Nawa 1992).

During the 1990s, increasing appreciation of the importance of large wood in natural riverine ecosystems triggered efforts to design structures that emulated the form and function of naturally occurring, stable accumulations of wood, particularly in rivers of the Pacific Northwest (Abbe, Montgomery, and Petroff 1997; Hilderbrand et al. 1998). However, rootwad composites, which are currently among the most popular types of large wood structures, do not resemble any commonly observed large wood formations.

Area of applicability

LWM structures are intended to provide habitat and stabilization until woody riparian vegetation and stable bank slopes can be established. LWM decays within a few years unless it is continuously submerged. Therefore, structures made entirely or partially of woody materials are not suited for long-term stabilization unless wood is preserved by continuous wetting or with chemicals. Woody structures are best applied to channels that are at least moderately stable, have gravel or with finer bed material, and need wood for habitat. More detailed criteria are summarized in table TS14J–1 (adapted from Fischenich and Morrow 2000).

Woody material structures, like most bank protection, are not suited for reaches with active bed degradation. Streams not transporting sediments or steep, high-energy systems transporting large cobbles and boulders are usually not good candidates for woody material structures. Although there are many examples of woody material projects, the basis for design is somewhat limited by a lack of quantitative data for design, performance, and environmental effects. Furthermore, many of the most important design variables are regional or site specific. An overview of published values computed or assumed for key design variables is provided in table TS14J–2. This table is intended to provide an impression of the limitations of current design criteria, and suggested design values are presented. Long-term performance information is limited (Thompson 2002; USDA NRCS 1999a). Accordingly, wood structures are not well suited for high-hazard, high-risk projects.

Environmental and habitat considerations

Although early interest in the use of wood structures for stream stabilization was driven by the need for low-cost approaches, current understanding includes consideration of the important role that woody materials play in creating and providing the diverse conditions typical of aquatic habitats (Gurnell et al. 2002). Knowledge regarding geomorphic and ecological functions of wood in rivers is rapidly increasing. Considerable evidence suggests that streams across North America were dominated by inputs and large accumulations of woody materials prior to European settlement (fig. TS14J–1).
<table>
<thead>
<tr>
<th>Variable</th>
<th>Considerations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Habitat requirements</td>
<td>Provides physical diversity, cover, velocity shelter, substrate sorting, pool development, undercut banks, and sites for terrestrial plant colonization using natural materials</td>
</tr>
<tr>
<td>Existing LWM density</td>
<td>Absent or depressed relative to similar nearby reaches that are lightly degraded</td>
</tr>
<tr>
<td>Sediment load</td>
<td>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</td>
</tr>
<tr>
<td>Bed material</td>
<td>Anchoring will be difficult in hard beds such as cobble, boulder, or bedrock</td>
</tr>
<tr>
<td>Bed stability</td>
<td>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</td>
</tr>
<tr>
<td>Bank material</td>
<td>LWM structures placed in banks with &gt;85% sand are subject to flanking</td>
</tr>
<tr>
<td>Bank erosion processes</td>
<td>Not recommended where the mechanism of failure is mass failure, subsurface entrainment, or channel avulsion. Best when toe erosion is the primary process</td>
</tr>
<tr>
<td>Flow velocity</td>
<td>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)</td>
</tr>
<tr>
<td>Site access</td>
<td>Heavy equipment access usually is needed to bring in and place large trees with rootwads</td>
</tr>
<tr>
<td>Conveyance</td>
<td>LWM structures can increase flow resistance if they occupy significant parts of the channel prism (Shields and Gippel 1995; Fischenich 1996)</td>
</tr>
<tr>
<td>Navigation and recreation</td>
<td>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</td>
</tr>
<tr>
<td>Raw materials</td>
<td>Suitable sources of trees needed nearby</td>
</tr>
<tr>
<td>Risk</td>
<td>Not suited for situations where failure would endanger human life or critical infrastructure</td>
</tr>
</tbody>
</table>
### Table TS14J–2

Published values for design variables for LWM structures

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Used for</th>
<th>Typical values</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density of wood in g/cm³</td>
<td>Buoyant force computation</td>
<td>0.4 to 0.5</td>
<td>Shields, Morin, and Cooper (2004)</td>
</tr>
<tr>
<td>(lowest, or worst-case condition)</td>
<td></td>
<td>0.5</td>
<td>D’Aoust and Millar (2000)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4 to 0.5</td>
<td>D’Aoust and Millar (1999)</td>
</tr>
<tr>
<td>Drag coefficient</td>
<td>Drag force computation</td>
<td>0.7 to 0.9</td>
<td>Shields and Gippel (1995)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Up to 1.5</td>
<td>Alonso (2004)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.4 to 1.2</td>
<td>Gippel et al. (1996)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.0</td>
<td>Fischenich and Morrow (2000)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.2 to 0.3 (tree)</td>
<td>D’Aoust and Millar (2000)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1.2 (rootwad)</td>
<td>D’Aoust and Millar (1999)</td>
</tr>
<tr>
<td>Design life for wood, yr</td>
<td>Planning</td>
<td>5 to 15</td>
<td>Fischenich and Morrow (2000)</td>
</tr>
<tr>
<td>Soil strength</td>
<td>Analysis of loads/anchoring provided by buried members</td>
<td>Soil forces on buried members neglected in order to be conservative. Range of values based on soil types</td>
<td>Shields, Morin, and Cooper (2004)</td>
</tr>
</tbody>
</table>

1/ Worst case conditions presume well-dried wood. Dry wood rapidly absorbs water and may increase its density by 100% after only 24-hr submergence (Thevenet, Citterio, and Piegay 1998). However, critical conditions, especially along smaller streams, are likely to occur before wood has had time to fully absorb water.

---

**Figure TS14J–1**

Large historical logjams of LWM, Great Raft, Red River, LA
Native communities of plants and animals depend on habitats provided by wood. Large wood has been observed to support step-pool morphology, generate local scour and deposition, and even to create dams and trigger avulsions on streams of all sizes. Natural wood accumulations reduce flow-through velocity at baseflow (Shields and Smith 1992), facilitating retention of organic materials for processing by lower levels of the food web. Woody material is an important substrate for benthic macroinvertebrates (Wallace and Benke 1984) and provides diverse pool habitat, cover, and velocity refugia for fish and other animals. Visual cover from predators is important for fish in many stream ecosystems. Terrestrial and amphibious animals use instream wood for basking and perching. Riparian plants often rapidly establish on deposition associated with woody material. Habitat rehabilitation projects often feature addition of woody materials to streams, primarily for habitat reasons and only secondarily for erosion control or channel stabilization (Fischenich and Morrow 2000). Local effects of wood structures (whether they induce scour or deposition) depend on structure design and site variables.

Design

Design of woody material structures should follow a geomorphic and ecological assessment of the watershed and a similar, more detailed assessment of the reach or reaches to be treated including an analysis of existing conditions and anticipated responses related to stability, as well as habitat diversity. Site assessments are described in more detail in NEH654.03.

Types of LWM structures

Existing designs for large wood structures may be grouped into a few basic configurations, as shown in table TS14J–3. Only general concepts are presented, as numerous variations are found. Combinations of woody materials with stone and living plant materials are common. The first three types shown in table TS14J–3 are intermittent structures, while the last three provide continuous protection along an eroding bank. Rootwads may be placed at spaced intervals or in an interlocking fashion so they may be considered either intermittent or continuous types. The design and construction of rootwads and tree revetments are also addressed in NEH654 TSTS14I. Intermittent structures provide greater aquatic habitat diversity than continuous protection. Existing design criteria for engineered log jams (ELJ) were developed based on experience in wide, shallow, coarse-bed streams in the Pacific Northwest. Application of these concepts to streams with relatively deep channels, sand beds, and flashy hydrology requires considerable modification (Shields, Morin, and Cooper 2004). Figure TS14J–2 depicts LWM (also known as large woody debris) where it is an impediment to flow or navigation, as illustrated in figure TS14J–2. Woody materials have been shown to be an integral part of stream ecosystems. However, LWM such as this can also be used for restoration purposes.

Selecting a type of structure

Configuration of a LWM structure should be selected using similar criteria that are employed for selecting any approach for stream stabilization or habitat rehabilitation:

- The configuration should address the dominant erosion processes operating on the site (Shields and Aziz 1992).
- Key habitat deficiencies (lack of pools, cover, woody substrate) should be addressed.

Figure TS14J–2 White River, IN, with large woody debris (Photo courtesy of USGS)
Table TS14J–3  Classification of large wood instream structures

<table>
<thead>
<tr>
<th>Configuration</th>
<th>Sketch</th>
<th>Description</th>
<th>Strengths</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Engineered logjams</td>
<td><img src="image" alt="engineered_logjams" /></td>
<td>Intermittent structures built by stacking whole trees and logs in crisscross arrangements</td>
<td>Emulates natural formations. Creates diverse physical conditions, traps additional debris</td>
<td>Abbe, Montgomery, and Petroff (1997); Shields, Morin, and Cooper (2004)</td>
</tr>
<tr>
<td>Log vanes</td>
<td><img src="image" alt="log_vanes" /></td>
<td>Single logs secured to bed protruding from bank and angled upstream. Also called log bendway weir</td>
<td>Low-cost, minimally intrusive</td>
<td>Derrick (1997); D'Aoust and Millar (2000)</td>
</tr>
<tr>
<td>Log weirs</td>
<td><img src="image" alt="log_weirs" /></td>
<td>Weirs spanning small streams comprised of one or more large logs</td>
<td>Creates pool habitat</td>
<td>Hilderbrand et al. 1998; Flosi et al. (1998)</td>
</tr>
<tr>
<td>Rootwads</td>
<td><img src="image" alt="rootwads" /></td>
<td>Logs buried in bank with rootwads protruding into channel</td>
<td>Protects low banks, provides scour pools with woody cover</td>
<td></td>
</tr>
<tr>
<td>Tree revetments or roughness logs</td>
<td><img src="image" alt="tree_revetments" /></td>
<td>Whole trees placed along bank parallel to current. Trees are overlapped (shingled) and securely anchored</td>
<td>Deflects high flows and shear from outer banks; may induce sediment deposition and halt erosion</td>
<td>Cramer et al. (2002)</td>
</tr>
<tr>
<td>Toe logs</td>
<td><img src="image" alt="toe_logs" /></td>
<td>One or two rows of logs running parallel to current and secured to bank toe. Gravel fill may be placed immediately behind logs</td>
<td>Temporary toe protection</td>
<td>Cramer et al. (2002)</td>
</tr>
</tbody>
</table>
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 \( R/W > 3 \) (radius of curvature/bankfull width), decreasing to 0 for \( R/W < 2.5 \). 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 overturned 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:

\[
\vec{F}_b = \left[ \rho_{\text{wood}} V_{\text{wood}} - \rho_{\text{water}} V_{\text{water}} \right] \vec{g}
\]  

(eq. TS14J–1)

where:
- \( \rho \) = density
- \( V \) = volume
- \( \vec{g} \) = the gravitational acceleration vector in the vertical direction

For a fully submerged structure,

\[
V_{\text{wood}} = V_{\text{water}} = V \quad \text{and} \quad \vec{F}_b = \left( \rho_{\text{wood}} - \rho_{\text{water}} \right) V \vec{g}
\]

(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. Thenevenet, 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 \( \gamma_\text{structure} \), a simple solution for the depth, \( d_w \), at which the structure becomes neutrally buoyant (buoyant force = gravitational forces) may be computed using:

\[
\gamma_\text{structure} = \frac{d_w}{h} \left( 2 - \frac{d_w}{h} \right)
\]

(eq. TS14J–3)

where:
- \( \gamma_w \) = specific weight of water

Friction—The movement of large wood structures by sliding along the bed will be resisted by a frictional force, \( \vec{F}_t \), with magnitude equal to the normal force, \( \vec{F}_n \), times the coefficient of friction between the woody material and the bed.

\[
\vec{F}_t = \mu_{\text{bed}} \vec{F}_n
\]

(eq. TS14J–4)

In the absence of measured data, Castro and Sampson (2001) assumed that \( \mu_{\text{bed}} = \tan \theta \), where \( \theta \) 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, \( \vec{F}_t \) may be effectively zero for design conditions.

Drag—The drag force on an LWM structure may be computed using the equation

\[
\vec{F}_d = \frac{C_D A}{2g} \left[ \vec{U}_o \times \vec{U}_a \right] \cdot \hat{c}
\]

(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
- \( \hat{c} \) = 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, \( C_D \):

\[
C_D = W \left[ 1 - 0.35 \exp \left( \frac{-4G}{d} \right) \right] \times \left[ 1.062 \times 2 \times 10^{-6} R_e - 3 \times 10^{-12} R_e^2 + 2 \times 10^{-15} R_e^3 \right]
\]

(eq. TS14J–6)
where:

- \( G \) = distance from the bottom of the log to the bed
- \( R_e \) = cylinder Reynolds number, \( \frac{Ud}{v} \)

where:

- \( U \) = magnitude of the approach flow velocity
- \( d \) = diameter of the log
- \( v \) = kinematic viscosity of the water
- \( W \) = factor to account for the increase in drag due to surface waves, and may be given by

\[
W = -0.28 \ln \left( \frac{z}{d} \right) + 1.4 \quad \text{(eq. TS14J–7)}
\]

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

\[
\vec{F}_L = C_L \gamma_s \left[ \vec{U}_m \times \vec{U}_a \right] \frac{2g}{\vec{e}} \quad \text{(eq. TS14J–8)}
\]

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 \( \gamma_{soil} \), friction angle \( \phi \) 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.
The vertical loading on the log due to the weight of the soil above it will be given by:

\[ F_{soil} = \sigma'_v L_{em} \]  
(eq. TS14J–9)

where:

\[ \sigma'_v = (D - D_w) (\gamma_{soil} - \gamma_{water}) + D_w \gamma_{soil} \]  
(eq. TS14J–10)

where:

\[ \gamma_{soil} = \text{moist or total unit weight of the soil above the log} \]

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,

\[ F_{soil} = C_d \sigma'_v B_d^2 \frac{L}{D} \]  
(eq. TS14J–11)

where:

\[ B_d = \text{width of the ditch} \]
\[ C_d = \text{a coefficient that captures the interaction between the ditch walls and the fill} \]

\[ C_d = \left[ \frac{1 - e^{-0.38 \frac{B_d}{R_i}}} {0.38} \right] \]  
(eq. TS14J–12)

for \( \frac{D}{B_d} < 2 \) and

\[ C_d = \frac{D}{B_d} \]  
(eq. TS14J–13)

for \( \frac{D}{B_d} \geq 2 \)  
(eq. TS14J–14)

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, \( \bar{F}_p \), is given by

\[ \bar{F}_p = 0.5 \sigma_p L_{em} d \]  
(eq. TS14J–16)

where:

\[ \sigma_p = \text{passive soil pressure} \]

is given by

\[ \sigma_p = \sigma'_v K_p + 2c (K_p)^{0.5} \]  
(eq. TS14J–17)

where:

\[ K_p = \text{coefficient of passive earth pressure} \]

is given by

\[ K_p = \tan^2 \left( 45 + \frac{\phi}{2} \right) \]  
(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, \( \bar{M}_d \), about the buried tip of the embedded log is given by the vector sum

\[ \bar{M}_d = \left[ \left( \bar{F}_{soil} \frac{1}{2} \right) L_{em} + \bar{F}_p \left( \frac{2}{3} \right) L_{em} + \bar{F}_c L_c \right] \times \bar{I} \]  
(eq. TS14J–19)

where \( \bar{I} \) 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, \( \bar{M}_r \), will act opposite the driving moment and is given by the vector sum

\[ \bar{M}_r = \left[ \left( \bar{F}_{soil} \frac{1}{2} \right) L_{em} + \bar{F}_p \left( \frac{2}{3} \right) L_{em} + \bar{F}_c L_c \right] \times \bar{I} \]  
(eq. TS14J–20)

where \( \bar{F}_c \) is the restraining force due to anchor cables or ballast, and \( L_c \) is the appropriate moment arm about the buried tip of the embedded log.

**Ballast and anchoring**

Forces and moments due to anchors may be added to the other forces acting on the LW structure to compute factors of safety. The factor of safety with respect to forces, \( F_{fs} \), 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.

\[ F_{fs} = \frac{F_{soil} + F_{px} + F_{py}}{F_p + F_L} \]  
(eq. TS14J–21)

\[ F_{fs} = \frac{F_{soil} + F_{px} + F_{py}}{F_D} \]  
(eq. TS14J–22)
M_d acts opposite M_r 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:

\[
F_{sm} = \frac{M_r}{M_d} \quad \text{(eq. TS14J–23)}
\]

Anchoring systems should be designed to achieve factors of safety greater than 2 due to the high level of uncertainty in computations for imposed forces. Anchoring approaches include placing ballast (soil, cobbles, boulders) on or within the structure, embedding part or all of the large wood in the bank or in a stone structure, and using cable, marine rope, or chain to secure the structure to boulders, soil anchors (NEH654 TS14E), stumps, trees, deadmen, or pilings (Cramer et al. 2002; Fischenich and Morrow 2000). When logs or woody elements are used as ballast, it is important for the designer to consider the implications of the wood rotting and becoming lighter. When boulders or bed material are used for ballast, buoyant, drag, and lift forces on the ballast rock must be considered in the force balance (D'Aoust and Millar 2000). An electronic spreadsheet may facilitate this calculation.

Logs in complex structures may be attached to one another or to boulders by drilling holes through the logs and pinning them together with steel rebar. Epoxy adhesive has also been used for attaching logs. Abbe, Montgomery, and Petroff (1997) favor an approach that may be termed passive anchoring (Cramer et al. 2002), in which the shape, weight, ballast, and placement of a structure are adequate to resist movement in events up to the design flow. Passively anchored structures may be comprised of wood members that are attached to one another, but not to external anchors. Passive anchoring is not recommended for high hazard situations, sites with vulnerable infrastructure downstream, or sites where structures will be frequently overtopped.

### Materials

Minimum dimensions, species, and sources for woody materials should be specified during design. Cramer et al. (2002) suggest the following guidelines for size of trees and rootwads:

<table>
<thead>
<tr>
<th>Dimension</th>
<th>Minimum size</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rootwad diameter</td>
<td>Bankfull discharge depth</td>
</tr>
<tr>
<td>Trunk diameter</td>
<td>0.5 × bankfull discharge depth</td>
</tr>
<tr>
<td>Tree length</td>
<td>0.25 × bankfull discharge width</td>
</tr>
</tbody>
</table>

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 (Juniperus virginiana), western red cedar (Thuja plicata), coastal redwood (Sequoia 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>
<thead>
<tr>
<th>Species</th>
<th>Durability (assumin...</th>
<th>Source of information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cottonwood (<em>Populus</em> spp.)</td>
<td>Poor</td>
<td>Johnson and Stypula (1993)</td>
</tr>
<tr>
<td>Alder (<em>Alnus</em> spp.)</td>
<td>Poor</td>
<td>Johnson and Stypula (1993)</td>
</tr>
<tr>
<td>Maple (<em>Acer</em> spp.)</td>
<td>Fair (will survive 5 to 10 yr)</td>
<td>Johnson and Stypula (1993)</td>
</tr>
<tr>
<td>Hemlock (<em>Tsuga</em> spp.)</td>
<td>Least durable of conifers</td>
<td>Johnson and Stypula (1993)</td>
</tr>
<tr>
<td>Sitka spruce (<em>Picea sitchensis</em>)</td>
<td>Excellent</td>
<td>Johnson and Stypula (1993)</td>
</tr>
<tr>
<td>Douglas-fir (<em>Pseudotsuga</em> spp.)</td>
<td>Excellent (will survive 25 to 60 yr)</td>
<td>Johnson and Stypula (1993); Harmon et al. (1986)</td>
</tr>
<tr>
<td>Western red cedar (<em>Thuja plicata</em>)</td>
<td>Most desirable (will survive 50 to 100 yr)</td>
<td>Johnson and Stypula (1993)</td>
</tr>
<tr>
<td>Yellow-poplar (<em>Liriodendron tulipifera</em>)</td>
<td>0.4 yr</td>
<td>Harmon et al. (1986)</td>
</tr>
<tr>
<td>Aspen (<em>P. tremuloides</em>)</td>
<td>5 yr</td>
<td>Harmon et al. (1986)</td>
</tr>
<tr>
<td>White fir (<em>A. concolor</em>)</td>
<td>4 yr</td>
<td>Harmon et al. (1986)</td>
</tr>
<tr>
<td>Black locust, red mulberry, Osage orange, Pacific yew</td>
<td>Exceptionally high heartwood decay resistance</td>
<td>Simpson and TenWolde (1999)</td>
</tr>
<tr>
<td>Old growth baldcypress, catalpa, cedars, black cherry, chestnut, Arizona cypress, junipers, honeylocust, mesquite, old growth redwood, sassafras, black walnut</td>
<td>Resistant or very resistant to heartwood decay</td>
<td>Simpson and TenWolde (1999)</td>
</tr>
<tr>
<td>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</td>
<td>Slightly or nonresistant to heartwood decay</td>
<td>Simpson and TenWolde (1999)</td>
</tr>
</tbody>
</table>

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.

(210–VI–NEH, August 2007)
Climate index = \[\frac{\sum_{m=1}^{12} [(T-35)(D-3)]}{30}\]  
(eq. TS14J–23)

where:

- \(T\) = mean monthly temperature (°F)
- \(D\) = mean number of days in the month with 0.01 inch or more of precipitation

The summation represents the sum of products for all of the months of the year. The sum is divided by 30 to make the index fall between 0 and 100 for most of the United States. For example, Scheffer computed values of 82.5, 44.8, and 22.0 for Atlanta, Georgia; Des Moines, Iowa; and Casper, Wyoming, respectively. This implies that a wood structure would last about four times longer in a climate typical of Wyoming than one typical of Georgia, all other factors being equal.

Synthetic LWM for stream work is available commercially (Bolton et al. 1998). These products are engineered to compare favorably with natural materials in terms of durability or habitat value. However, they may be less effective in terms of habitat creation or more costly than natural materials. Cost comparisons should consider full project life cycles.

**Cost**

Costs for LWM structures are heavily influenced by site variables and material sources. Cramer et al. (2002) provide typical cost ranges for large wood of $500 to $750 per tree with rootwad and $200 to $300 per tree without rootwad. These figures include material, hauling to the site, excavation, spoilage, and installation. Additional cost information is summarized in table TS14J–5.

**Maintenance**

LWM structures should be viewed as temporary measures to trigger desirable natural changes in channels and banks. Accordingly, structures gradually degrade and break down. However, structures should be maintained until planted or invading woody plants have succeeded in establishing in the treated area. A relatively high level of maintenance is necessary if initial configurations are to be maintained for more than a few years. Annual low-water inspections are advisable, with particular attention to anchoring systems, decay status of woody materials, hazards to downstream infrastructure, and erosion patterns. Habitat monitoring may be qualitative, but field measurement of water depth, width, and velocity (Shields, Knight, Morin, and Blank 2003) is preferable. Photo documentation and cross-sectional and thalweg surveys are most helpful in detecting changes. Cramer et al. (2002) recommend additional inspections following any event that equals or exceeds the 1-year flow during the first 3 years following construction.
## Use of Large Woody Material for Habitat and Bank Protection

### Table TS14J-5

<table>
<thead>
<tr>
<th>Year</th>
<th>Location</th>
<th>Protected bank length, m</th>
<th>Unit cost(^1)/(^2)/(^3), $/m</th>
<th>Comments</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>1987</td>
<td>Nestucca River and Elk Creek, OR</td>
<td>1,960</td>
<td>24</td>
<td>119 woody debris structures using 99 mature conifers placed for habitat objectives, not stabilization</td>
<td>House and Crispin (1990)</td>
</tr>
<tr>
<td>1990–91</td>
<td>North Fork Porter Creek, WA</td>
<td>500</td>
<td>165</td>
<td>Five different log configurations anchored with cables and boulders for habitat purposes only</td>
<td>Cederholm et al. (1997)</td>
</tr>
<tr>
<td>1990–91</td>
<td>North Fork Porter Creek, WA</td>
<td>500</td>
<td>13</td>
<td>60 trees &gt; 30 cm diameter cut felled into stream from banks and tethered to stumps with cable for habitat purposes only</td>
<td>Cederholm et al. (1997)</td>
</tr>
<tr>
<td>1994</td>
<td>Buffalo River, AR</td>
<td>66</td>
<td>6</td>
<td>Cedar tree revetments and willow rootwads planted in ditches. Two of 13 sites have not performed well</td>
<td>Personal communication, David Mott, National Park Service</td>
</tr>
<tr>
<td>1996</td>
<td>Bayou Pierre, MS</td>
<td>240</td>
<td>117</td>
<td>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</td>
<td>Personal communication, Larry Marcy, U.S. Fish and Wildlife Service</td>
</tr>
<tr>
<td>1988–97</td>
<td>Six urban gravel bed streams, Puget Sound, WA</td>
<td>2,960</td>
<td>493</td>
<td>Anchored and unanchored LWM added for flood control, sediment/erosion control and habitat enhancement</td>
<td>Larson, Booth, and Morley (2001)</td>
</tr>
<tr>
<td>1998</td>
<td>Various, MO</td>
<td>72 (^2)</td>
<td>Double row tree revetment installed using heavy equipment</td>
<td>Personal communication, Brian Todd, State of Missouri</td>
<td></td>
</tr>
<tr>
<td>2000</td>
<td>Little Topashaw Creek, MS</td>
<td>1,500</td>
<td>80</td>
<td>72 LWM structures in small, sand-bed stream. Unit cost = $95/m when willow planting is included</td>
<td>Shields, Morin, and Cooper (2004)</td>
</tr>
</tbody>
</table>

\(^1\) 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
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

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose</td>
<td>TS14K–1</td>
</tr>
<tr>
<td>Introduction</td>
<td>TS14K–1</td>
</tr>
<tr>
<td>Benefits of using stone</td>
<td>TS14K–1</td>
</tr>
<tr>
<td>Stone considerations</td>
<td>TS14K–1</td>
</tr>
<tr>
<td>Design considerations</td>
<td>TS14K–3</td>
</tr>
<tr>
<td>Placement of rock</td>
<td>TS14K–5</td>
</tr>
<tr>
<td>Dumped rock riprap</td>
<td>TS14K–5</td>
</tr>
<tr>
<td>Machine-placed riprap</td>
<td>TS14K–6</td>
</tr>
<tr>
<td>Treatment of high banks</td>
<td>TS14K–8</td>
</tr>
<tr>
<td>Embankment bench method</td>
<td>TS14K–8</td>
</tr>
<tr>
<td>Excavated bench method</td>
<td>TS14K–8</td>
</tr>
<tr>
<td>Surface flow protection</td>
<td>TS14K–8</td>
</tr>
<tr>
<td>Treatment of bedrock controlled streams</td>
<td>TS14K–8</td>
</tr>
<tr>
<td>Precast toe blocks method</td>
<td>TS14K–10</td>
</tr>
<tr>
<td>Steel dowel method</td>
<td>TS14K–10</td>
</tr>
<tr>
<td>Other structural treatments</td>
<td>TS14K–12</td>
</tr>
<tr>
<td>Stone with soil bioengineering</td>
<td>TS14K–12</td>
</tr>
<tr>
<td>Longitudinal peak stone toe (LPST)</td>
<td>TS14K–12</td>
</tr>
<tr>
<td>Timber and rock cribbing</td>
<td>TS14K–13</td>
</tr>
<tr>
<td>Wire mesh gabions</td>
<td>TS14K–14</td>
</tr>
<tr>
<td>Vegetated gabion</td>
<td>TS14K–15</td>
</tr>
<tr>
<td>Grouted riprap</td>
<td>TS14K–15</td>
</tr>
<tr>
<td>Habitat enhancement with stone</td>
<td>TS14K–17</td>
</tr>
<tr>
<td>Conclusion</td>
<td>TS14K–17</td>
</tr>
</tbody>
</table>
### Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Name</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS14K–1</td>
<td>Specified rock sizes for gabions</td>
<td>TS14K–14</td>
<td></td>
</tr>
</tbody>
</table>

### Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Name</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>TS14K–1</td>
<td>Capillary breakdown of stone</td>
<td>TS14K–3</td>
<td></td>
</tr>
<tr>
<td>TS14K–2</td>
<td>Typical riprap section</td>
<td>TS14K–7</td>
<td></td>
</tr>
<tr>
<td>TS14K–3</td>
<td>Embankment bench method</td>
<td>TS14K–9</td>
<td></td>
</tr>
<tr>
<td>TS14K–4</td>
<td>Excavated bench method</td>
<td>TS14K–9</td>
<td></td>
</tr>
<tr>
<td>TS14K–5</td>
<td>Steel dowel method</td>
<td>TS14K–10</td>
<td></td>
</tr>
<tr>
<td>TS14K–6</td>
<td>Precast toe block</td>
<td>TS14K–11</td>
<td></td>
</tr>
<tr>
<td>TS14K–7</td>
<td>Stone toe and live poles</td>
<td>TS14K–12</td>
<td></td>
</tr>
<tr>
<td>TS14K–8</td>
<td>Brush layer over stone toe</td>
<td>TS14K–12</td>
<td></td>
</tr>
<tr>
<td>TS14K–9</td>
<td>Vertical bundle and stone toe</td>
<td>TS14K–12</td>
<td></td>
</tr>
<tr>
<td>TS14K–10</td>
<td>Timber and rock cribbing</td>
<td>TS14K–13</td>
<td></td>
</tr>
<tr>
<td>TS14K–11</td>
<td>Gabions showing a neat, compact, placement of stone with a uniform appearance</td>
<td>TS14K–15</td>
<td></td>
</tr>
<tr>
<td>TS14K–12</td>
<td>Vegetated gabions under construction</td>
<td>TS14K–15</td>
<td></td>
</tr>
<tr>
<td>TS14K–13</td>
<td>Assembly sequence of a Green Gabion™</td>
<td>TS14K–16</td>
<td></td>
</tr>
</tbody>
</table>
Technical Supplement 14K

Purpose

Structural measures for streambank protection, particularly rock riprap, have been used extensively in support of stream restoration designs. Stone continues to be an important component of many stream restoration and stabilization projects, where stone or rock provides the needed weight or erosion protection, as well as providing a needed foundation for other design elements. This technical supplement is intended to provide field staffs with an understanding of some of the basic principles, design considerations, and techniques used to treat streambank erosion with rock. Design considerations that are applicable to any structure involving the use of stone are addressed. The use of stone as part of soil bioengineering and to complement instream habitat is also addressed.

Introduction

Stone has long been used to provide immediate and permanent stream and river protection. It continues to be a major component in many of the newer and more ecologically friendly projects, as well. The use of stone in a stream restoration design is a function of the engineering and ecological requirements of the final design. While the term stone can also be used to refer to a unique size of material (between cobbles and boulders), it is used interchangeably in this technical supplement with the term rock. Herein, these terms refer to large, engineered, geologic material used as an integral part of the restoration design.

This technical supplement describes some of the typical applications of both integrated streambank stabilization systems and stand-alone riprap treatments. It is recognized that stone and rock are also used to create desired habitat elements, but this technical supplement focuses primarily on the design of stone treatments for streambank stabilization and protection. Basic principles, stone requirements, design considerations, and techniques used to treat streambank erosion with rock are all described. While much of the guidance described herein was developed for application of stone riprap revetments, it is also applicable for other designs involving rock.

Benefits of using stone

Structural measures are designed to withstand high streamflows and provide adequate protection as soon as installation is complete. Rock may be readily available to most sites, but where it is not, alternative structural measures are designed based on the local cost of available materials (concrete, steel, manufactured materials, wood). Established techniques exist for rock design and construction. Rock riprap measures have a great attraction as a material of choice for emergency programs where quick response and immediate effectiveness are critical.

Rock riprap is needed for many streambank stabilization designs, especially where requirements for slope stability are restrictive, such as in urban areas. It is one of the most effective protection measures at the toe of an eroding or unstable slope. The toe area generally is the most critical concern in any bank protection measure. The primary advantages of stone over vegetative approaches are the immediate effectiveness of the measure with little to no establishment period. The use of stone may offer protection against stream velocities that exceed performance criteria for vegetative measures.

Stone considerations

Not all rocks are created equal. A variety of important stone design characteristics and requirements exist that must be accounted for to successfully use rock in the stream.

Stone size

The stone used in a project, whether it is part of a combined structure or used as a traditional riprap revetment, must be large enough to resist the forces of the streamflow during the design storm. A stone-sizing technique appropriate for the intended use must also be selected. Many established and tested techniques are available for sizing stone. Most techniques use an estimate of the stream’s energy that the rock will need to resist, so some hydraulic analysis is generally required. Guidance for stone sizing techniques is provided in NEH654 TS14C.
Stone shape

Some methods use different dimensions to characterize stone size. The critical dimension is the minimum sieve size through which the stone will pass. Some techniques assume that riprap is the shape of a sphere, cube, or even a football shape (prolate spheroid). To avoid the use of thin, platy rock, neither the breadth nor the thickness of individual stones is less than a third of its length. The U.S. Department of Agriculture (USDA) Natural Resources Conservation Service (NRCS) riprap specifications allow riprap to be a spheroid three times as long as it is thick (L/B = 3).

Note that the shape of most riprap can be represented as the average between a sphere and a cube. An equation for an equivalent diameter of riprap shaped between a cube and a sphere is:

\[
D = \left[ \frac{2 \times W}{s \times \left( 1 + \frac{\pi}{6} \right)} \right] \quad \text{(eq. TS14K–1)}
\]

where:

- \( W \) = weight of the stone, lb
- \( s \) = 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 (\( \gamma_s \)) 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 \( \gamma_s = 2.65 \times 62.4 \text{ pounds per cubic foot (density of water)} = 165.36 \text{ pounds per cubic foot (a normal design assumption for rock density)}. \) NRCS specifications for riprap allow a minimum \( G_s = 2.50 \). Note that specific gravity is also shown as \( \rho \) 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).

**Design considerations**

Stabilizing channel banks is a complex problem and does not always lend itself to precise design. The success of a given installation depends on the judgment, experience, and skill of the planners, designers, technicians, and installers. Several important issues that must be considered for the successful design of projects that depend on the rock performance are briefly described.

**Filter layer**

Where stone is placed against a bank that is composed of fine-grained or loose alluvium, a filter layer or bedding is often used. This filter layer prevents the smaller grained particles from being lost through the interstitial spaces of the riprap material, while allowing seepage from the banks to pass. This filter layer needs to be appropriately designed to protect the in-place bank material and remain beneath the designed stone or riprap. Therefore, the gradation is based in part of the gradation of the riprap layer and the bank material. The filter layer typically consists of a geosynthetic layer or an 8-inch-thick layer of sand, gravel, or quarry spalls. For design of appropriate filters under rock riprap, refer to NEH633.26.

Banks with fine-grained silts or sands may require a geotextile to provide separation and filtration under riprap. Geosynthetics are covered in more detail in NEH654 TS14D, as well as in Design Note #1 and Material Specification 595 for the design and material considerations for geotextiles. A useful reference for geotextile design considerations is the American Association of State Highway and Transportation Officials (AASHTO) M28.

Some soil bioengineering techniques do not function well under geotextiles, and placing holes through the geotextile for plantings may provide a seepage path that would weaken the structure. This may require a
trade-off analysis to balance the advantages of incorporating soil bioengineering against the advantages of an intact geotextile filter. Finally, there will also be cases where the banks may have sufficient gravel or cobbles, so that neither bedding nor geotextiles are needed.

Bank slope
Many stone sizing techniques also require information about the bank slope. In addition, a geotechnical embankment analysis may impose a limit on the bank slope. The recommended maximum slope for most riprap placement is 2H:1V. Short sections of slopes at 1.5H:1V are sometimes unavoidable, but are not desirable. Most rock cannot be stacked on a bank steeper than 1.5H:1V and remain there permanently. For riprap placement of 1.5H:1V and steeper, grouting of the rock to keep it in place must be strongly considered. Alternative measures, such as gabion baskets, are well suited to steep banks. Also, flatter slopes increase the opportunity for vegetation establishment.

Height
Stone should extend up the bank to a point where the existing vegetation or other proposed treatment can resist the forces of the water during the design event. In a soil bioengineering project, a stone revetment typically does not exceed the elevation of the level of the channel-forming flow event. However, there are exceptions where it is advisable to extend the riprap to the top of the bank.

Thickness
Different stone-sizing techniques may have different assumptions concerning the blanket thickness. The thickness of the placed rock should equal or exceed the diameter of the largest rock size in the gradation. In practice, this thickness will be one and a half to three times the median rock diameter ($D_{50}$). A typical minimum thickness is the greater of 0.75 times the $D_{100}$ or one and a half times the $D_{50}$. The ability to use vegetative methods within a riprap revetment is diminished by additional riprap depth. While posts have been installed in revetments up to 4 feet thick, live cuttings or joint planting within a riprap thickness larger than 24 inches has had limited success.

Length
The revetment should significantly overlap the eroding area. The starting point needs to be well protected, properly keyed into the bank, and located sufficiently upstream of the major point of streamflow attack. Starting the treatment upstream helps prevent the streamflow from getting behind the structure and progressively eroding and undermining the protection. Likewise, if the bank protection does not extend sufficiently past the critical area of attack to a point where the streamflow is safely guided back into the primary channel, severe erosion can occur and start progressive failure in an upstream direction.

Where it is not possible to begin and end a structural revetment at a stable area, it is recommended that a stone revetment be extended a minimum distance of one channel width upstream and one and a half channel widths downstream of the eroded area. However, this limited treatment area has a higher risk of failure.

Tiebacks
Tiebacks or key-ins are used to reduce the likelihood of high flows concentrating behind stone slope protection. Tiebacks are used on both the upstream and downstream ends of a stone revetment. A typical rule of thumb for the depth to key into the bank is the bank height plus the anticipated scour depth. On long stone revetments, intermediate tiebacks are often used to ensure the reach integrity. Also, it is suggested that key-ins not be positioned at 90 degrees to the flow, but rather at an angle (30 to 45 degrees to the direction of flow) into the bank. Keying at an angle reduces sudden transitions of flow at the beginning and end of the revetment, and if the stream migrates, the key-in will act as a deflector.

Scour
Toe scour is the most frequent cause of failure in streambank armor protection projects. Scour can be long term, general, and local. More information on scour is provided in NEH654 TS14B.

The greatest scour depths generally occur on the outside and lower portion of curves. Scour depths may increase immediately below and adjacent to structural protection due to the higher velocity section of a stream adjacent to the relatively smooth structure surface. This may undermine the structure and result in failure.

Common methods for providing toe protection are:

- placing the stone to the maximum expected scour depth
• placing sufficient stone along the toe of the revetment to launch or fall in, and fill any expected scour
• providing a sheet-pile toe to a depth below the anticipated depth of scour or to a hard point
• paving the bed

The most commonly employed method is to extend (or key-in) the bank protection measures down to a point below the probable maximum depth of the anticipated bed scour. Where the project involves a significant investment for the protection of valuable property, potential scour can be calculated using the procedures described in NEH654 TS14B. Where there is less of an investment, approximations can be employed. A typical rule of thumb for a minimum key-in depth is one and a half times the riprap thickness or a minimum of 2 feet below the existing streambed. This practical solution generally gives good protection against undermining. Designers can review reliable data on local scour in the area, regional data, or use local experience in determining this minimum depth.

Ice and debris
River ice can have a major impact on riprap protection. Ice and debris increase the stresses on riprap by impact and flow concentration. Ice attached to stone may also dislodge stone and decrease blanket stability. Ice rafting, lifting or plucking, raft impact damage, ice raft push, and velocity increase below ice jams can all cause problems. Detailed discussions of these issues are available (Vaughan, Albert, and Carlson 2002; USACE EM 1110–2–1612, 1999).

A general rule of thumb for riprap subject to attack by large floating debris is that thickness should be increased by 6 to 12 inches, accompanied by an appropriate increase in stone size. Riprap damage from debris impacts is usually more extensive on banks with steep slopes. Therefore, streams with heavy debris loads should be not have armored slopes steeper than 1V:2.5 H (USACE EM 1110–2–1601, 1994f).

Vandalism
Many rock treatments are composed of a relatively thin layer of stone, and unauthorized removal of selected stones from the rock matrix can cause serious problems. Stone is often removed from projects for landscaping and other personal uses. Monitoring and maintenance activities should be in place to protect the project, minimize vandalism, and provide timely repair. Where vandalism is expected, it may be advisable to use larger stone than that required for stability to reduce the likelihood of removal by hand.

Placement of rock
Rock should be placed from the lowest to the highest elevation to allow gravitational forces to minimize void spaces and help lock the rock matrix together. It is important that riprap be placed at full-course thickness in one operation. Final finished grade of the slope should be achieved as the material is placed. Care should be taken not to segregate or group material sizes together during placement. Allowing the stone to be pushed or rolled downslope will cause stone size segregation. See ASTM D6825 on placement of riprap revetments.

An advantage of using riprap structures is that materials are generally readily available, and contractors with appropriate equipment and experience can be found. However, careful consideration should be given early in the design process to the stone installation method. Two commonly employed installation methods are described below.

Dumped rock riprap
This method of protection may be necessary where access to the streambed is limited or for emergency situations. Streambank work using dumped rock requires a source of low-cost rock. Access roads must be available near the stream channel, so that rock can be hauled to the streambank and either dumped over the bank or along the edge. If the job requires large quantities of rock, the operation must be set up to accommodate regular deliveries to the job site. In some cases, the banks may be too weak to support a loaded truck, thereby preventing dumping of rock directly over the streambank. In such cases, the rock may be dumped as close to the edge as possible and pushed over the edge with a bulldozer or front-end loader. Larger rock should be placed at the bottom of the revetment work to provide a stable toe section. The use of a front-end loader may be useful to select rock by size and push it over the bank.
This type of placement usually results in a poor gradation of material due to material segregation, requiring more volume to make up for the lack of gradation. While this type of bank protection requires more stone per square yard of bank protection than machine-placed riprap, it generally requires less labor and equipment operating hours.

### Machine-placed riprap

This type of riprap is placed using a track-mounted backhoe or a power crane with a clam shell or orange peel bucket. The riprap is placed on a prepared slope of the streambank to a minimum design thickness of 12 to 18 inches. The larger stones are placed in a toe trench at the base of the slope. This method requires an experienced equipment operator to achieve uniform and proper placement. The toe or scour trench can be dug with the backhoe or clam shell as the machine moves along the slope. The machine can do the backfilling with rock in the same manner.

The bank sloping or grading generally is accomplished with a backhoe or sometimes a Gradall®. If a power crane is used, a dragline bucket must be used with the crane for slope grading. A perforated dragline bucket works best because it allows excess water to drain from the bucket.

Appropriate bedding and/or geotextile can be installed after the grading and slope preparation are completed. The primary function of these materials is for filtration—to prevent movement of soil base materials through the rock riprap. Bedding is normally placed by dump truck and spread to the desired thickness with a backhoe bucket, a front-end loader, or a small dozer. Geotextile must be placed by hand, secured in place as recommended by the manufacturer, consistent with site specifications. It is important that the geotextile be placed in intimate contact with the base to preclude voids beneath the geotextile. Under larger stone, a coarse bedding may be placed on the geotextile to assure that the geotextile stays in contact with the subbase. In some locations, geotextiles may also be used as a reinforcement in very soft foundation conditions. As previously noted, there will also be situations where the banks may have sufficient gravel content, so that neither bedding nor geotextiles are needed.

Riprap should be placed to provide a reasonably well-graded and dense mass of rock with a minimum of voids and with the final surface meeting the specified lines and grades. The larger stones should be placed in the toe trench or well distributed in the revetment. The finished stone protection should be consolidated by the backhoe bucket or other acceptable means so that the surface is free from holes, noticeable projections, and clusters or pockets of only small or only large stones.

Riprap placement should begin at the toe trench and progress up the slope maintaining the desired rock placement thickness as the work proceeds. After the toe trench has been filled to the original stream bottom level, the operator should build a wall or leading edge with the riprap, which is the full layer thickness. That thickness should be maintained throughout the placement of the riprap. The wall should be maintained at about a 45-degree angle from a transverse line down the slope, as the placement progresses from the initial starting point at the streambed and progresses up and across the slope (fig. TS14K–2).

Riprap rock should be handled and placed to the full layer thickness in one operation so that segregation is minimized and bedding or geotextile materials used under the riprap are not disturbed after the initial rock placement. Adding rock to the slope or removing it after the initial placement is not practical and generally produces unsatisfactory results. Dumping stone from the top and rolling it into place should also be avoided. This type of operation causes segregation and defeats the purpose of a rock gradation. Running on the riprap slope with track equipment, such as a bulldozer or rubber tire mounted front end loader, should also be avoided. It can damage the rock mass already in place. This operation can also tear the geotextile or damage the bedding by displacing material throughout the rock course. Tamping of the rock with the backhoe bucket can sometimes be used effectively to even up the surface appearance of riprap placement and further consolidate the rock course.

It is advisable to have a test section when riprap is being placed over geotextile to check for geotextile puncturing. After the riprap is placed, it is removed, and the geotextile is evaluated.
Figure TS14K–2  Typical riprap section

Side view

Creek bottom

Original ground line

Fill

Geotextile or bedding material as needed

This part of slope planted to shrubs and grass

Placement direction

Front view

Top of slope

Edge of rock

2 ft 6 in minimum

Variable

2 ft minimum

1 ft minimum

3 ft 6 in minimum

2H:1V

45°

Variable

2 ft 6 in minimum
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.
Cut the gravel or sand bar to compensate for lost channel capacity and to provide material to build the bench.
Steel dowel method

This method uses No. 8 or No. 6 steel reinforcing rods, depending on the size of the rock riprap. These rods are typically about 3 feet long and are grouted in place in holes that have been drilled into the bedrock (fig. TS14K–5). This method requires the larger rock be placed along the outer edge of the toe. The steel dowels are placed in position downslope against the large rocks that act as key stones in the toe to support the remainder of the rock riprap on the slope above. A modification of this approach is to drill holes into the toe rock and fit the stones over the steel dowels.

Precast toe blocks method

This method uses precast concrete blocks (fig. TS14K–6) to anchor the bottom row of riprap. The precast blocks should be 12 inches square and 5 feet long. Re-inforcing 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–6  Precast toe block method

Plan view
Installation details of concrete toe blocks

Section
Installation details of concrete toe blocks

1-in diameter bar

12 in

24 in

5 ft

6.5 ft

4 in

2V:1H maximum slope

Backfill after riprap is placed

Bedrock

Concrete toe block

4.5 in

12 in

18 in

12 in

5 ft

6.5 ft

2V:1H maximum slope

Backfill after riprap is placed

Bedrock

Concrete toe block

4.5 in

12 in

18 in

12 in
Other structural treatments

There are many structural streambank treatment techniques which involve the use of riprap. Several are briefly described, and others are described elsewhere in NEH654.14.

Stone with soil bioengineering

Combining rock with soil bioengineering treatments can achieve benefits from both techniques. Soil bioengineering is covered in more detail in NEH654 TS14J. The inert rock material often provides immediate toe protection, while the living plant materials protect, reinforce, and stabilize the banks.

Figure TS14K–7 shows a stone toe and live poles. The stone is keyed into the bed below an anticipated scour depth. Live poles can be installed with the aid of a waterjet stinger.

Figure TS14K–8 shows a brush layer being installed over a stone toe. Since the stone is not keyed into the bed, additional stone is placed in the toe. As the bed is scoured adjacent to the bank protection, this additional stone is available to fall into the scour hole.

Figure TS14K–9 shows a vertical bundle being installed under a stone toe. The bundles are placed in trenches which are then filled with soil. This minimizes potential damage to the live material during stone placement, as well as maximizes soil-to-stem contact.

Longitudinal peak stone toe

Longitudinal peak stone toe (LPST) involves the placement of a windrow of stone in a peak ridge along the toe of an eroding bank. The top of the stone is typically one-third to two-thirds of the bank height (Biedenharn, Elliott, and Watson 1997). LPST is particularly applicable where the upper bank is fairly stable, and the erosion is due to mass wasting from the toe of the bank. This technique protects the toe, while allowing the upper bank to stabilize on its own.

The main advantage of this technique is cost savings. An LPST is designed by specifying a weight or volume of rock to be placed along the length of the project reach, rather than finished elevations or dimensions.
On moderate-sized tributaries along the Mississippi River, typical applications can be 1 to 2 tons per linear foot, resulting in a triangular peak between 3 and 5 feet above the streambed (Biedenharn, Elliott, and Watson 1997). Usually, this simple technique is constructed by dumping stone from the bank. Since neither a filter layer nor geotextile fabric is used, a self-filtering, well-graded quarry run stone is specified. This technique depends on the rapid establishment of vegetation landward from the stone. Therefore, it is important to minimize disturbance of natural vegetation during installation, and it may be advisable to consider the addition of soil bioengineering practices.

An LPST is often enhanced with the inclusion of woody debris and stone spurs along the length. These encourage deposition along the toe, create edge habitat, and move the higher velocity flow away from the bank.

**Timber and rock cribbing**

Timber cribbing backfilled with rock and coarse gravel is a traditional bank protection technique. This type of protection was popular many years ago when hand labor was more readily used in streambank protection. It has held up reasonably well, but becomes difficult to repair and maintain with age. Figure TS14K–10 illustrates a method of timber and rock cribbing.

The construction of a timber and rock crib requires considerable hand labor, and its useful life depends on the length of time the logs will hold the rock in place before rotting. As with gabions, the cribbing allows for the protection of unstable banks with stones that would be too small if used in a riprap revetment. While not exactly duplicating a riprap revetment, similar design characteristics are required for its design, such as scour, filtration, drainage, and length.

---

**Figure TS14K–10** Timber and rock cribbing

Start installation safely upstream from active erosion point.

End installation at least 20 ft downstream from active erosion point.

Eight–12-in diameter logs

1/2-in drift pins to penetrate three logs

Flow

8−12 ft

6−8 ft

Front view

Side view
**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.

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.

### Table TS14K–1

<table>
<thead>
<tr>
<th>Gabion</th>
<th>Predominant rock size (in)</th>
<th>Minimum rock dimension (in)</th>
<th>Maximum rock dimension (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12-, 18-, or 36-in basket</td>
<td>4 to 8</td>
<td>4</td>
<td>9</td>
</tr>
<tr>
<td>6-, 9-, or 12-in mattress</td>
<td>3 to 6</td>
<td>3</td>
<td>7</td>
</tr>
</tbody>
</table>
Various types of fasteners and lacing are used to assemble and secure gabion baskets and mattresses. The manufacturer's recommendations should be followed along with the applicable provisions in CS #64.

Vegetated gabion

In some locations, traditional gabions may be unacceptable from either an aesthetic or ecological perspective. A modification to traditional gabion protection that may satisfy these concerns is the vegetated gabion. A vegetated gabion incorporates topsoil into the void spaces of the gabion. The resulting gabion volume consists of 30 to 40 percent soil that allows root propagation between the stones. The resulting structure is interlocked with stone, wire, and roots (fig. TS14K–12).

Various commercial products, such as the Maccaferri Green Gabion™, provide improved shapes and an organic fiber matting to hold the soil in place while the plants become established. Figure TS14K–13 illustrates the assembly steps of such a gabion.

Grouted riprap

Grouted riprap is a riprap bed where the voids have been filled with concrete. It is often used where the required stone size cannot be obtained or at sites where a significant and damaging debris load is expected. Typical applications include grade protection, bank protection, spillways, inlets to debris basins, and as a repair to conventional riprap structures that have been damaged by high velocity flows. Culvert outfalls and ditch linings have also been constructed with grouted riprap. It has also been used to provide improved recreational access across riprap revetments.

While the stone used for a grouted riprap installation can be smaller than what is required for a loose riprap installation, there is no available guidance that specifies a minimum size. Sizing is usually based on experience with similar projects in the area. The stone used should be as coarse as possible to allow for deep penetration of the grout. A general recommendation is that less than 5 percent of the stone should be less than 2 inches in diameter. Stone quality should be similar to that specified for conventional riprap structures.

The grout strength is typically 2,000 to 2,500 pounds per square inch. The grout must fully penetrate the stone to the subbase. Shoveling the grout over the stone may not fully penetrate the riprap. An immersion or pencil vibrator is often used to ensure that the voids between the stones are filled. The concrete mix should have a slump of 5 to 7 inches to allow for proper penetration. The maximum aggregate in the mix should be three-fourths inch. Typically, the grout is placed up to the top of the stones. However, in some applications,
Figure TS14K–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.

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.

**Habitat enhancement with stone**

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.
Technical Supplement 14L

Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects
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.
# Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects

## Contents

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Purpose</td>
<td>TS14L-1</td>
</tr>
<tr>
<td>Introduction</td>
<td>TS14L-1</td>
</tr>
<tr>
<td>Applications</td>
<td>TS14L-1</td>
</tr>
<tr>
<td>Materials</td>
<td>TS14L-6</td>
</tr>
<tr>
<td>- Blocks</td>
<td>TS14L-6</td>
</tr>
<tr>
<td>- Connections</td>
<td>TS14L-6</td>
</tr>
<tr>
<td>- Geotextiles</td>
<td>TS14L-6</td>
</tr>
<tr>
<td>- Granular filter</td>
<td>TS14L-6</td>
</tr>
<tr>
<td>- Performance testing and evaluation</td>
<td>TS14L-9</td>
</tr>
<tr>
<td>Design procedure</td>
<td>TS14L-11</td>
</tr>
<tr>
<td>- Factor of safety</td>
<td>TS14L-11</td>
</tr>
<tr>
<td>- Blocks</td>
<td>TS14L-11</td>
</tr>
<tr>
<td>- Filter</td>
<td>TS14L-11</td>
</tr>
<tr>
<td>Specifying ACB revetment systems</td>
<td>TS14L-14</td>
</tr>
<tr>
<td>- Blocks</td>
<td>TS14L-14</td>
</tr>
<tr>
<td>- Connections</td>
<td>TS14L-14</td>
</tr>
<tr>
<td>- Geotextile</td>
<td>TS14L-14</td>
</tr>
<tr>
<td>- Testing</td>
<td>TS14L-14</td>
</tr>
<tr>
<td>- Design</td>
<td>TS14L-14</td>
</tr>
<tr>
<td>Installation</td>
<td>TS14L-14</td>
</tr>
<tr>
<td>- Subgrade preparation</td>
<td>TS14L-14</td>
</tr>
<tr>
<td>- Geotextile placement</td>
<td>TS14L-16</td>
</tr>
<tr>
<td>- Placement of the ACB</td>
<td>TS14L-16</td>
</tr>
<tr>
<td>- Termination</td>
<td>TS14L-16</td>
</tr>
<tr>
<td>- Anchor penetrations</td>
<td>TS14L-16</td>
</tr>
<tr>
<td>- Filling</td>
<td>TS14L-16</td>
</tr>
<tr>
<td>ACB example calculations</td>
<td>TS14L-19</td>
</tr>
<tr>
<td>Conclusion</td>
<td>TS14L-21</td>
</tr>
</tbody>
</table>
Technical Supplement 14L

Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects

Part 654 National Engineering Handbook

Tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Table TS14L–1</td>
<td>Steel cable specifications</td>
<td>TS14L–8</td>
</tr>
<tr>
<td>Table TS14L–2</td>
<td>Polyester cable specifications</td>
<td>TS14L–8</td>
</tr>
<tr>
<td>Table TS14L–3</td>
<td>Design equations for ACB revetment systems</td>
<td>TS14L–13</td>
</tr>
<tr>
<td>Table TS14L–4</td>
<td>Block physical requirements</td>
<td>TS14L–15</td>
</tr>
<tr>
<td>Table TS14L–5</td>
<td>NRCS specifications for woven geotextiles</td>
<td>TS14L–15</td>
</tr>
<tr>
<td>Table TS14L–6</td>
<td>NRCS specifications for nonwoven geotextiles</td>
<td>TS14L–15</td>
</tr>
</tbody>
</table>

Figures

<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Figure TS14L–1</td>
<td>Armoring the entire cross section</td>
<td>TS14L–2</td>
</tr>
<tr>
<td>Figure TS14L–2</td>
<td>Armoring the toe and lower slope cross section</td>
<td>TS14L–2</td>
</tr>
<tr>
<td>Figure TS14L–3</td>
<td>Armoring the toe and slide slope cross section</td>
<td>TS14L–3</td>
</tr>
<tr>
<td>Figure TS14L–4</td>
<td>Streambed grade stabilization profile</td>
<td>TS14L–3</td>
</tr>
<tr>
<td>Figure TS14L–5</td>
<td>Armoring of pipe/culvert outlets profile</td>
<td>TS14L–4</td>
</tr>
<tr>
<td>Figure TS14L–6</td>
<td>Scour protection around bridge piers plan</td>
<td>TS14L–5</td>
</tr>
<tr>
<td>Figure TS14L–7</td>
<td>Examples of ACB blocks</td>
<td>TS14L–7</td>
</tr>
<tr>
<td>Figure TS14L–8</td>
<td>Steel cables</td>
<td>TS14L–8</td>
</tr>
<tr>
<td>Figure TS14L–9</td>
<td>Polyester cables</td>
<td>TS14L–8</td>
</tr>
<tr>
<td>Figure TS14L–10</td>
<td>ACB section with a geotextile filter and combination geotextile and granular filter</td>
<td>TS14L–8</td>
</tr>
<tr>
<td>Figure TS14L–11</td>
<td>ACBs in combination with granular and geotextile filter</td>
<td>TS14L–9</td>
</tr>
<tr>
<td>Figure TS14L–12</td>
<td>Schematic of a typical laboratory test flume for ACB performance testing</td>
<td>TS14L–9</td>
</tr>
<tr>
<td>Figure TS14L–13</td>
<td>Forces on an ACB revetment system during performance test</td>
<td>TS14L–10</td>
</tr>
<tr>
<td>-----------------</td>
<td>----------------------------------------------------------</td>
<td>-----------</td>
</tr>
<tr>
<td>Figure TS14L–14</td>
<td>Forces on a protruding block</td>
<td>TS14L–10</td>
</tr>
<tr>
<td>Figure TS14L–15</td>
<td>Block on a side slope with design variables</td>
<td>TS14L–12</td>
</tr>
<tr>
<td>Figure TS14L–16</td>
<td>Block moment arms</td>
<td>TS14L–12</td>
</tr>
<tr>
<td>Figure TS14L–17</td>
<td>Granular filter encapsulation by a geotextile</td>
<td>TS14L–17</td>
</tr>
<tr>
<td>Figure TS14L–18</td>
<td>Spreader bar for placement of cabled mats</td>
<td>TS14L–17</td>
</tr>
<tr>
<td>Figure TS14L–19</td>
<td>Hand placement of ACB blocks</td>
<td>TS14L–17</td>
</tr>
<tr>
<td>Figure TS14L–20</td>
<td>ACB termination trench</td>
<td>TS14L–18</td>
</tr>
<tr>
<td>Figure TS14L–21</td>
<td>Filling ACBs with top soil</td>
<td>TS14L–18</td>
</tr>
<tr>
<td>Figure TS14L–22</td>
<td>ACB revetment system 1 year after completion</td>
<td>TS14L–18</td>
</tr>
<tr>
<td>Figure TS14L–23</td>
<td>ACB revetment system 2 years after completion</td>
<td>TS14L–18</td>
</tr>
</tbody>
</table>
Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects

Purpose

A variety of natural and manufactured materials can provide erosion protection for stream restoration and stabilization projects. One of these products is the articulating concrete block (ACB) revetment system. An ACB revetment system is a matrix of interconnected concrete block units installed to provide an erosion resistant revetment with specific hydraulic characteristics. It is static protection and is applicable in high risk applications where no additional bank or grade movement is allowable. This technical supplement describes the ACBs currently available and some of the benefits of their use. The system consists of concrete blocks, a filter (typically a geotextile), and cables in some products. A summary of testing for hydraulic performance is presented along with a design procedure for open channel flow. Critical installation features are described for typical installations including subgrade preparation, ancillary components (such as drainage layers), filter placement, ACB placement, system termination, anchors, and penetrations.

Introduction

Stream restoration and stabilization may require the use of armoring countermeasures to provide lateral or vertical stability to a stream. Armoring countermeasures include concrete lining and other rigid revetments, rock riprap, gabion baskets, gabion mattresses, or ACB revetment systems. These countermeasures result in a statically stable stream within the armored area. Armoring countermeasures provide permanent erosion protection to underlying soil from the forces of flowing water. Armoring countermeasures may be used when vegetation and other soil bioengineering practices are not suitable or unstable under the stress or duration of the design event or where the consequences of failure are unacceptable. The designer should keep in mind that since its use results in a static section, other stability and ecological issues may become a concern. Typical applications of the ACB revetment system include entire channel cross-sectional protection, toe and lower side slope protection, and grade stabilization structures.

Applications

ACB revetment systems have been used in a variety of applications for streambank stabilization and restoration projects (figs. TS14L–1 through TS14L–6). These applications include:

- armoring the entire cross section
- armoring the toe and lower slope
- armoring the toe and side slope
- streambed grade stabilization
- armoring of pipe/culvert outlets
- scour protection around bridge piers
Figure TS14L–1  Armoring the entire cross section

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

d = maximum depth of scour
Figure TS14L–3  Armoring the toe and slide slope cross section

Figure TS14L–4  Streambed grade stabilization profile
Figure TS14L–5  Armoring of pipe/culvert outlets profile

Plan view

Section A–A
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 non-cabled. 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™
Technical Supplement 14L
Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects

**Figure TS14L–8** Steel cables

**Figure TS14L–9** Polyester cables

<table>
<thead>
<tr>
<th>Table TS14L–1</th>
<th>Steel cable specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter</td>
<td>Construction</td>
</tr>
<tr>
<td>1/8 in</td>
<td>1 by 19</td>
</tr>
<tr>
<td>5/32 in</td>
<td>1 by 19</td>
</tr>
<tr>
<td>3/16 in</td>
<td>1 by 19</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table TS14L–2</th>
<th>Polyester cable specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cable diameter</td>
<td>Average strength</td>
</tr>
<tr>
<td>(in)</td>
<td>(lb)</td>
</tr>
<tr>
<td>1/4</td>
<td>3,700</td>
</tr>
<tr>
<td>5/16</td>
<td>7,000</td>
</tr>
<tr>
<td>3/8</td>
<td>10,000</td>
</tr>
<tr>
<td>1/2</td>
<td>15,000</td>
</tr>
</tbody>
</table>

**Figure TS14L–10** ACB section with a geotextile filter and combination geotextile and granular filter (*Figure courtesy of HCFCD*)
Performance testing and evaluation

Due to the proprietary nature and unique characteristics of the ACB revetment systems available, a hydraulic stability test should be completed on each family of blocks. The hydraulic stability test should be conducted in accordance with U.S. Department of Transportation Federal Highway Administration (FHWA) RD–89–199 (Clopper 1989). Research conducted throughout the 1980s (Clopper and Chen 1988; Clopper 1989) led to a definition of failure for ACB revetment systems as the local loss of intimate contact between the ACB and the subgrade. The FHWA study (Clopper 1989) identified the following four conditions which may lead to this definition of failure:

- loss of soil beneath the system by gradual erosion beneath the system or washout through the system at joints and open cells
- deformation of the subgrade due to liquefaction and shallow slip failures caused by the ingress of water beneath the system (especially in silty soils on steep slopes)
- loss of block or a group of blocks (uncabled systems) which directly exposes the subgrade to the flow
- flow beneath the ACB causing uplift pressures and separation of the block from the subgrade

Although loss of intimate contact may not lead to total failure of the system, the stability and continued performance of the system has been compromised.

Each ACB revetment system obtains its stability from a unique set of weight, interblock restraint, geometry, and block-to-block articulation. Therefore, laboratory testing of each family of ACB revetment systems is required to determine the critical shear stress. A schematic of a typical laboratory test flume is shown in figure TS14L–12.
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:

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

(eq. TS14L–2)

Where:
- $F'_D$ = drag force due to block protrusion (lb)
- $C_D$ = drag coefficient ($C_D \approx 1.0$)
- $\Delta Z$ = height of protrusion (ft)
- $b$ = block width perpendicular to flow (ft)
- $\rho$ = 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:

$$\tau_{CU} = \tau_{CT} \times \left( \frac{W_{S1} \ell_2 \cos \theta_U - \ell_1 \sin \theta_U}{W_{ST} \ell_2 \cos \theta_T - \ell_1 \sin \theta_T} \right)$$

(eq. TS14L–3)

Where:
- $\tau_{CU}$ = critical shear stress for untested bed slope (lb/ft$^2$)
- $\tau_{CT}$ = critical shear stress for tested bed slope (lb/ft$^2$)
- $\theta_U$ = untested bed slope (degrees)
- $\theta_T$ = tested bed slope (degrees)
- $\ell_x$ = 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:

$$\tau_{CU} = \tau_{CT} \times \left( \frac{W_{S1} \ell_2 \cos \theta_U - \ell_1 \sin \theta_U}{W_{ST} \ell_2 \cos \theta_T - \ell_1 \sin \theta_T} \right)$$

(eq. TS14L–4)
where:
\[ \tau_{CU} = \text{critical shear stress for untested block (lb/ft}^2\text{)} \]
\[ \tau_{CT} = \text{critical shear stress for tested block (lb/ft}^2\text{)} \]
\[ W_{SU} = \text{submerged weight of untested blocks (lb)} \]
\[ W_{ST} = \text{submerged tested blocks (lb)} \]
\[ L_{xU} = \text{moment arms of untested blocks} \]
\[ L_{xT} = \text{moment arms of tested blocks (ft)} \]

The moment arms used in these two equations should apply to the orientation of the block during testing and are not necessarily the same as those suggested later in the document for design.

**Design procedure**

The design of ACB revetment systems must be based on hydraulic analyses of the open channel during the design event. The hydraulic analyses should provide the shear stress and velocity associated with the design event. An example calculation is provided at the end of this technical supplement. The cross-sectional average shear stress may be used for most open channel flow applications. For applications such as bends, confluences, flow constrictions, or flow obstructions, a more detailed, area-specific hydraulic analysis should be considered. Site aesthetics and impacts to habitat should also be considered.

**Factor of safety**

The design engineer must determine the factor of safety to be used for a particular project. The determination should consider the risks associated with the failure of the ACB revetment system, complexity of the hydraulic system, the uncertainties in hydrologic and hydraulic analyses, and uncertainties associated with ACB revetment system installation. Typically, a minimum factor of safety of 1.5 is used for stream revetment project design. A higher factor of safety of 2.0 should be considered for protection around bridge piers, abutments, at channel bends, or other complex hydraulic systems. A systematic procedure to select a project-specific factor of safety is presented in HCFCD (2001).

**Blocks**

Failure (loss of intimate contact) is typically the result of the overturning of a block or group of blocks about the downstream contact point of the block. The hydraulic stability of a block on a channel side slope is a function of the magnitude and direction of stream velocity and shear stress, the depth of flow, channel side slope, channel bed slope, interblock restraint, block geometric properties, and the weight of the block. The definition of the forces, dimensions, and angles used in the equation for the factor of safety are depicted in figures TS14L–15 and TS14L–16. The factor of safety equations are defined in table TS14L–3 (HCFCD 2001).

**Filter**

An appropriate filter design is critical to the successful performance of the ACB revetment system. Design of both a geotextile filter and a granular filter includes determining criteria for filtering and permeability.

References available for design of a geotextile filter include HCFCD (2001); U.S. Department of Agriculture Soil Conservation Service (SCS) (1991); AASHTO (2000), and USACE (1995b). Each of the references includes an analysis of the appropriate geotextile Apparent Opening Size and its permeability. The maximum Apparent Opening Size will allow suitable retention of soil particles, while the minimum geotextile permeability will allow the free flow of water without a buildup of excessive hydrostatic pressure.

Granular filter design criteria are presented in the Natural Resources Conservation Service (NRCS) National Engineering Handbook (NEH), part 633, chapter 26, Gradation design of sand and gravel filters (USDA NRCS 1994). This document provides filter criteria based on the percent finer than the number 200 sieve of the subgrade soil. It also recommends a minimum permeability for any subgrade soil.
Use of Articulating Concrete Block Revetment Systems for Stream Restoration and Stabilization Projects

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

<table>
<thead>
<tr>
<th>Expression</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>FS = ( \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{\ell_3 F'_D \cos \delta + \ell_4 F'_L}{\ell_1 W_S} )</td>
<td>factor of safety</td>
</tr>
<tr>
<td>( \delta + \beta + \theta = 90^\circ ) or ( \frac{\pi}{2} ) radians</td>
<td></td>
</tr>
<tr>
<td>( \eta_1 = \left( \frac{\ell_4}{\ell_3} + \frac{\sin(\theta_0 + \theta + \beta)}{\eta_0 \left( \frac{\ell_2}{\ell_1} \right)} \right) \eta_0 )</td>
<td>stability number for a sloped surface</td>
</tr>
<tr>
<td>( \beta = \arctan \left( \frac{\cos(\theta_0 + \theta)}{\left( \frac{\ell_4}{\ell_3} + \frac{\sqrt{1 - a_\theta^2 \cos \beta}}{\eta_0 \left( \frac{\ell_2}{\ell_1} \right)} + \sin(\theta_0 + \theta) \right)} \right) )</td>
<td>angle between side slope projection of W_S and the vertical</td>
</tr>
<tr>
<td>( \theta = \arctan \left( \frac{\tan \theta_o}{\tan \theta_1} \right) = \arctan \left( \frac{\tan \theta_o}{\tan \theta_1} \right) )</td>
<td>angle between side slope projection of W_S and the vertical</td>
</tr>
<tr>
<td>( a_\theta = \sqrt{\cos^2 \theta_1 - \sin^2 \theta_0} )</td>
<td></td>
</tr>
<tr>
<td>( \eta_0 = \frac{\tau_{des}}{\tau_c} )</td>
<td>mass density of water (1.94 slugs/ft^3)</td>
</tr>
<tr>
<td>( F'_L = F'<em>D = 0.5 \times (\Delta Z) b \rho V</em>{\text{des}}^2 )</td>
<td>critical shear stress for block on a horizontal surface (lb/ft^2)</td>
</tr>
<tr>
<td>( W_S = W \times \left( \frac{G_c - 1}{G_c} \right) )</td>
<td>design shear stress (lb/ft^2)</td>
</tr>
</tbody>
</table>

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 \( \theta_1 \cong 0 \)

\( \rho = \) mass density of water (1.94 slugs/ft^3)

\( \tau_c = \) critical shear stress for block on a horizontal surface (lb/ft^2)

\( \tau_{\text{des}} = \) 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/ft²)
- 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

<table>
<thead>
<tr>
<th>Minimum compressive strength lb/in²</th>
<th>Maximum water absorption lb/ft²</th>
<th>Minimum density lb/ft³</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 unit avg. 4,000</td>
<td>3 unit avg. Individual unit 9.1</td>
<td>3 unit avg. Individual unit 130</td>
</tr>
<tr>
<td>Individual unit 3,500</td>
<td>Individual unit 11.7</td>
<td>Individual unit 125</td>
</tr>
</tbody>
</table>

Table TS14L–5  NRCS specifications for woven geotextiles

<table>
<thead>
<tr>
<th>Property</th>
<th>Test method</th>
<th>Class I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength (lb) ⅓</td>
<td>ASTM D4632</td>
<td>200 minimum in any principal direction</td>
</tr>
<tr>
<td>Elongation at failure (%) ⅓</td>
<td>ASTM D4632</td>
<td>&lt;50</td>
</tr>
<tr>
<td>Puncture (lb) ⅓</td>
<td>ASTM D4833</td>
<td>90 minimum</td>
</tr>
<tr>
<td>UV (% residual tensile strength)</td>
<td>ASTM D4355 150-hr exposure</td>
<td>70 minimum</td>
</tr>
<tr>
<td>Apparent opening size ⅔</td>
<td>ASTM D4751</td>
<td>As specified, but no smaller than 0.212 mm (#70) ⅔</td>
</tr>
<tr>
<td>Percent open area (%)</td>
<td>CWO-02215</td>
<td>4.0 minimum</td>
</tr>
<tr>
<td>Permittivity s⁻¹</td>
<td>ASTM D4491</td>
<td>0.10 minimum</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Property</th>
<th>Test method</th>
<th>Class I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tensile strength (lb) ⅓</td>
<td>ASTM D4632</td>
<td>180</td>
</tr>
<tr>
<td>Elongation at failure (%) ⅓</td>
<td>ASTM D4632</td>
<td>&gt;50</td>
</tr>
<tr>
<td>Puncture (lb) ⅓</td>
<td>ASTM D4833</td>
<td>80 minimum</td>
</tr>
<tr>
<td>UV (% residual tensile strength)</td>
<td>ASTM D4355 150-hr exposure</td>
<td>70 minimum</td>
</tr>
<tr>
<td>Apparent opening size ⅔</td>
<td>ASTM D4751</td>
<td>As specified, max. #40 ⅔</td>
</tr>
<tr>
<td>Permittivity s⁻¹</td>
<td>ASTM D4491</td>
<td>0.70 minimum</td>
</tr>
</tbody>
</table>

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.
Granular filter encapsulation by a geotextile (Figure courtesy of HCFCD)

- Edges of adjoining geotextiles wrapped under downstream encapsulation cell
- Granular transition layer
- Flow
- Geotextile
- 1 ft min
- 20 ft max

Figure TS14L–18  Spreader bar for placement of cabled mats

Figure TS14L–19  Hand placement of ACB blocks
Figure TS14L–20  ACB termination trench

Figure TS14L–21  Filling ACBs with top soil (Photo courtesy of Joe Polulech)

Figure TS14L–22  ACB revetment system 1 year after completion (Photo courtesy of Joe Polulech)

Figure TS14L–23  ACB revetment system 2 years after completion (Photo courtesy of Joe Polulech)
**ACB example calculations**

*Given:* An ACB revetment system is to be installed on the side slopes of a stream channel in the vicinity of a highway bridge. A hydraulic analysis has been conducted, and the following conditions are recommended for the design:

- Design velocity: 11 ft/s
- Design shear stress: 2 lb/ft²
- 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/ft²
- Specific gravity of concrete: 2.2

*Design velocity:* 11 ft/s  
*Design shear stress:* 2 lb/ft²
*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/ft²
- 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.

\[
\ell_1 = \frac{1}{2} \times \text{block thickness} = \frac{1}{2} \times \frac{4.75}{12} = 0.198
\]

(eq. TS14L–5)

\[
\ell_2 = \ell_4 = \ell_3 = 0.8 \times \text{block thickness}
\]

(eq. TS14L–6)

(eq. TS14L–7)

**Step 2** Calculate the submerged unit weight of block.

\[
W_S = W \times \left( \frac{G_c - 1}{G_c} \right)
\]

\[
= 35 \times \frac{2.2 - 1}{2.2}
\]

(eq. TS14L–8)

\[
= 19.1 \text{ lb}
\]

**Step 3** Calculate the stability number on a horizontal surface.

\[
\eta_0 = \frac{\tau_{\text{des}}}{\tau_c}
\]

\[
= \frac{2}{15}
\]

(eq. TS14L–9)

\[
= 0.133
\]

**Step 4** Calculate additional lift and drag forces from block protrusion.

\[
F'_L = F'_D = 0.5 \times (\Delta\bar{z}) \rho \bar{V}_{\text{des}}^2
\]

\[
= 0.5 \times \left( \frac{0.5}{12} \right) \times 1.1 \times 1.94 \times 11^2
\]

\[
= 5.4 \text{ lb}
\]

**Step 5** Calculate \(a_\theta\).

\[
a_\theta = \sqrt{\cos^2 \theta_1 - \sin^2 \theta_0}
\]

(eq. TS14L–11)

\[
\theta_0 = \text{ATAN}(\text{bed slope}) = \text{ATAN}(0.03)
\]

\[
= 1.72^\circ
\]

(eq. TS14L–12)

\[
\theta_1 = \text{ATAN}\left( \frac{1}{\text{channel slope}} \right) = \text{ATAN}\left( \frac{1}{2} \right)
\]

\[
= 26.57^\circ
\]

\[
\alpha_\theta = \sqrt{\cos^2 (26.57^\circ) - \sin^2 (1.72^\circ)}
\]

\[
= 0.894
\]
Step 6  Calculate \( \theta \).

\[
\theta = \arctan \left( \frac{\sin \theta_0 \times \cos \theta_1}{\sin \theta_1 \times \cos \theta_0} \right) \quad \text{(eq. TS14L–13)}
\]

\[
= \arctan \left( \frac{\tan \theta_0}{\tan \theta_1} \right)
\]

\[
= \arctan \left( \frac{\tan 1.72^\circ}{\tan 26.57^\circ} \right)
\]

\[
= 3.43^\circ
\]

Step 7  Calculate \( \beta \).

\[
\beta = \arctan \left( \frac{\cos (\theta_0 + \theta)}{(\frac{\ell_4}{\ell_3} + 1) \sqrt{1 - a_0^2} + \sin (\theta_0 + \theta)} \right) \quad \text{(eq. TS14L–14)}
\]

\[
= \arctan \left( \frac{0.733}{0.317 + 1} \sqrt{1 - 0.894^2} \right. + \left. \sin (1.72^\circ + 3.43^\circ) \right)
\]

\[
= 17.82^\circ \quad \text{(eq. TS14L–15)}
\]

Step 8  Calculate stability number for a sloped surface \( \eta_1 \):}

\[
\eta_1 = \left( \frac{\ell_4}{\ell_3} + 1 \right) \eta_0 + \left( \frac{0.733}{0.317 + 1} \right) + \left( \frac{1.72^\circ + 3.43^\circ + 17.82^\circ}{0.733 + 1} \right)
\]

\[
= 0.109 \quad \text{(eq. TS14L–16)}
\]

Step 9  Calculate angle between drag force and block motion, \( \delta \).

\[
\delta + \beta + \theta = 90^\circ \quad \text{(eq. TS14L–17)}
\]

\[
\delta = 90^\circ - \beta - \theta
\]

So,

\[
90^\circ - 17.82^\circ - 3.43^\circ = -68.75^\circ \quad \text{(eq. TS14L–18)}
\]

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:

\[
FS = \frac{\left( \frac{\ell_2}{\ell_1} \right)^3 \theta_0}{\sqrt{1 - a_0^2} \cos \beta + \eta_1 \left( \frac{\ell_2}{\ell_1} \right) + \left( \frac{\ell_4 \cos \delta + \ell_4 F_d}{\ell_1 \mathbb{W}_s} \right)}
\]

\[
FS = \frac{0.733 \times 0.894}{\sqrt{1 - 0.894^2} \cos 17.82^\circ + 0.109 \left( \frac{0.733}{0.198} \right) + \frac{0.317 \times 5.379 \times \cos 68.75^\circ + 0.733 \times 5.379}{0.198 \times 1.9}
\]
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.