# The WES Stream Investigation and Streambank Stabilization Handbook

*Presented by*

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# THE WES STREAM INVESTIGATION

# AND

# STREAMBANK STABILIZATION HANDBOOK

by

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## PREFACE

The challenge of stabilizing an entire watershed, stream, or even small section of stream is a daunting, difficult, and formidable task. In recognition of the serious environmental and economic losses occurring throughout the Nation as the result of streambank erosion, the Environmental Protection Agency (EPA) contracted with the U.S. Army Engineer Waterways Experiment Station (WES) to develop a streambank protection manual. The technical contact with the EPA for this work was Dr. Christopher F. Zabawa.

This manual was written by Dr. David S. Biedenharn (WES), Mr. Dave Derrick (WES), Mr. Charles Elliott (Private Consultant), and Dr. Chester Watson (Colorado State University). This work was conducted under the direction of Dr. James Houston, Director of the Coastal & Hydraulics Laboratory, Dr. Phil Combs, Chief of the River and Structures Division, and Mr. Mike Trawle, Chief of the River Sedimentation Branch. The principle investigator for this study was Dr. Nolan Raphelt. At the time of publication of this report, the Director of WES was Dr. Robert W. Whalin.

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# CHAPTER 1

# INTRODUCTION

One of the most challenging problems of environmental hazard management faced in the United States today is the stabilization of eroding streambanks. The U.S. Army Corps of Engineers estimated that in 1981, of the 3.5 million stream-miles of channel in the United States, approximately 575,000 bank-miles were eroding. This erosion results in serious economic losses of private and public lands, disrupts transportation infrastructure (bridges, pipelines, railroad lines, etc.), and degrades water quality. The sediments that are eroded from these channels are deposited downstream in flood control and navigation channels, and in valuable wetland areas. Consequently, streambank erosion is not simply a local problem which only affects a few landowners, but rather, produces system-wide economic and environmental consequences that affects all taxpayers.

Over the past several years there has been a growing interest in the development of low-cost, environmentally friendly bank protection techniques, that are suitable for landowners, local governments and other groups with limited resources. Many of these techniques have been quite successful, while others have not performed as intended. At the other end of the spectrum is reliance solely on complete riprap paving when another, less costly, and perhaps more environmentally acceptable technique would be just as effective. Unfortunately, many designers, after having success with a particular protection technique, make the mistake of trying to apply this single method to all situations, regardless of the site conditions. This often results in an ineffective design leading to structure failure. Even if the appropriate protection technique is selected, failure may still occur if proper design procedures are not followed. However, there is no published, definitive guidance or criteria that addresses the suitability and effectiveness of the various bank stabilization techniques for varying site conditions and project constraints. This manual provides the necessary guidance for making intelligent decisions when selecting and designing streambank protection measures or deciding not to install bank protection.

## 1.1 PURPOSE

Bank stabilization structures are often considered to be very simple features requiring very little planning and design effort. However, the hydraulic and geomorphic processes associated with these structures are as complex and challenging as those of many of the more elaborate hydraulic structures, and in many cases their design is even more complicated due to the lack of definitive design guidance. This manual is designed to provide general guidance for the design, construction, and monitoring of streambank protection projects. It also introduces the reader to the basic concepts of channel stability, and procedures for understanding and analyzing stream processes.

## 1.2 SCOPE

There are hundreds of different types of possible bank stabilization techniques which are used on a wide range of stream types and physical environments ranging from the Mississippi River to small ephemeral streams draining only a few square miles. A range of structure types and applications is presented, from traditional techniques such as riprap bank paving, stone dikes, and retards, to the low-cost and innovative techniques such as bendway weirs, and bio-engineering measures. This is a comprehensive manual covering a wide range of techniques and design guidance that will be of benefit to all groups, large or small, undertaking a streambank protection project. Whenever possible, layman's language is used in this manual, so that the information and guidance contained herein can be utilized by the broadest possible audience.

# CHAPTER 2

# FUNDAMENTALS OF FLUVIAL GEOMORPHOLOGY AND CHANNEL PROCESSES

## 2.1 FLUVIAL GEOMORPHOLOGY

Webster's New World Dictionary defines ***fluvial*** as: *of, found in, or produced by a river or rivers*. The same reference defines ***morphology*** as: ***any scientific study of form and structure, as in physical geography, etc***. With a little guess work, we can correctly extrapolate that fluvial geomorphology is the study of the form and structure of the surface of the earth (geo) as affected by flowing water. Another definition, although given in jest, may be the one most remembered after this next section. *Geomorphology is the triumph of terminology over common sense*. An equally important term is the ***fluvial system***. A system is an arrangement of things to form a whole. The primary goal on which we want to focus in this section is that you are working with a system and the complete system must be considered.

### 2.1.1 BASIC CONCEPTS

Six basic concepts that should be considered in working with watersheds and rivers are: 1) the river is only part of a system, 2) the system is dynamic, 3) the system behaves with complexity, 4) geomorphic thresholds exist, and when exceeded, can result in abrupt changes, 5) geomorphic analyses provide a historical prospective and we must be aware of the time scale, and 6) the scale of the stream must be considered. Is the stream a small, mountain meadow trout stream, or is it the Mississippi River?

#### 2.1.1.1 The Fluvial System

Schumm (1977) provides an idealized sketch of a fluvial system (Figure 2.1). The parts are referred to as:

Zone 1 - the upper portion of the system that is the watershed or drainage basin; this portion of the system functions as the sediment supply.

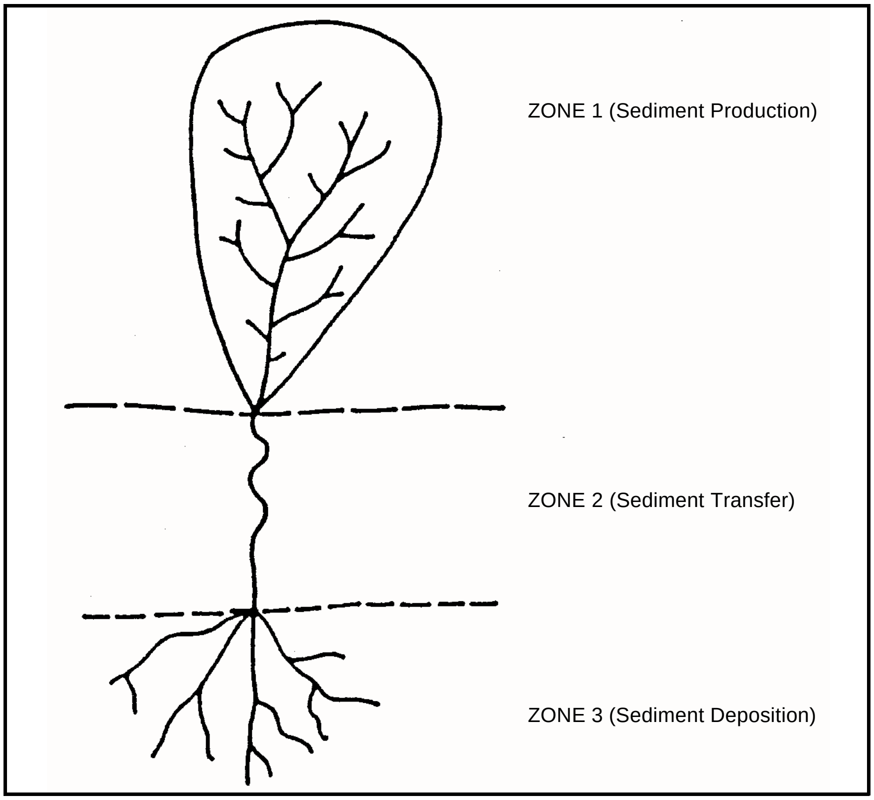



Figure 2.1  The Fluvial System (after Schumm, 1977)

Zone 2 - the middle portion of the system that is the river; this portion of the system functions as the sediment transfer zone.

Zone 3 - the lower portion of the system may be a delta, wetland, lake, or reservoir; this portion of the system functions as the area of deposition.

These three zones are idealized, because in actual conditions sediments can be stored, eroded, and transported in all zones. However, within each zone one process is usually dominant. For our purposes in planning channel stabilization, we are primarily concerned with Zone 2, the transfer zone. We may need to treat only a small length of a stream bank (Zone 2) to solve a local instability problem; however, from a system viewpoint we must insure that our plan does not interfere with the transfer of sediment from upstream (Zone 1) to downstream (Zone 3). In channel stabilization planning we must not neglect the potential effects that may occur throughout the system.

The fundamental concept that a stream is a portion of a large and complex system may have been most eloquently stated by Dr. Hans Albert Einstein:

> ***If we change a river we usually do some good somewhere and "good" in quotation marks. That means we achieve some kind of a result that we are aiming at but sometimes forget that the same change which we are introducing may have widespread influences somewhere else. I think if, out of today's emphasis of the environment, anything results for us it is that it emphasizes the fact that we must look at a river or a drainage basin or whatever we are talking about as a big unit with many facets. We should not concentrate only on a little piece of that river unless we have some good reason to decide that we can do that.***

#### 2.1.1.2 The System is Dynamic

In each of the idealized zones described above, a primary function is listed. Zone 1 is the sediment source that implies that erosion of sediment occurs. Zone 2 is the transfer zone that implies that as rainfall increases soil erosion from the watershed, some change must result in the stream to enable transfer of the increased sediment supply. Zone 3 is the zone of deposition and change must occur as sediment builds in this zone, perhaps the emergence of wetland habitat in a lake then a change to a floodplain as a drier habitat evolves. The function of each zone implies that change is occurring in the system, and that the system is dynamic.

From an engineering viewpoint some of these changes may be very significant. For example, loss of 100 feet of stream bank may endanger a home or take valuable agricultural land. From a geomorphic viewpoint, these changes are expected in a dynamic system and change does not necessarily represent a departure from a natural equilibrium system. In planning stabilization measures, we must realize that we are forced to work in a dynamic system and we must be try to avoid disrupting the system while we are accomplishing our task.

#### 2.1.1.3 Complexity

Landscape changes are usually complex (Schumm and Parker, 1973). We are working in a system and we have defined a system as an arrangement of things to form a whole. Change to one portion of the system may result in complex changes throughout the system.

When the fluvial system is subjected to an external influence such as channelization of part of a stream, we can expect change to occur throughout the system. Channelization usually increases stream velocity and this would allow the stream to transfer more sediment, resulting in erosion upstream and deposition downstream of the portion of the stream channelized. For example, some Yazoo Basin streams in north Mississippi that were channelized in the 1960s responded initially, but an equilibrium has not yet been reestablished as repeated waves of degradation, erosion, and aggradation have occurred.

#### 2.1.1.4 Thresholds

Geomorphic thresholds may be thought of as the straw that broke the camel's back. In the fluvial system this means that progressive change in one variable may eventually result in an abrupt change in the system. If a river erodes a few grains of soil from the toe of the river bank, no particular response will be noticed. If that continues with no deposition to balance the loss, the bank may eventually fail abruptly and dramatically due to undermining. The amount of flow impinging along a bank may vary considerably with no apparent effect on the stabilization; however, at some critical point the bank material will begin to move and disastrous consequences can result.

In these examples the change was a gradual erosion of a few grains of soil and a variability of stream velocity, both which could be considered to be within the natural system. This type of threshold would be called an intrinsic threshold. Perhaps the threshold was exceeded due to an earthquake or caused by an ill-planned bank stabilization project. These would be called an extrinsic threshold. The planner must be aware of geomorphic thresholds, and the effect that their project may have in causing the system to exceed the threshold.

Channel systems have a measure of elasticity that enables change to be absorbed by a shift in equilibrium. The amount of change a system can absorb before that natural equilibrium is disturbed depends on the sensitivity of the system, and if the system is near a threshold condition, a minor change may result in a dramatic response.

#### 2.1.1.5 Time

We all have been exposed to the geologists view of time. The Paleozoic Era ended only 248 million years ago, the Mesozoic Era ended only 65 million years ago, and so on. Fortunately, we do not have to concern ourselves with that terminology. An aquatic biologist may be concerned with the duration of an insect life stage, only a few hours or days. What we should be aware of is that the geologist temporal perspective is much broader than the temporal perspective of the engineer, and the biologist perspective may be a narrowly focused time scale. Neither profession is good nor bad because of the temporal perspective; just remember the background of people or the literature with which you are working.

Geomorphologists usually refer to three time scales in working with rivers: 1) geologic time, 2) modern time, and 3) present time. Geologic time is usually expressed in thousands or millions of years and in this time scale only major geologic activity would be significant. Formation of mountain ranges, changes in sea level, and climate change would be significant in this time scale. The modern time scale describes a period of tens of years to several hundred years, and has been called the graded time scale (Schumm and Lichty, 1965). During this period a river may adjust to a balanced condition, adjusting to watershed water and sediment discharge. The present time is considered a shorter period, perhaps one year to ten years. No fixed rules govern these definitions. Design of a major project may require less than ten years, and numerous minor projects are designed and built within the limitations of present time. Project life often extends into graded time. From a geologists temporal point of view, engineers built major projects in an instant of time, and expect the projects to last for a significant period.

In river related projects time is the enemy, time is our friend, and time is our teacher. We must learn all we can by adopting a historical perspective for each project that we undertake.

#### 2.1.1.6 Scale

The physical size of the stream may impose limits on the type of planned enhancements to the stream. For example, many variations of anchoring trees along the bank have been successfully used along small and moderate size streams to provide cover and to decrease erosion of the bank. Anchoring of trees along the bank is a reasonable method of stabilization. However, for large rivers that may have bank heights of 30 feet and a yearly water surface elevation fluctuation of 20 to 30 feet, the anchored tree may be an unreasonable method for stabilization. Applications designed for a small stream may not be directly transferrable to larger streams. If we are to transfer techniques for enhancement from stream to stream; we must also understand the design principles of those techniques. Principles, such as increasing the cover and decreasing the water velocity at the water-bank interface are transferable; however, the direct technique may not be transferable.

### 2.1.2 LANDFORMS

Now it is time to give you a brief introduction into what you may see when you go to the field. The following discussion will be confined primarily to depositional landforms along meandering rivers, and a little information concerning terraces.

A ***floodplain*** is the alluvial surface adjacent to a channel that is frequently inundated (Figure 2.2). This is a simple definition of a floodplain; however, the concept that the bankfull discharge is the sole discriminator between channel-forming and floodplain-building process is especially difficult. Although much of the literature until the 1970s suggested that the mean annual flood was the bankfull discharge, Williams (1978) clearly showed that out of thirty-five floodplains he studied in the U.S., the bankfull discharge

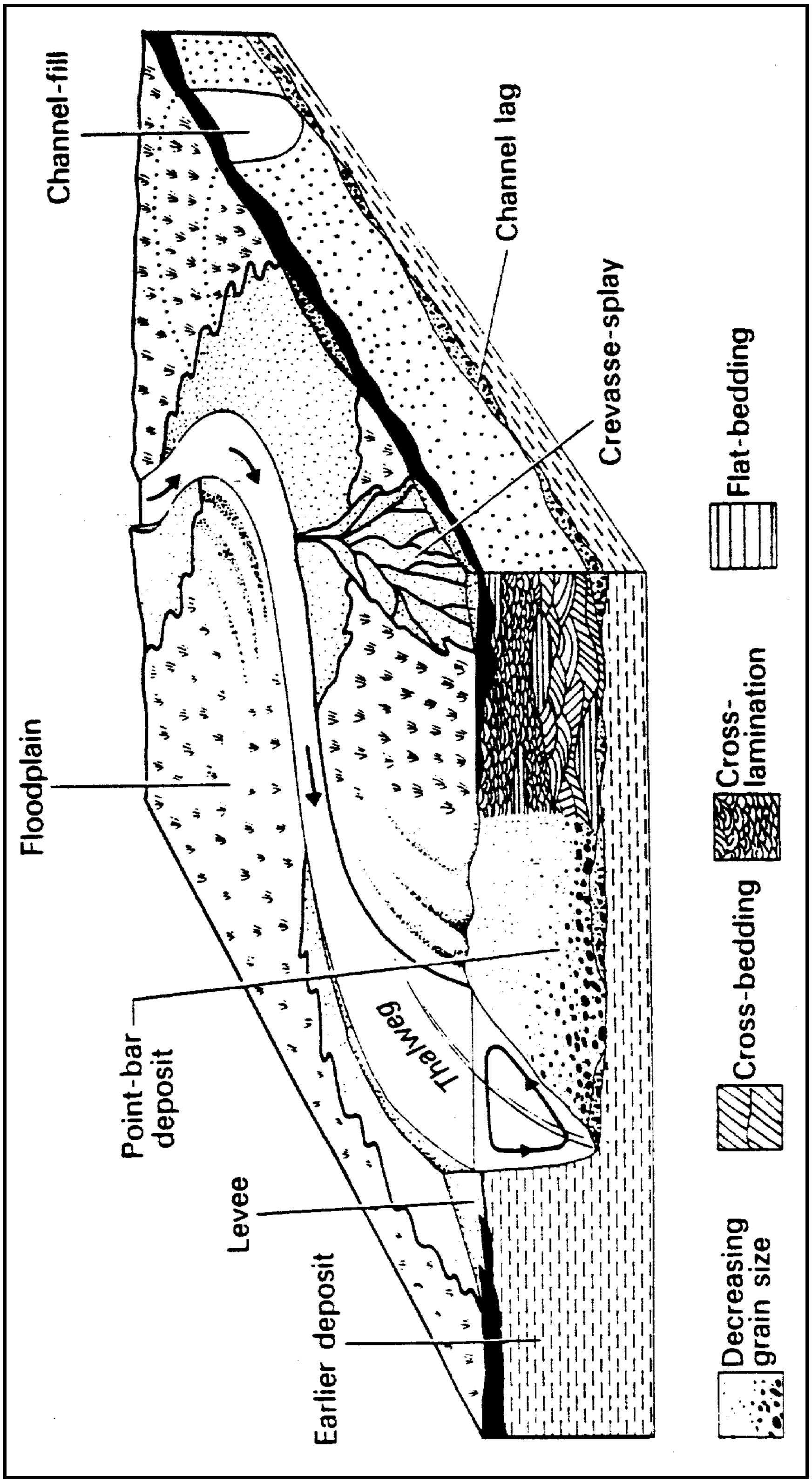



Figure 2.2 Landforms for a Meandering River (Collinson, 1978 after Allen, 1970)

varied between the 1.01- and 32-year recurrence interval. Only about a third of those streams had a bankfull discharge between the 1- and 5-year recurrence interval discharge. Knowledge of alluvial landforms will allow a more informed determination of bankfull than depending solely on the magnitude of the flood.

Table 2.1 and Figure 2.2 together provide a quick summary of some alluvial landforms found along a meandering stream. From the perspective of a stream stabilization planner, it is extremely important to know that all the materials along the bank and in the floodplain are not the same. The materials are deposited under different flow conditions. For example, ***backswamps*** and ***channel fills*** will usually be fine-grained and may be very cohesive. This is because both landforms are deposited away from the main flow in the channel, in a lower energy environment. ***Natural-levee*** deposits are coarser near the channel and become finer away from the channel as the energy to transport the larger particles dissipates.

Table 2.1 Classification of Valley Sediments

| **Place of Deposition (1)** | **Name (2)** | **Characteristics (3)** |
|---|---|---|
| Channel | Transitory channel deposits | Primarily bedload temporarily at rest; for example, alternate bar deposits. |
| | Lag deposits | Segregation of larger of heavier particles, more persistent than transitory channel deposits, and including heavy mineral placers. |
| | Channel fills | Accumulations in abandoned or aggrading channel segments, ranging from relatively coarse bedload to plugs of clay and organic muds filling abandoned meanders. |
| Channel margin | Lateral accretion deposits | Point and marginal bars which may be preserved by channel shifting and added to overbank floodplain by vertical accretion deposits at top; point-bar sands and silts are commonly trough cross-bedded and usually form the thickest members of the active channel sequence. |
| Overbank flood plain | Vertical accretion deposits | Fine-grained sediment deposited from suspended load of overbank floodwater, including natural levee and backswamp deposits; levee deposits are usually horizontally bedded and rippled fine sand, grading laterally and vertically into point-bar deposits. Backswamp deposits are mainly silts, clays and peats. |
| | Splays | Local accumulations of bedload materials, spread from channels on to adjacent floodplains; splays are cross-bedded sands spreading across the inner floodplain from crevasse breaches. |
| Valley margin | Colluvium | Deposits derived chiefly from unconcentrated slope wash and soil creep on adjacent valley sides. |
| | Mass movement deposits | Earthflow, debris avalanche and landslide deposits commonly intermix with marginal colluvium; mudflows usually follow channels but also spill overbank. |

***Point bars*** represent a sequence of deposition in which the coarser materials are at the bottom and the finer materials at the top. From the viewpoint of the channel stabilization planner, the more erosion resistant materials may then be silts and clays deposited at the top and very erosive sand may comprise the toe of the slope. Therefore, if the channel you are attempting to stabilize is eroding into an old point bar deposit, you may encounter several problems. Along the same line of thinking, an abandoned channel fill may appear on the eroding bank as a clay plug.

Different types of bank instability can also arise depending on how the materials were deposited. Consider a point bar deposit with a sandy base that has been deposited over a backswamp clay deposit. This can result in sub-surface flow at the sand-clay interface that can cause the granular material to be washed out of the bank and failure to occur some distance back from the channel. Stabilization could include proper drainage of the top of the bank to deprive the failure mechanism of the percolating groundwater source.

In addition to the landforms briefly described in Table 2.1, we should introduce ***terraces***. Terraces are abandoned floodplains formed when the river flowed at a higher level than now (Ritter, 1978). Terraces are produced by incision of the floodplain (Schumm, 1977). In other words, the stream channel has down cut leaving the previous floodplain, and is establishing a new, lower floodplain. The appearance of a terrace or a series of terraces in a surveyed cross-section may be as broad stair steps down to the stream. The steps may be broad and continuous throughout the length of the stream segment, or may be discontinuous and could be only a few feet in width.

### 2.1.3 RIVER MECHANICS

River mechanics is the subset of both fluvial geomorphology and open channel hydraulics which focuses on the form and structure of rivers. Specifically it address the channel pattern, channel geometry (cross section shape), planform geometry, and the channel slope. The purpose of this section is to introduce you to some of the basic characteristics of rivers, and help define some of the confusing terminology you may encounter when dealing with rivers.

### 2.1.4 RIVER CHARACTERISTICS AND BASIC DEFINITIONS

Rivers and streams are dynamic and continuously change their position, shape, and other morphological characteristics with variations in discharge and with the passage of time. It is important not only to study the existing river but also the possible variations during the lifetime of the project, particularly in terms of effective treatment of bank erosion. The characteristics of the river are determined by the water discharge, the quantity and character of sediment discharge, the composition of the bed and bank material of the channel, geologic controls, the variations of these parameters in time, and man's activities. To predict the behavior of a river in a natural state or as affected by man's activities, we must understand the characteristics of the river as well as the mechanics of formation.

### 2.1.4.1 Channel Pattern

Channel pattern describes the **planform** of a channel. The primary types of planform are meandering, braided, and straight. In many cases, a stream will change pattern within its length. The type pattern is dependent on slope, discharge, and sediment load.

The most common channel pattern is the **meandering stream** (Figure 2.3). A meandering channel is one that is formed by a series of alternating changes in direction, or bends. Relatively straight reaches of alluvial rivers rarely occur in nature. However, there are instances where a reach of river will maintain a nearly straight alignment for a long period of time. Even in these relatively straight reaches, the thalweg may still meander and alternate bars may be formed. Straight streams generally occur in relatively low energy environments. The **braided pattern** is characterized by a division of the river bed into multiple channels (Figure 2.4). Most braided streams are relatively high gradient and relatively coarse streams.

### 2.1.4.2 Channel Geometry and Cross Section

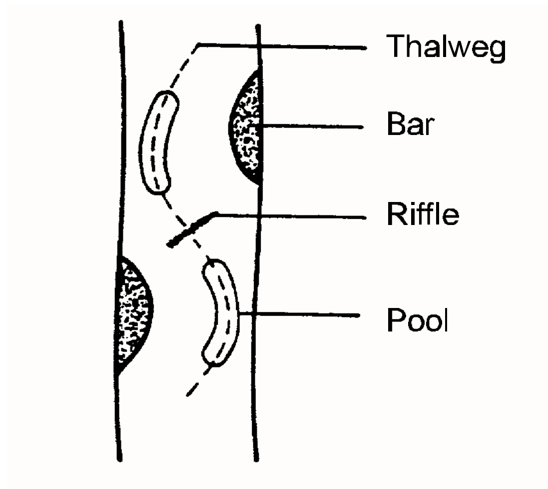
The following paragraphs describe the channel geometry and cross sectional characteristics of streams. Since meandering streams are the most common form of alluvial channels this discussion will focus primarily on this stream type.

**Pools and Crossings.** A schematic showing features associated with meanders and straight channels is given in Figure 2.5. As the **thalweg**, or trace formed by the deepest portion of the channel, changes from side to side within the channel, the momentum of the flow affects the cross-sectional geometry of the stream. In bends, there is a concentration of flow due to centrifugal forces. This causes the depth to increase at the outside of the bend, and this area is known as a **pool**. As the thalweg again changes sides below a bend, it crosses the centerline of the channel. This area is known as the **riffle or crossing**. At the point of tangency between adjacent bends, the velocity distribution is fairly consistent across the cross section, which is approximately rectangular in shape. The concentration of flow in the bend is lost and the velocity decreases accordingly, thus causing deposition in the crossing.

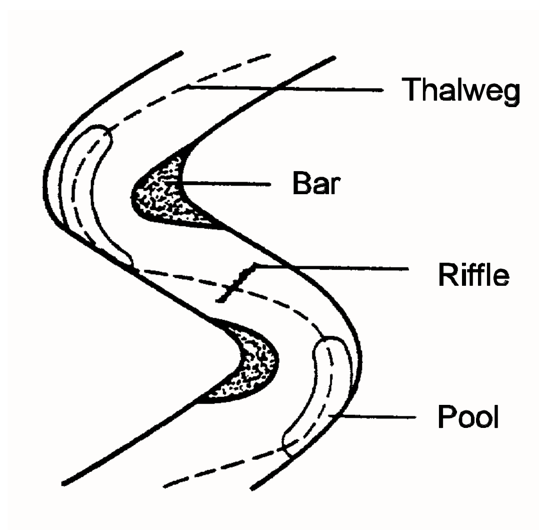
**Cross Section Shape.** The shape of a cross section in a stream depends on the point along the channel with reference to the plan geometry, the type channel, and the characteristics of the sediment forming and transported within the channel. The cross section in a bend is deeper at the **concave** (outer bank) side with a nearly vertical bank, and has a shelving bank as formed by the point bar on the **convex** side. The cross section will be more trapezoidal or rectangular in a crossing. These are shown in Figure 2.6. Cross section shape can be described by a number of variables. Some of these such as the **area**, **width**, and **maximum depth** are self explanatory. However, other commonly used parameters warrant some explanation. The **wetted perimeter (P)** refers to the length of the wetted cross section measured normal to the direction of flow. The **width-depth (w/d)** ratio is the channel width divided by the **average depth (d)** of the channel. The average depth is

Figure 2.3 Typical Meandering River

Figure 2.4 Typical Braided River




(a) Straight




(b) Meandering

Figure 2.5   Features Associated With (a) Straight and (b) Meandering Rivers

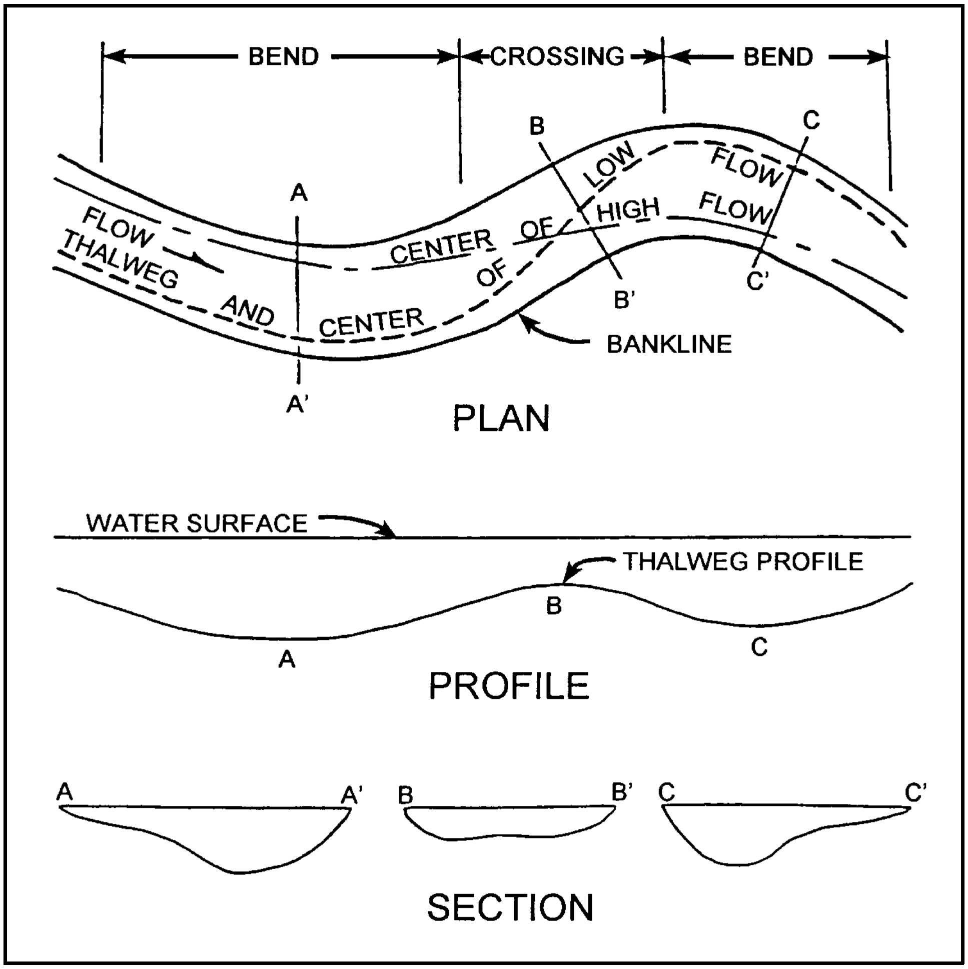



Figure 2.6 Typical Plan and Cross Sectional View of Pools and Crossings

calculated by dividing the cross section area by the channel width. The **hydraulic radius (r)**, which is important in hydraulic computations is defined as the cross sectional area divided by the wetted perimeter. In wide channels with w/d greater than about 20 the hydraulic radius and the mean depth are approximately equal. The **conveyance**, or capacity of a channel is related to the area and hydraulic radius and is defined as $AR^{2/3}$.

**Channel Bars.** Channel bars are depositional features that occur within the channel. The size and location of bars are related to the sediment transport capacity and local geometry of the reach. The enlargement of a bar generally results in caving of the opposite banks in order to maintain conveyance of the discharge. The primary types of bars are point bars, middle bars, and alternate bars.

**Point bars** form on the inside (convex) bank of bends in a meandering stream. A typical point bar is shown in Figure 2.3. The size and shape of the point bar are determined by the characteristics of the flow. The development of a point bar is partially due to the flow separation zone caused by centrifugal forces in the bend, and secondary flow. **Middle bar** is the term given to areas of deposition lying within, but not connected to the banks. Figure 2.7 shows a typical middle bar on the Mississippi River. Middle bars tend to form in reaches where the crossing areas between bends are excessively long and occasionally in bends due to the development of chutes. **Alternate bars** are depositional features that are positioned successively down the river on opposite sides (Figure 2.8). Alternate bars generally occur in straight reaches and may be the precursor to a fully developed meander pattern.

### 2.1.4.3 Planform Geometry

**Sinuosity** is a commonly used parameter to describe the degree of meander activity in a stream. Sinuosity is defined as the ratio of the distance along the channel (channel length) to the distance along the valley (valley length). Think of sinuosity as the ratio of the distance the fish swims to the distance the crow flies. A perfectly straight channel would have a sinuosity of 1.0, while a channel with a sinuosity of 3.0 or more would be characterized by tortuous meanders.

The **meander wave length (L)** is twice the straight line distance between two consecutive points of similar condition (i.e. pools or crossings) in the channel as depicted in Figure 2.9. This is sometimes referred to as the axial meander wavelength to distinguish it from the channel length between inflection points which is also sometimes referred to as the meander wave length. The **meander amplitude (A)** is the width of the meander bends measured perpendicular to the valley or straight line axis (Figure 2.9). The ratio of the amplitude to meander wavelength is generally within the range 0.5 to 1.5. It should be noted that the meander amplitude and the width of the meander belt will probably be unequal. The meander belt of a stream is formed by and includes all the locations held by a stream during its development history. In many cases, this may include all portions of the present flood plain. Meander wave length and meander width are primarily dependent on the water and sediment discharge, but may also be modified by confines of the material in which the channel is formed. The effects of bank materials is shown by the irregularities found in the alignment of natural channels. If the material forming the banks was homogeneous over long distances, a sinusoidal alignment having a unique and uniform meander wavelength would be expected although this rarely occurs in nature.

The **radius of curvature (r)** is the radius of the circle defining the curvature of an individual bend measured between adjacent inflection points (Figure 2.9). The **arc angle (è)** is the angle swept out by the radius of curvature between adjacent inflection points. The radius of curvature to width ratio (r/w) is a very useful parameter that is often used in the description and comparison of meander behavior, and in particular, bank erosion rates. The radius of curvature is dependent on the same factors as the meander wavelength and width. Meander bends generally develop a radius of curvature to width ratio (r/w) of 1.5 to 4.5, with the majority of bends falling in the 2 to 3 range.

Figure 2.7 Typical Middle Bar

Figure 2.8 Typical Alternate Bar Pattern

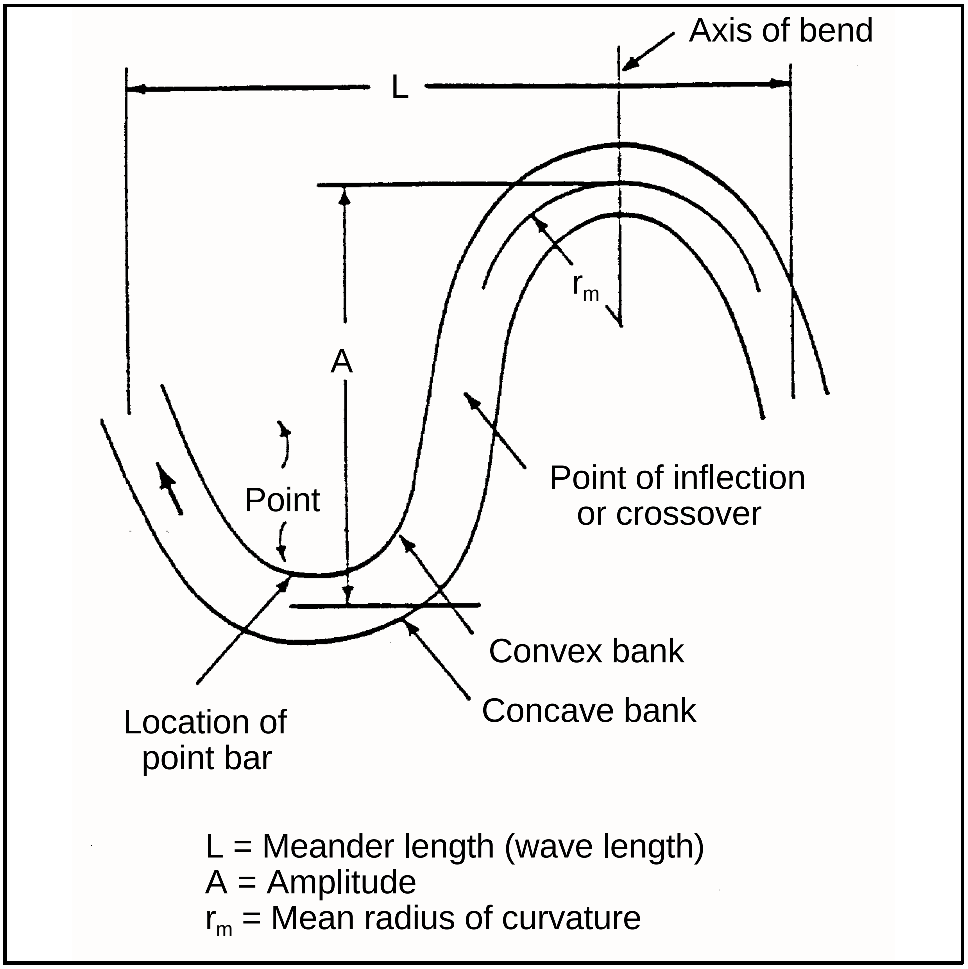



Figure 2.9 Definition Sketch for Channel Geometry (after Leopold et al., 1964)

### 2.1.4.4 Channel Slope

The slope (longitudinal profile) of a stream is one of the most significant parameters in the study and discussion of river behavior. The slope is one of the best indicators of the ability of the river to do work. Rivers with steep slopes are generally much more active with respect to bank erosion, bar building, sediment movement, etc., than lower slope channels.

Slope can be defined in a number of ways. If sufficient data exists, then the water surface slope may be calculated using stage readings at gage locations along the channel. However, in many instances, particularly in small streams, gage information is non-existent. In these cases, the thalweg slope is generally calculated. The thalweg refers to the deepest point in a cross section. The thalweg slope not only provides a good expression of the energy of the stream, but also may aid in locating areas of scour and fill, geologic controls, and outcrops of non-erodible materials.

### 2.1.5 RELATIONSHIPS IN RIVERS

One interesting aspect of meandering rivers is the similarity in the proportion of planform characteristics. Various empirical relationships have been developed which relate radius of curvature and meander wavelength to channel width and discharge. Brice (1984) suggested that these similarities regardless of size, account for the fact that the meandering planform is sensibly independent of scale. In other words, if scale is ignored all meandering rivers tend to look alike in plan view. This fact provides us with a glimmer of hope that we might be able to develop some relationships to help explain the behavior of complex river systems.

Investigation by Lane (1957) and Leopold and Wolman (1957) showed that the relationships between discharge and channel slope can define thresholds for indicating which rivers tend to be braided or meandering, as shown in Figures 2.10 and 2.11. Lane's relationship is somewhat more realistic because an intermediate range is included; however, both relationships are very similar in the variables used and the appearance of the graphs. Rivers that are near the threshold lines may exhibit segments that transitions between the two plan forms. These relationships can be useful if the planform of a river is to be changed. For instance, a meandering river positioned at point ‘A’ in Figure 2.11 might be shifted to point ‘B’ if the slope is increased due to the construction of man-made cutoffs. Shifting the channel into the transition zone would cause some concern about the possibility of the channel becoming braided.

Another set of empirical relationships is related to meander geometry. Leopold et al. (1964) reported the relationship between meander wave length (L) and channel width (w), meander amplitude (A) and channel width (w), and meander wave length (L) and bendway radius of curvature ($R_c$) as defined by Leopold and Wolman (1960). The relationships are:

$$L = 10.9\, w^{1.01}$$

$$A = 2.7\, w^{1.1}$$

$$L = 4.7\, R_c^{0.98}$$

Leopold et al. (1964) stated that the exponents for the relationships are approximately unity, and these relationships can be considered linear. Also, they pointed out that channel meander form is affected by the cohesiveness of the channel boundaries. Dury (1964) found that meander wave length is related to the mean annual flood ($Q_{ma}$):

$$L = 30\, Q_{ma}^{0.5}$$

Schumm (1960, 1977) investigated the effect of the percentage silt and clay (M) in the stream boundaries and reported the following relationship for meander wave length:

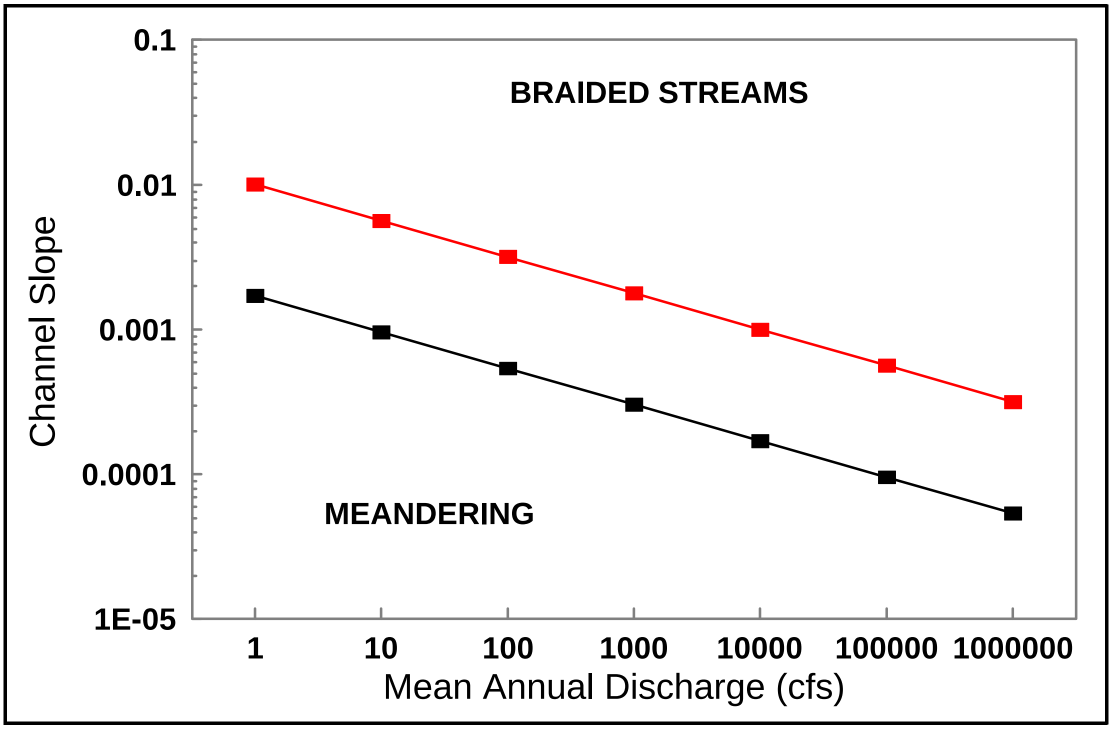



Figure 2.10 Lane's (1957) Relationship Between Channel Patterns, Channel Gradient, and Mean Discharge

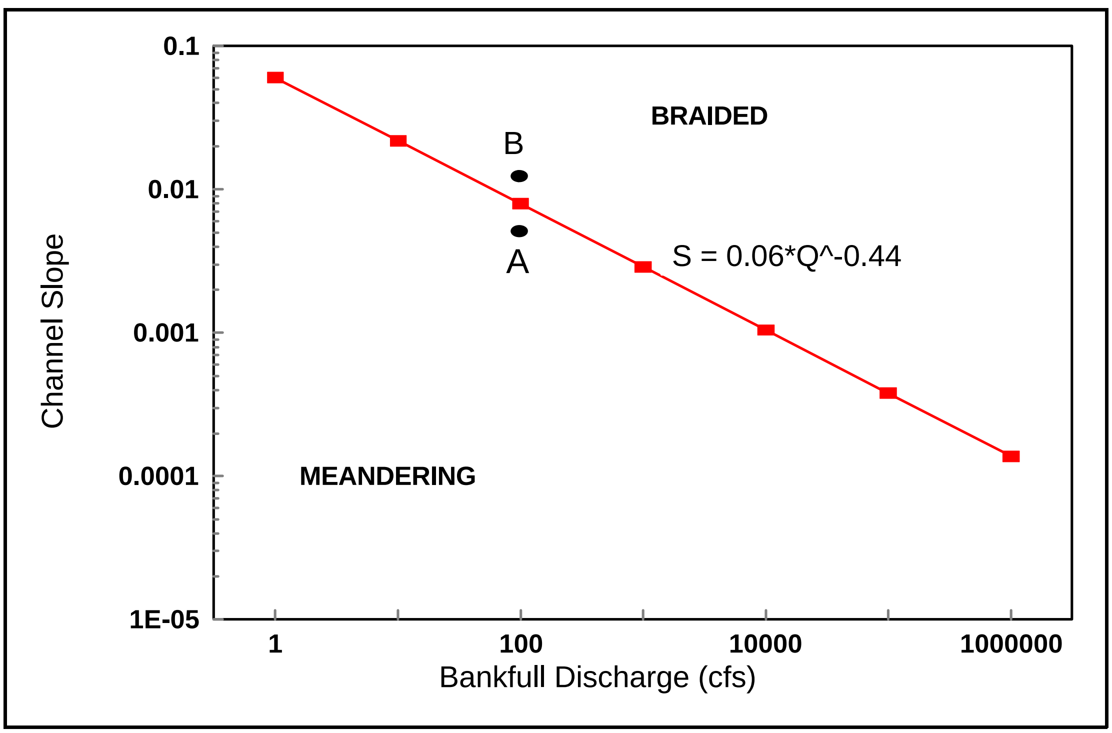



Figure 2.11 Leopold and Wolman's (1957) Relationship Between Channel Patterns, Channel Gradient, and Bankfull Discharge

$$L = 1890\, Q_m^{0.34}\, M^{-0.74}$$

where $Q_m$ is the average annual flow. The width to depth ratio (F) is also related to the percentage silt and clay:

$$F = 255\, M^{-1.08}$$

Channel slope (S) was found to be related to the mean annual discharge ($Q_m$) and percentage silt and clay:

$$S = 60\, M^{-0.38}\, Q_m^{-0.32}$$

Regime theory is an application of the idea that the width, depth, slope, and planform of a river are adjusted to a channel-forming discharge. In his review of the history of regime theory, Lane (1955) states that in 1895 Kennedy proposed the following relationship:

$$V = cD^m$$

in which V is the mean channel velocity, D is the channel depth, and c and m are constants developed for various channel locations. Much of the early work in developing regime relationships was conducted in the irrigation canals of India, and since the early 1900s, many relationships have been proposed.

Leopold and Maddock (1953) compiled a significant statistical data base using USGS gauging records and developed **hydraulic geometry** relationships for the width, depth, velocity, and other hydraulic characteristics for some streams in the United States. The hydraulic geometry relationships are of the same general form as Kennedy (1895):

$$W = a\, Q^b$$
$$D = c\, Q^f$$
$$V = k\, Q^m$$

in which W is channel width, Q is discharge, D is depth, and V is velocity.

All of the relationships presented, including the hydraulic geometry relationships, are strictly empirical, i.e., the relationships describe observed physical correlations. As conditions change from watershed to watershed, the relationships must be modified. For example, stream width for sandy banks would be expected to be different from clay banks. Schumm's relationship between width to depth ratio (F) and the weighted percent silt-clay in the channel perimeter (M) is an empirical relationship that describes this observation. If Schumm's relationship is correct, then is the hydraulic geometry relationship valid that predicts width (W) based only as a function of discharge? Both relationships can be valid for the data set used in developing the relationship.

An example of the improper use of empirical relationships was provided by Mark Twain in *Life on the Mississippi* (Clemens, 1944). In his wonderfully sarcastic manner, he describes Mississippi River cutoffs of which he had knowledge. Therefore, he developed an empirical relationship to predict the eventual length of the Mississippi River. He eloquently describes the modeling process:

> ***"Now, if I wanted to be one of those ponderous scientific people, and "let on" to prove what had occurred in the remote past by what had occurred in a given time in the recent past, or what will occur in the far future by what has occurred in late years, what an opportunity is here! Geology never had such a chance, nor such exact data to argue from! Nor "development of species," either! Glacial epochs are great things, but they are vague - vague. Please observe:***
>
> ***"In the space of 176 years, the Lower Mississippi has shortened itself 242 miles. That is an average of a trifle over one mile and a third per year. Therefore, any calm person, who is not blind or idiotic, can see that in the Old Oölitic Silurian Period, just a million years ago next November, the Lower Mississippi River was upwards of 1,300,000 miles long, and stuck out over the Gulf of Mexico like a fishing rod. And by the same token, any person can see that 742 years from now the Lower Mississippi will be only a mile and three-quarters long, and Cairo and New Orleans will have joined their streets together, and be plodding comfortably along under a single mayor and a mutual board of aldermen. There is something fascinating about science. One gets such wholesale returns of conjecture out of such a trifling investment of fact."***

The primary point of this delightful sarcasm is that we should not fall into the trap of attempting to plan a project based on "...wholesale returns of conjecture out of such a trifling investment of fact." Empirical relationships can be very useful. We cannot be certain that New Orleans and St. Louis will have a common Board of Aldermen on September 13, 2604; however we must be certain that the data from which the relationship was developed is valid for the project location, for the scale of the project, and that the relationship makes physical sense in application to the project.

### 2.1.6 CHANNEL CLASSIFICATION

Several primary methods of river classification are presented in the following paragraphs, and these methods can be related to fundamental variables and processes controlling rivers. One important classification is either alluvial or non-alluvial. An **alluvial** channel is free to adjust dimensions such as size, shape, pattern, and slope in response to change and flow through the channel. The bed and banks of an alluvial river are composed of material transported by the river under present flow conditions. Obviously, a **non-alluvial river** is not free to adjust. An example of a non-alluvial river is a bedrock controlled channel.

In other conditions, such as in high mountain stream flowing in very coarse glacially deposited materials or significantly controlled by fallen timber would suggest a non-alluvial system.

Alluvial channels may also be classified as either perennial, intermittent, or ephemeral. A **perennial stream** is one which has flow at all times. An **intermittent stream** has the potential for continued flow, but at times the entire flow is absorbed by the bed material. This may be seasonal in nature. An **ephemeral stream** only has flow following a rainfall event. When carrying flow, intermittent and ephemeral streams both have characteristics very similar to perennial streams.

Another classification methodology by Schumm (1977) includes consideration of the type of sediment load being transported by the stream, the percentage of silt and clay in the channel bed and banks, and the stability of the channel. **Sediment load** refers to the type or size of material being transported by a stream. The total load can be divided into the **bed sediment load** and the **wash load**. The bed sediment load is composed of particles of a size found in appreciable quantities in the bed of the stream. The wash load is composed of those finer particles that are found in small quantities in the shifting portions of the bed. Frequently, the sediment load is divided into the bed load, those particles moving on or near the bed, and the **suspended load**, those particles moving in the water column. The size of particles moving as suspended load may include a portion of the bed sediment load, depending on the energy available for transport (ASCE, 1977). For example, the suspended load frequently reported by U.S. Geological Survey publications usually includes a portion of the bed sediment load and all of the wash load. **Sediment discharge** is the rate at which the sediment load is being supplied or transported through a reach.

For purposes of this classification system, a stable channel complies with Mackin's definition of a graded stream. An unstable stream may be either **degrading** (eroding) or **aggrading** (depositing). In the context of the definition of a graded stream being in balance between sediment supplied and sediment transported, an aggrading stream has excess sediment supply and a degrading stream has a deficit of sediment supply.

Table 2.2 presents a summary of this classification system and describes the response of the river segment to instability and a description of the stable segment. It is very important to note that the work on which this classification was based was conducted in the Midwestern U.S.; therefore, the classification system represents an interpretation of empirical data. Extrapolation of the classification beyond the data base should be done cautiously.

Schumm and Meyer (1979) presented the channel classification shown in Figure 2.12, which is based on channel planform, sediment load, energy, and relative stability. As with any classification system, Figure 2.12 implies that river segments can be conveniently subdivided into clearly discernable groups. In reality, a continuum of channel types exists and the application of the classification system requires judgement.

Table 2.2 Classification of Alluvial Channels (after Schumm, 1977)

| **Mode of sediment transport and type of channel** | **Channel sediment (M) (percent)** | **Bedload (percentage of total load)** | **Channel stability** | | |
|---|---|---|---|---|---|
| | | | **Stable (graded stream)** | **Aggrading (excess sediment discharge)** | **Degrading (deficiency of sediment discharge)** |
| Suspended load | >20 | <3 | Stable suspended-load channel. Width/depth ratio <10; sinuosity usually >2.0; gradient, relatively gentle | Depositing suspended load channel. Major deposition on banks cause narrowing of channel; initial streambed deposition minor | Eroding suspended-load channel. Streambed erosion predominant; initial channel widening minor |
| Mixed load | 5-20 | 3-11 | Stable mixed-load channel. Width/depth ratio >10, <40; sinuosity usually <2.0, >1.3; gradient moderate | Depositing mixed-load channel. Initial major deposition on banks followed by streambed deposition | Eroding mixed-load channel. Initial streambed erosion followed by channel widening |
| Bed load | <5 | >11 | Stable bed-load channel. Width/depth ratio >40; sinuosity usually <1.3; gradient, relatively steep | Depositing bed-load channel. Streambed deposition and island formation | Eroding bed-load channel. Little streambed erosion; channel widening predominant |

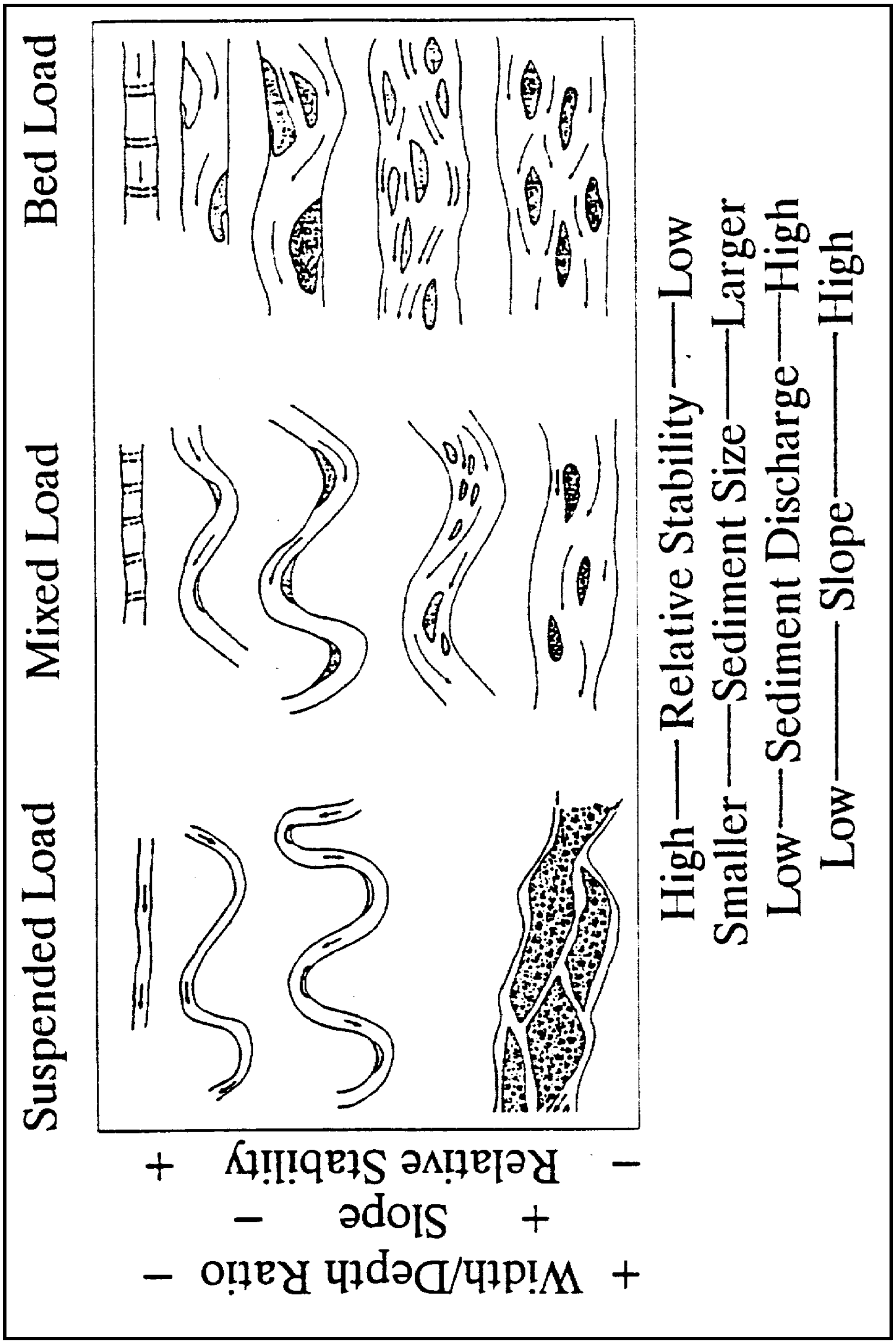



Figure 2.12 Channel Classification Based on Pattern and Type of Sediment Load (after Schumm, 1981)

Other stream classifications include those by Neill and Galay (1967) and by Rundquist (1975). These systems go well beyond a description of the channel, and include description of land use and vegetation in the basin, geology of the watershed, hydrology, channel bed and bank material, sediment concentration, channel pattern, and channel stability.

Rosgen (1994) presented a stream classification system similar to the Runquist (1975) system. A primary difference between the two systems is that planform and bed material character are combined into one code, improving the ease of use. Rosgen (1994) also included an entrenchment ratio, which is the ratio of the width of the flood-prone area to the surface width of the bankfull channel. Like Runquist (1975), Rosgen (1996) has also added valley type classification. Table 2.3 is a summary of delineative criteria for broad-level classification from Rosgen (1994). Each of the stream types can be associated with dominant bed material types as follows: Bedrock - 1, Boulder - 2, Cobble - 3, Gravel - 4, Sand - 5, and Silt/Clay - 6.

Table 2.3 Summary of Delineative Criteria for Broad-level Classification (Rosgen, 1994)

| **Stream Type** | **Entrench. Ratio** | **W/D Ratio** | **Sinuosity** | **Slope** | **Meander Belt/ Bankfull Width** | **Dominant Bed Material*** |
|---|---|---|---|---|---|---|
| **Aa+** | <1.4 | <12 | 1.0 - 1.1 | > 0.10 | 1.0 - 3.0 | 1,2,3,4,5,6 |
| **A** | <1.4 | <12 | 1.0 - 1.2 | 0.04 - 0.10 | 1.0 - 3.0 | 1,2,3,4,5,6 |
| **B** | 1.4 - 2.2 | >12 | >1.2 | 0.02 - 0.039 | 2.0 - 8.0 | 1,2,3,4,5,6 |
| **C** | >2.2 | >12 | >1.4 | < 0.02 | 4.0 - 20 | 1,2,3,4,5,6 |
| **D** | na | >40 | na | < 0.04 | 1.0 - 2.0 | 3,4,5,6 |
| **DA** | >4.0 | <40 | variable | < 0.005 | na | 4,5,6 |
| **E** | >2.2 | <12 | >1.5 | < 0.02 | 20 - 40 | 3,4,5,6 |
| **F** | <1.4 | >12 | >1.4 | < 0.02 | 2.0 - 10 | 1,2,3,4,5,6 |
| **G** | <1.4 | <12 | >1.4 | 0.02 - 0.039 | 2.0 - 8.0 | 1,2,3,4,5,6 |

***Dominant Bed Material Key**

**1 - Bedrock**
**2 - Boulder**
**3 - Cobble**
**4 - Gravel**
**5 - Sand**
**6 - Silt/Clay**

With some modifications to Figure 2.12, Figure 2.13 is a combination of some concepts of Schumm and Rosgen. Schumm's classification system was heavily dependent on his Midwestern experience, while Rosgen's experience began in steep mountain streams. In addition, Schumm's (1977) classification did not specifically include incised channels, which are included in Rosgen's (1994) F and G classes. Figure 2.13 includes C, D, DA, and E classes, and could be expanded to include all of Rosgen (1994) classes. The value of Figure 2.13 is to demonstrate that moving from class to class is a predictable response that manages energy, materials, and channel planform to reestablish a balance of sediment and water discharge with sediment and water supply.

## 2.2 CHANNEL STABILITY CONCEPTS

Streambank protection measures often fail, not as the result of inadequate structural design, but rather because of the failure of the designer to incorporate the existing and future channel morphology into the design. For this reason, it is important for the designer to have some general understanding of stream processes to insure that the selected stabilization measures will work in harmony with the existing and future river conditions. This section describes the basic concepts of channel stability. This will allow the designer to assess whether the erosion at a particular site is due to local instability processes or is the result of some system-wide instability problems that may be affecting the entire watershed.

### 2.2.1 THE STABLE CHANNEL

The concept of a stable river is one that has generated controversy between engineers, scientists, landowners, and politicians for many years. An individual's definition of stability is often subjectively based on past experiences or project objectives. To the navigation engineer, a stable river might be one that maintains adequate depths and alignment for safe navigation. The flood control engineer on the other hand is more concerned with the channel maintaining the ability to pass the design flood, while to the local landowner a stable river is one that does not erode the bankline. Therefore, bank erosion would not be an acceptable component of these groups' definition of a stable river. Geomorphologists and biologists, on the other hand, might maintain that bank erosion is simply part of the natural meandering process of stable rivers and would be perfectly acceptable in their definition of a stable river. Consequently, there is no universally accepted definition of a stable river. However, some manner of defining stability is needed before the concept of instability can be discussed. Therefore, the following paragraphs will attempt to establish a definition of a stable river to be used for this manual.

River behavior may be influenced by a number of factors. Schumm (1977) identified these as independent and dependent variables. Independent variables may be thought of as the basin inputs or constraints that cause a change in the channel morphology. Independent variables include: basin geology, hydrology (discharge of water and sediment), valley

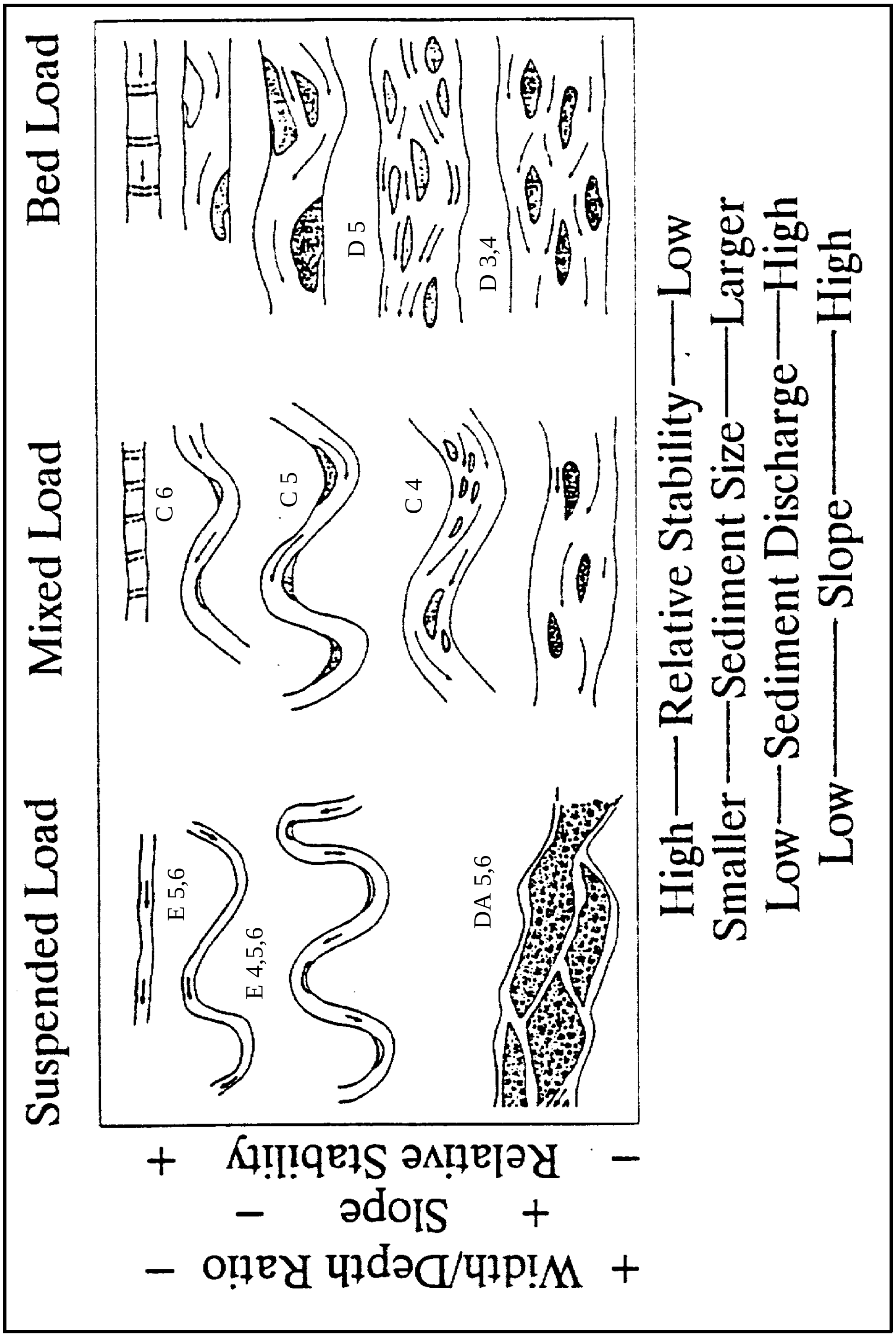



Figure 2.13 Channel Classification Combining Aspects of Schumm (1981) and Rosgen (1994)

dimensions (slope, width, depth), vegetation (type and density), and climate. Dependent variables include: channel slope, depth, width, and planform.

A channel that has adjusted its dependent variables to accommodate the basin inputs (independent variables) is said to be stable. Mackin (1948) gave the following definition of a **graded stream**:

> ***A graded stream is one in which, over a period of years, slope is delicately adjusted to provide, with available discharge and with prevailing channel characteristics, just the velocity required for the transportation of the load supplied from the drainage basin. The graded stream is a system in equilibrium.***

Mackin did not say that a stream in equilibrium is unchanging and static. A more commonly used term today for this type of stability is **dynamic equilibrium**. A stream in dynamic equilibrium has adjusted its width, depth and slope such that the channel is neither aggrading nor degrading. However, change may be occurring in the stream bank, erosion may result, and bank stabilization may be necessary, even on the banks of a stream in dynamic equilibrium.

The equilibrium concept of streams discussed above can also be described by various qualitative relationships. One of the most widely used relationships is the one proposed by Lane (1955) which states that:

$$QS \,\%\, Q_s D_{50}$$

where Q is the water discharge, S is the slope, $Q_s$ is the bed material load, and $D_{50}$ is the median size of the bed material. This relationship, commonly referred to as Lane's Balance, is illustrated in Figure 2.14. Mackin's concept of adjustment to changes in the controlling variables is easily illustrated by Lane's balance (Figure 2.14) which shows that a change in any of the four variables will cause a change in the others such that equilibrium is restored. When a channel is in equilibrium, it will have adjusted these four variables such that the sediment being transported into the reach is transported out, without significant deposition of sediment in the bed (aggradation), or excessive bed scour (degradation). It should be noted that by this definition of stability, a channel is free to migrate laterally by eroding one of its banks and accreting the one opposite at a similar rate.

Meandering can be thought of as nature's way of adjusting its energy (slope) to the variable inputs of water and sediment. Cutoffs (oxbow lakes) and abandoned courses in the floodplain attest to the dynamic behavior of rivers. Oftentimes the engineer or scientist draws the erroneous conclusion that a dis-equilibrium condition exist because natural cutoffs are occurring. However, this type of dynamic behavior is quite common in rivers that are in a state of dynamic equilibrium. In this situation, as natural cutoffs occur, the river may be

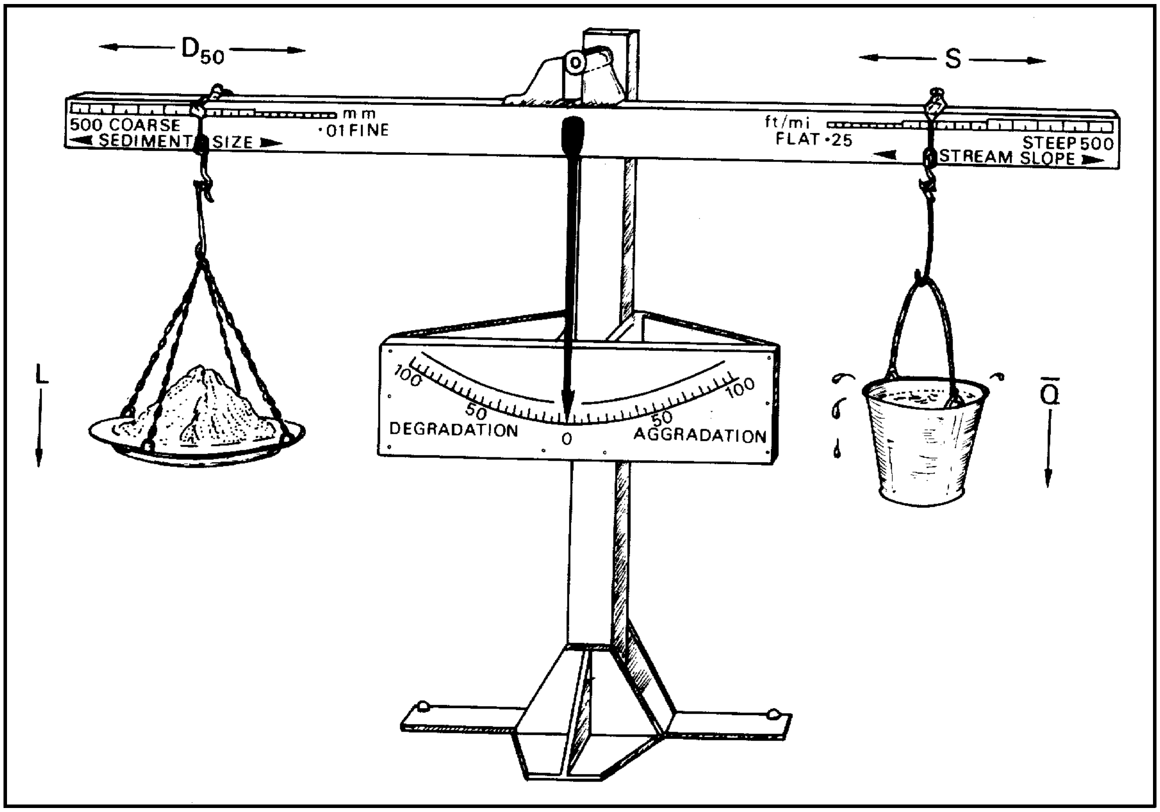



Figure 2.14 Lane's Balance (after E. W. Lane, from W. Borland)

obtaining additional length elsewhere through meandering, with the net result being that the overall reach length, and therefore slope, remains unchanged.

In summary, a stable river, from a geomorphic perspective, is one that has adjusted its width, depth, and slope such that there is no significant aggradation or degradation of the stream bed or significant plan form changes (meandering to braided, etc.) within the engineering time frame (generally less than about 50 years). By this definition, a stable river is not in a static condition, but rather is in a state of dynamic equilibrium where it is free to adjust laterally through bank erosion and bar building. This geomorphic definition of stability (dynamic equilibrium) is developed here to establish a reference point for the discussion of system and local instability in the following sections.

### 2.2.2 SYSTEM INSTABILITY

The equilibrium of a river system can be disrupted by various factors. Once this occurs the channel will attempt to re-gain equilibrium by making adjustments in the dependent variables. These adjustments are generally reflected in channel aggradation (increasing bed elevation), degradation (decreasing bed elevation), or changes in planform characteristics (meander wavelength, sinuosity, etc.). Depending upon the magnitude of the change and the basin characteristics (bed and bank materials, hydrology, geologic or man-made controls, sediments sources, etc.) these adjustments can propagate throughout the entire watershed and even into neighboring systems. For this reason, the disruption of the equilibrium condition will be referred to as system instability.

As defined above system instability is a broad term describing the dis-equilibrium condition in a watershed. System instability may be evidenced by channel aggradation, degradation, or plan form changes. This manual does not attempt to provide a complete discussion of all aspects of channel response, but rather, focuses primarily on the degradational and plan form processes because these have the most significant impact on bank stability. For a more complete discussion of channel processes, the reader is referred to Simons and Sentürk (1992), Schumm (1972), Richards (1982), Knighton (1984), and Thorne et al. (1997).

Before the specific causes are addressed a brief discussion of the consequences of system instability is necessary. The consequences of system instability can generally be discussed in terms of two components: (1) hydraulic consequences, and (2) geotechnical consequences. The consequences of system instability are illustrated in Figure 2.15. The hydraulic consequences of system instability are usually reflected in increased energy (discharge and slope) which result in excessive scour and erosion of the bed and banks. This erosion endangers bridges, buildings, roads, and other infrastructure, undermines pipeline and utility crossings, results in the loss of lands adjacent to the stream, and generates a significant amount of sediment that is ultimately deposited downstream in navigation and flood control channels. The geotechnical consequences of system instability are a direct function of the hydraulic consequences of bed lowering. As degradation proceeds through a system, the channel bank heights and angles are increased, which reduces the bank stability with respect to mass failures under gravity. If degradation continues, eventually the banks become unstable and fail. Bank failures may then no longer be localized in the bendways, but rather may also be occurring along both banks in straight reaches on a system-wide basis. When this occurs, conventional bank stabilization measures are generally not suitable, and a more comprehensive treatment plan involving grade control or flow control dams, diversion structures, etc., is required.

### 2.2.2.1 Causes of System Instability

The stability of a channel system can be affected by a number of natural or man-induced factors. Natural geologic processes obviously cause dramatic changes but these changes generally occur over thousands or perhaps millions of years and, therefore, are not often a direct concern to the individual trying to stabilize a streambank. However, channel systems are significantly impacted within the engineering time span by the natural forces of earthquakes or volcanic eruptions. Although these phenomenon may have catastrophic

(a) Bed and Bank Instability

(b) Formation of Gullies in Floodplain

Figure 2.15 Consequences of System Instability

(c) Damage to Infrastructure

(d) Excessive Sediment Deposition in Lower Reaches of Watershed

Figure 2.15 (cont.)  Consequences of System Instability consequences and receive considerable media attention, the most commonly encountered system instability problems can generally be attributed, at least in part, to man's activities.

Any time one or more of the controlling variables (runoff, sediment loads, sediment size, channel slope, etc.) in a watershed are altered there is a potential for inducing system instability. The particular system response will reflect the magnitude of change and the existing morphological sensitivity of the system. Therefore, each system is unique and there is no standard response that applies to all situations. With this in mind it is not practical to attempt to discuss all the possible scenarios of channel response. Rather, the aim of this discussion is to present some of the more common factors causing system instability, and to illustrate how a particular channel response might be anticipated using the stability concepts discussed earlier.

A list and brief discussion of some of the more common causes of system instability are presented in the following sections. For this discussion the causes have been grouped into three categories: (1) downstream factors, (2) upstream factors, and (3) basin-wide factors. Following this, a brief discussion is presented concerning complex response and the complications involved when a system is subjected to multiple factors.

**Downstream Factors.** The stability of a channel system can be significantly affected by a downstream **base level** lowering. Base level refers to the downstream controlling water surface or bed elevation for a stream. One of the most common causes of base level lowering is the implementation of cutoffs or channelization as part of channel improvement projects (Figure 2.16). As indicated by Lane's relation (Figure 2.14) the increased slope must be offset by one of the other variables. Consequently, there is an imbalance between the sediment transport capacity and supply. If the discharge and bed material are assumed to remain constant (which may not always be the case), then the channel must adjust to the increased slope (i.e., sediment transport capacity) by increasing its bed material load. This increased sediment load will be derived from the bed and banks of the channel in the form of channel degradation and bank erosion. As the bed continues to degrade the zone of increased slope will migrate upstream and the increased bed material load is transmitted downstream to drive aggradational instability there.

The manner in which degradation migrates through a channel system is a very complex process. Before this process is discussed some of the relevant terminology must first be addressed. The following definition of terms is based on the terminology used by Schumm et al. (1984). Channel degradation simply refers to the lowering of the channel bed. Field indicators of degradation occur in the form of knickpoints or knickzones. A **knickpoint** is a location on the thalweg of an abrupt change of elevation and slope (Figure 2.17). This may also be visualized as a waterfall or vertical discontinuity in the stream bed. A steep reach of channel representing the headward migrating zone is referred to as a **knickzone** (Figure 2.18). A knickzone is often composed of a series of small knickpoints. Knickpoints and knickzones are often referred to as **headcuts**. While headcut is a commonly used term, it does generate some confusion because it is also used as a description of the headward migration process of degradation.

Figure 2.16 Channelized Stream and Abandoned Old Channel

Figure 2.17  Knickpoint in a Degrading Channel

Figure 2.18 Knickzone in a Degrading Channel

To avoid this confusion the field indicators of degradation (knickpoints and knickzones) will not be referred to as headcuts. Rather, a headcut (or headcutting) is defined as a headward migrating zone of degradation. This headcutting may occur with or without the formation of knickpoints or knickzones which are purely a function of the materials encountered.

Once headcutting is initiated it may proceed rapidly through the system. The rate of headward advance is a direct function of the materials encountered in the bed and also the basin hydrology. If the channel bed is composed primarily of non-cohesive sands and silts, then no knickpoints or knickzones will form and headcutting will work upstream by parallel lowering of the bed. However, if consolidated materials such as clays, sandstones, or other resistant materials occur in the channel bed, then knickpoints or knickzones will form as degradation encounters these resistant layers. When this occurs the headward migration rate may slow considerably. Therefore, the dominant factor affecting the headward migration rate is the relative resistance to erosion of the bed materials, and to a lesser degree the discharge in the stream.

As degradation migrates upstream it is not restricted to the main stem channel. When headcutting passes tributary junctions it lowers the base level of these streams. This initiates the degradation process for the tributaries. The localized increased slope at the confluence produces an excess sediment transport capacity that results in degradation of the stream bed. This process can continue upstream rejuvenating other tributaries until the entire basin has been affected by the downstream base level lowering.

**Upstream Factors.** System instability is often initiated by upstream alterations in the basin. This may result from a change in any of the controlling variables, but is most commonly associated with modifications to the incoming discharges of water and sediment. Looking at Lane's balance (Figure 2.14) it can be seen that either an increase in the water discharge or a decrease in the sediment load can initiate channel degradation. These factors are often altered by dams or channel diversions. A brief discussion of the effects of these features on the channel stability follows.

Channel response to flow regulation may vary considerably depending upon the purpose and manner of operation of the dam. Construction of a dam has a direct impact on the downstream flow and sediment regime. Channel adjustments to the altered flow duration and sediment loads include changes in the bed material (armoring), bed elevation, channel width, plan form, and vegetation. Lane's balance (Figure 2.14) indicates that a reduction in the discharge and sediment load, as might be expected downstream of a dam, tends to produce counter-acting results. Consequently, the response of a channel system to dam construction is extremely complex. The specific channel response will depend upon the magnitude of changes in the flow duration and sediment loads and the existing channel regime downstream of the dam. Therefore, channel response downstream of a dam is very complex and may vary from stream to stream. Generally, the initial response downstream of a dam is degradation of the channel bed close to the dam and sedimentation further downstream due to increased supply from the degrading reach. This is the typical response most commonly anticipated downstream of a dam. Degradation may migrate downstream with time, but generally it is most significant during the first few years following closure of the dam. In some situations, a channel may shift from a degradational to an aggradational phase in response to slope flattening due to degradation, increased sediment inputs from tributaries and bed and bank erosion, and reduction in the dominant discharge.

System instability can also be introduced by the diversion of water into or out of the stream. Channel diversion structures are designed to divert a portion of the water and/or sediment from a stream and deliver it to another location. Diversions are often needed for water supply, irrigation, hydropower, flood control, or environmental reasons. The system effects and complexities are similar to those downstream of major dams. According to Lane's balance the sediment load in the receiving stream will be increased due to extra, transport capacity of the increased discharge. In time, the erosion of bed sediments decreases as the slope is reduced through bed degradation.

An increase in discharge due to a flow diversion can have a significant impact on the channel plan form as well as the vertical stability. Schumm (1977) proposed a qualitative relation similar to Lane's that included meander wavelength. His relation states that:

$$Q \propto \frac{b\ d\ L}{S}$$

where Q is the discharge, b is the width, d is the depth, S is the slope, and L is the meander wavelength. The above relation indicates that an increase in discharge may result in an increase in the meander wavelength which would be accomplished through accelerated erosion of the streambanks. Therefore, whenever diversions such as this are proposed the potential for increased meander activity must be considered. If a stream is in the process of increasing meander wavelength, then stabilization of the bends along the existing alignment is likely to be unsuccessful and is not recommended.

**Basin Wide Factors.** Sometimes the changes in the controlling variables can not be attributed to a specific upstream or downstream factor, but rather are occurring on a basin-wide basis. This often results from a major land use change or urbanization. These changes can significantly modify the incoming discharge and sediment loads to a channel system. For example, urbanization can increase peak flows and reduce sediment delivery, both of which would tend to cause channel degradation in the channel system. A land use change from forest to row crop on the other hand might cause a significant increase in the sediment loading resulting in aggradation of the channel system. Unfortunately, it is difficult, if not impossible, to predict when basin wide changes such as these will occur. Therefore, the best the designer can do in most cases is to simply try to design the bank protection measures to accommodate the most likely future changes in the watershed. For instance, if there is a possibility of future urbanization in the upper watershed, then additional launching stone may be needed to protect the bank from the destabilizing impact of any future bed lowering.

### 2.2.2.2 Complexities and Multiple Factors

Lane's balance and other geomorphic analyses of initial morphological response to system disturbance provide a simple qualitative method for predicting the channel response to an altered condition. However, it does not take into account the magnitude of the change and the existing morphologic condition of the stream. For instance according to Lane's balance a channel cutoff should induce degradation. While this is often the case, there are many examples where there may be no observable change in the channel morphology following the construction of cutoffs. Brice (1981) documented the stability of streams at 103 sites in different regions of the United States where channels had been relocated. He found that following the cutoffs 52% of the channels showed no change, 32% showed improvement, and 16% exhibited channel degradation. This study indicates that predicting the channel response to cutoffs is not nearly as simple as might be inferred from Lane's balance. Therefore, the designer should always be aware of the considerable uncertainties that exist when attempting to predict, even in qualitative terms, the behavior of river systems.

Previous discussions have focused primarily on the initial response of a channel to various alterations in the watershed. However, it must be remembered that the entire watershed is connected and that changes in one location can, and often do, affect the channel stability at other locations, which in turn provides a feedback mechanism whereby the original channel response may be altered. For example, the initial response to a base level lowering due to channelization may be channel degradation. However, as this degradation migrates upstream the sediment supply to the downstream reach may be significantly increased due to the upstream bed and bank erosion. This increased sediment load coupled with the slope flattening due to the past degradation may convert the channel from a degradational to an aggradational phase. Multiple response to a single alteration has been referred to as **complex response** by Schumm (1977).

Another complicating factor in assessing the cause and effect of system instability is that very rarely is the instability a result of a single factor. In a watershed where numerous alterations (dams, levees, channelization, land use changes, etc.) have occurred, the channel morphology will reflect the integration of all these factors. Unfortunately, it is extremely difficult and often impossible to sort out the precise contributions of each of these components to the system instability. The interaction of these individual factors coupled with the potential for complex response makes assessing the channel stability and recommending channel improvement features, such as bank protection, extremely difficult. There are numerous qualitative and quantitative procedures that are available. Regardless of the procedure used, the designer should always recognize the limitations of the procedure, and the inherent uncertainties with respect to predicting the behavior of complex river systems.

### 2.2.3 LOCAL INSTABILITY

For this discussion local instability refers to bank erosion that is not symptomatic of a dis-equilibrium condition in the watershed (i.e., system instability) but results from site-specific factors and processes. Perhaps the most common form of local instability is bank erosion along the concave bank in a meander bend which is occurring as part of the natural meander process. Local instability does not imply that bank erosion in a channel system is occurring at only one location or that the consequences of this erosion are minimal. As discussed earlier, erosion can occur along the banks of a river in dynamic equilibrium. In these instances the local erosion problems are amenable to local protection works such as bank stabilization measures. However, local instability can also exist in channels where severe system instability exists. In these situations the local erosion problems will probably be accelerated due to the system instability, and a more comprehensive treatment plan will be necessary.

#### 2.2.3.1 Overview of Meander Bend Erosion

Depending upon the academic training of the individual, streambank erosion may be considered as either a hydraulic or a geotechnical process. However, in most instances the bank retreat is the result of the combination of both hydraulic and geotechnical processes. The material may be removed grain by grain if the banks are non-cohesive (sands and gravels), or in aggregates (large clumps) if the banks are composed of more cohesive material (silts and clays). This erosion of the bed and bank material increases the height and angle of the streambank which increases the susceptibility of the banks to mass failure under gravity. Once mass failure occurs, the bank material will come to rest along the bank toe. The failed bank material may be in the form of a completely disaggregated slough deposit or as an almost intact block, depending upon the type of bank material, the degree of root binding, and the type of failure (Thorne, 1982). If the failed material is not removed by subsequent flows, then it may increase the stability of the bank by forming a buttress at the bank toe. This may be thought of as a natural form of toe protection, particularly if vegetation becomes established. However, if this material is removed by the flow, then the stability of the banks will be again reduced and the failure process may be repeated.

As noted above, erosion in meander bends is probably the most common process responsible for local bank retreat and, consequently, is the most frequent reason for initiating a bank stabilization program. A key element in stabilization of an eroding meander bend is an understanding of the location and severity of erosion in the bend, both of which will vary with stage and plan form geometry.

As streamflow moves through a bend, the velocity (and tractive force) along the outer bank increases. In some cases, the tractive force may be twice that in a straight reach just upstream or downstream of the bend. Consequently, erosion in bends is generally much greater than in straighter reaches. The tractive force is also greater in tight bends than in longer radius bends. This was confirmed by Nanson and Hickin (1986) who studied the migration rates in a variety of streams, and found that the erosion rate of meanders increases as the radius of curvature to width ratio (r/w) decreased below a value of about 6, and reached a maximum in the r/w range of 2 to 3. Biedenharn et al. (1989) studied the effects of r/w and bank material on the erosion rates of 160 bends along the Red River in Louisiana and also found that the maximum erosion rates were observed in the r/w range of 2 to 3. However, the considerable scatter in their data indicate that other factors, particularly bank material composition, were also modifying the meander process.

The severity and location of bank erosion also changes with stage. At low flows, the main thread of current tends to follow the concave bank alignment. However, as flow increases, the flow tends to cut across the convex bar to be concentrated against the concave bank below the apex of the bend. Friedkin (1945) documented this process in a series of laboratory tests on meandering in alluvial rivers. Because of this process, meanders tend to move in the downvalley direction, and the zone of maximum erosion is usually in the downstream portion of the bend due to the flow impingement at the higher flows. This explains why the protection of the downstream portion of the bend is so important in any bank stabilization scheme. The material eroded from the outer bank is transported downstream and is generally deposited in the next crossing or point bar. This process also results in the deposition of sediment along the upper portion of the concave bank. This depositional feature is often a good indicator of the upstream location to start a bank protection measure.

### 2.2.3.2 Streambank Erosion and Failure Processes

The terms streambank erosion and streambank failure are often used to describe the removal of bank material. **Erosion** generally refers to the hydraulic process where individual soil particles at the bank's surface are carried away by the **tractive force** of the flowing water. The tractive force increases as the water velocity and depth of flow increase. Therefore, the erosive forces are generally greater at higher flows. **Streambank failure** differs from erosion in that a relatively large section of bank fails and slides into the channel. Streambank failure is often considered to be a geotechnical process. A detailed discussion of the erosion and failure processes discussed below is provided by Thorne (1993).

Identifying the processes responsible for bank erosion is not an easy task and often requires some training. The primary erosion processes are parallel flow, impinging flow, piping, freeze/thaw, sheet erosion, rilling/gullying, wind waves, and vessel forces. These erosional forces are illustrated in Figures 2.19 through 2.25 and discussed below.

**Parallel flow** erosion is the detachment and removal of intact grains or aggregates of grains from the bank face by flow along the bank. Evidence includes: observation of high flow velocities close to the bank; near-bank scouring of the bed; under-cutting of the

Figure 2.19  Erosion Generated by Parallel Flow

Figure 2.20  Erosion Generated by Impinging Flow

Figure 2.21 Erosion Generated by Piping

Figure 2.22 Erosion Generated by Freeze/Thaw

Figure 2.23  Sheet Erosion with Rilling and Gullying

Figure 2.24  Erosion Generated by Wind Waves

Figure 2.25  Erosion Generated by Vessel Forces

toe/lower bank relative to the bank top; a fresh, ragged appearance to the bank face; absence of surficial bank vegetation.

**Impinging flow** erosion is detachment and removal of grains or aggregates of grains by flow attacking the bank at a steep angle to the long-stream direction. Impinging flow occurs in braided channels where braid-bars direct the flow strongly against the bank, in tight meander bends where the radius of curvature of the outer bank is less than that of the channel centerline, and at other locations where an in-stream obstruction deflects and disrupts the orderly flow of water. Evidence includes: observation of high flow velocities approaching the bank at an acute angle to the bank; braid or other bars directing the flow towards the bank; tight meander bends; strong eddying adjacent to the bank; near-bank scouring of the bed; under-cutting of the toe/lower bank relative to the bank top; a fresh, ragged appearance to the bank face; absence of surficial bank vegetation.

**Piping** is caused by groundwater seeping out of the bank face. Grains are detached and entrained by the seepage flow (also termed sapping) and may be transported away from the bank face by surface run-off generated by the seepage, if there is sufficient volume of flow. Piping is especially likely in high banks or banks backed by the valley side, a terrace, or some other high ground. In these locations the high head of water can cause large seepage pressures to occur. Evidence includes: pronounced seep lines, especially along sand layers or lenses in the bank; pipe shaped cavities in the bank; notches in the bank associated with seepage zones and layers; run-out deposits of eroded material on the lower bank. Note that the effects of piping erosion can easily be mistaken for those of wave and vessel force erosion (Hagerty, 1991a,b).

**Freeze/thaw** is caused by sub-zero temperatures which promote freezing of the bank material. Ice wedging cleaves apart blocks of soil. Needle-ice formation loosens and detaches grains and crumbs at the bank face. Freeze/thaw activity seriously weakens the bank and increases its erodibility. Evidence includes: periods of below freezing temperatures in the river valley; a loose, crumbling surface layer of soil on the bank; loosened crumbs accumulated at the foot of the bank after a frost event; jumbled blocks of loosened bank material.

**Sheet erosion** is the removal of a surface layer of soil by non-channelized surface run-off. It results from surface water draining over the bank edge, especially where the riparian and bank vegetation has been destroyed by encroachment of human activities. Evidence includes: surface water drainage down the bank; lack of vegetation cover, fresh appearance to the soil surface; eroded debris accumulated on the lower bank/toe area.

**Rilling and gullying** occurs when there is sufficient uncontrolled surface run-off over the bank to initialize channelized erosion. This is especially likely where flood plain drainage has been concentrated (often unintentionally) by human activity. Typical locations might be near buildings and parking lots, stock access points and along stream-side paths. Evidence includes: a corrugated appearance to the bank surface due to closely spaced rills; larger gullied channels incised into the bank face; headward erosion of small tributary gullies into the flood plain surface; and eroded material accumulated on the lower bank/toe in the form of alluvial cones and fans.

**Wind waves** cause velocity and shear stresses to increase and generate rapid water level fluctuations at the bank. They cause measurable erosion only on large rivers with long fetches which allow the build up of significant waves. Evidence includes: a large channel width or a long, straight channel with an acute angle between eroding bank and longstream direction; a wave-cut notch just above normal low water plane; a wave-cut platform or run-up beach around normal low-water plane. Note that it is easy to mistake the notch and platform produced by piping and sapping for one cut by wave action (Hagerty, 1991a,b).

**Vessel Forces** can generate bank erosion in a number of ways. The most obvious way is through the generation of surface waves at the bow and stern which run up against the bank in a similar fashion to wind waves. In the case of large vessels and/or high speeds these waves may be very damaging. If the size of the vessel is large compared to the dimensions of the channel hydrodynamic effects produce surges and drawdown in the flow. These rapid changes in water level can loosen and erode material on the banks through generating rapid pore water pressure fluctuations. If the vessels are relatively close to the bank, propeller wash can erode material and re-suspend sediments on the bank below the water surface. Finally, mooring vessels along the bank may involve mechanical damage by the hull. Evidence includes: use of river for navigation; large vessels moving close to the bank; high speeds and observation of significant vessel-induced waves and surges; a wave-cut notch just above the normal low-water plane; a wave-cut platform or "spending" beach around normal low-water plane. Note that it is easy to mistake the notch and platform produced by piping and sapping for one cut by vessel forces (Hagerty, 1991a,b).

**Ice rafting** erodes the banks through mechanical damage to the banks due to the impact of ice-masses floating in the river and due to surcharging by ice cantilevers during spring thaw. Evidence includes: severe winters with river prone to icing over; gouges and disruption to the bank line; toppling and cantilever failures of bank-attached ice masses during spring break-up.

**Other** erosion processes (trampling by stock, damage by fishermen, etc.) could be significant but it is impossible to list them all.

Serious bank retreat often involves geotechnical bank failures as well as direct erosion by the flow. Such failures are often referred to as "bank sloughing" or "caving," but these terms are poorly defined and their use is to be discouraged. Examples of different modes of geotechnical stream bank failure include soil fall, rotational slip, slab failure, cantilever failure, pop-out failure, piping, dry granular flow, wet earth flow, and other failure modes such as cattle trampling (Figures 2.26 through 2.34). Each of these is discussed below.

Figure 2.26 Soil Fall

Figure 2.27 Rotational Slip

Figure 2.28 Slab Failure

Figure 2.29 Cantilever Failure

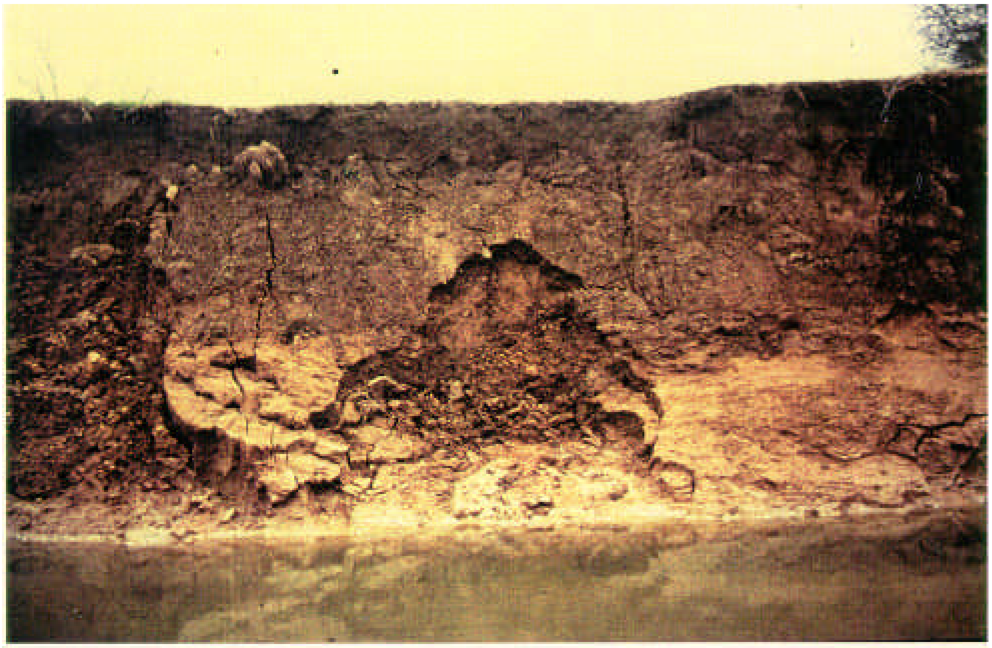


Figure 2.30  Pop-out Failure

Figure 2.31  Piping

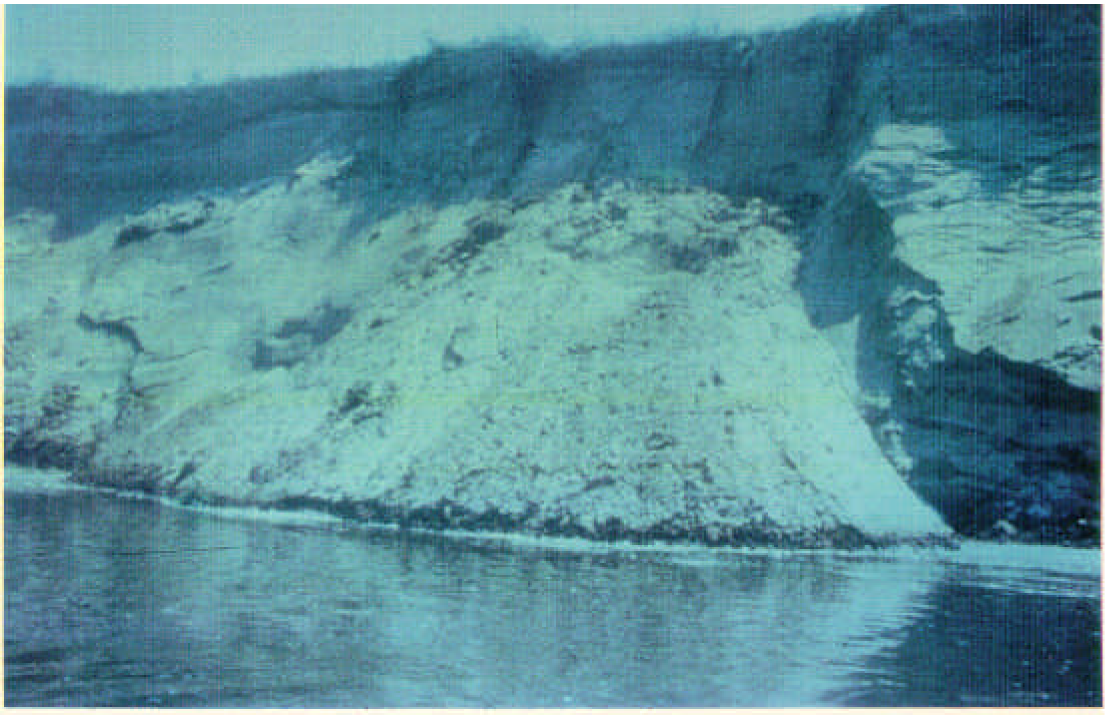


Figure 2.32  Dry Granular Flow

Figure 2.33  Wet Earth Flow

Figure 2.34  Cattle Trampling

**Soil/rock fall** occurs only on a steep bank where grains, grain assemblages or blocks fall into the channel. Such failures are found on steep, eroding banks of low operational cohesion. Soil and rock falls often occur when a stream undercuts the toe of a sand, gravel or deeply weathered rock bank. Evidence includes: very steep banks; debris falling into the channel; failure masses broken into small blocks; no rotation or sliding failures.

**Shallow slide** is a shallow seated failure along a plane somewhat parallel to the ground surface. Such failures are common on banks of low cohesion. Shallow slides often occur as secondary failures following rotational slips and/or slab failures. Evidence includes: weakly cohesive bank materials; thin slide layers relative to their area; planar failure surface; no rotation or toppling of failure mass.

**Rotational slip** is the most widely recognized type of mass failure mode. A deep seated failure along a curved surface results in back-tilting of the failed mass toward the bank. Such failures are common in high, strongly cohesive banks with slope angles below about $60^o$. Evidence includes: banks formed in cohesive soils; high, but not especially steep, banks; deep seated, curved failure scars; back-tilting of the top of failure blocks towards intact bank; arcuate shape to intact bank line behind failure mass.

**Slab-type block** failure is sliding and forward toppling of a deep seated mass into the channel. Often there are deep tension cracks in the bank behind the failure block. Slab failures occur in cohesive banks with steep bank angles greater than about $60^o$. Such banks are often the result of toe scour and under-cutting of the bank by parallel and impinging flow erosion. Evidence includes: cohesive bank materials; steep bank angles; deep seated failure surface with a planar lower slope and nearly vertical upper slope; deep tension cracks behind the bank-line; forward tilting of failure mass into channel; planar shape to intact bank-line behind failure mass.

**Cantilever failure** is the collapse of an overhanging block into the channel. Such failures occur in composite and layered banks where a strongly cohesive layer is underlain by a less resistant one. Under-mining by flow erosion, piping, wave action and/or pop-out failure leaves an overhang which collapses by a beam, shear or tensile failure. Often the upper layer is held together by plant roots. Evidence includes: composite or layered bank stratigraphy; cohesive layer underlain by less resistant layer; under-mining; overhanging bank blocks; failed blocks on the lower bank and at the bank toe.

**Pop-out failure** results from saturation and strong seepage in the lower half of a steep, cohesive bank. A slab of material in the lower half of the steep bank face falls out, leaving an alcove-shaped cavity. The over-hanging roof of the alcove subsequently collapses as a cantilever failure. Evidence includes: cohesive bank materials; steep bank face with seepage area low in the bank; alcove shaped cavities in bank face.

**Piping failure** is the collapse of part of the bank due to high groundwater seepage pressures and rates of flow. Such failures are an extension of the piping erosion process described previously, to the point that there is complete loss of strength in the seepage layer.

Sections of bank disintegrate and are entrained by the seepage flow (sapping). They may be transported away from the bank face by surface run-off generated by the seepage, if there is sufficient volume of flow. Evidence includes: pronounced seep lines, especially along sand layers or lenses in the bank; pipe shaped cavities in the bank; notches in the bank associated with seepage zones; run-out deposits of eroded material on the lower bank or beach. Note that the effects of piping failure can easily be mistaken for those of wave and vessel force erosion.

**Dry granular flow** describes the flow-type failure of a dry, granular bank material. Other terms for the same mode of failure are ravelling and soil avalanche. Such failures occur when a noncohesive bank at close to the angle of repose is undercut, increasing the local bank angle above the friction angle. A carpet of grains rolls, slides and bounces down the bank in a layer up to a few grains thick. Evidence includes: noncohesive bank materials; bank angle close to the angle of repose; undercutting; toe accumulation of loose grains in cones and fans.

**Wet earth flow** failure is the loss of strength of a section of bank due to saturation. Such failures occur when water-logging of the bank increases its weight and decreases its strength to the point that the soil flows as a highly viscous liquid. This may occur following heavy and prolonged precipitation, snow-melt or rapid drawdown in the channel. Evidence includes: sections of bank which have failed at very low angles; areas of formerly flowing soil that have been preserved when the soil dried out; basal accumulations of soil showing delta-like patterns and structures.

**Other** failure modes could be significant, but it is impossible to list them all. Cattle trampling is just one example of a common failure mode.

## 2.3 CLOSING

In planning a project along a river or stream, awareness of even the fundamentals of geomorphology and channel processes allows you to begin to see the relationship between form and process in the landscape. Go into the field and take notes, sketches, pictures - and above all, observe carefully, think about what you are seeing, and use this information to infer the morphological status of the river. When you are in the field, look at your surroundings and try to establish a connection between what you see (form) and why it is there (process). Then you will begin to have some understanding and can perhaps begin to predict what sort of changes may result if your project alters the flow patterns. Then you are beginning to think like a geomorphologist. Dr. Einstein (1972) said in the closing comments of his retirement symposium:

> ***It is in the field where we can find out whether our ideas are applicable, where we can find out what the various conditions are that we have to deal with, and where we can also find out what the desired improvements are.***

# CHAPTER 3

# GEOMORPHIC ASSESSMENT OF CHANNEL SYSTEMS

The previous chapter introduced the concepts of fluvial geomorphology and river mechanics. In this chapter we discuss the tools used in conducting a geomorphic assessment of a channel system. Often users will focus on the particular streambank which is eroding their land, or threatening a building or some piece of valuable infrastructure, and may be tempted to ignore the processes that are occurring both upstream and downstream of the project site. However, we must always remember that the streambank is part of a watershed system that may have a number of interrelated problems that require an integrated solution. Adopting a narrowly focused approach may seem efficient and may even save money in the short term, but may lead to problems in the long term.

## 3.1 GEOMORPHIC ASSESSMENT OF THE SYSTEM

The geomorphic assessment provides the process-based framework to define past and present watershed dynamics, develop integrated solutions, and assess the consequences of remedial actions such as bank stabilization measures. This is an essential part of the design process whether you are planning bank protection for a single streambank, or are attempting to develop a comprehensive plan for an entire watershed. A geomorphic assessment may be divided into the following three components: (1) data assembly; (2) field investigation; and (3) channel stability assessment.

### 3.1.1 DATA ASSEMBLY

The first step in the geomorphic assessment is the gathering and compilation of existing data. The use of historical data enables the identification of trends and provides useful information on rates of change in the watershed. The types of information that should be gathered depend upon the project objectives and types of problems in the watershed. Typical relevant data includes: channel and reservoir surveys, flood history, watershed workplans from the NRCS or other government agencies, bridge plans and surveys, watershed erosion information, geological data, drainage district records, land use records, historical sediment yield information, and aerial photography. This list is not exhaustive, but does provide a guide to the types of information that may be available for a specific watershed. The following is a list of possible sources of historical information:

- U.S. Army Corps of Engineers
- U.S. Department of Agriculture, National Resource Conservation Service
- Agriculture Research Service
- State Highway Department
- State Archives
- United States Geological Survey
- State Land Office
- County Offices
- City and Municipality Offices
- State and Local Historical Societies
- Newspapers
- Local Drainage and Levee Districts
- Local and County Soil and Water Districts

A wealth of information can be gleaned from topographic and geological maps, aerial photos, and in the case of larger watersheds, satellite images. In the planning phase, emphasis is placed on determining means of legal access to the stream, locating areas of possible erosion, breaks in the plan geometry of the stream, channelized sections of the stream, land use, and the location of existing structures. Through the use of a stereoscope, aerial photographs can be utilized to ascertain channel dimensions and gain a more detailed view of the river than is possible using topographic maps alone.

Geologic reports and, particularly, geologic maps are very beneficial in the geomorphic analysis of a drainage basin. The geology and stratigraphy of a drainage basin are the two parameters which have the most effect upon the drainage pattern and long profile of the streams. The natural tendency of a stream is to adopt a course which coincides with the most easily erodible materials available within the drainage basin or to follow surface expressions of structural weaknesses within the earth's surface. Being able to identify the geological and structural features within the basin that exert an element of control on drainage pattern, and determining the stream's response to each different unit is the key to understanding the development of the basin's drainage network.

A historical background of the changes which have taken place within the basin is necessary to fully evaluate the river's response to changing conditions. Historic maps and photos may be available from archives maintained by many of the agencies listed above.

The culmination of this preliminary data assembly phase will enable you to employ field reconnaissance time more judiciously. You will be able to select key reaches of the drainage network where abnormalities in planform and/or profile occur, to locate areas where there are changes in the stratigraphy of the basin, and to obtain a preliminary determination of zones between which the stream may respond differently to the conditions imposed upon it.

## 3.1.2 FIELD INVESTIGATION

The purpose of this section is to provide information that will help you conduct a field investigation of a stream system. General guidance that will help make your field investigation more effective are given and a list of some of the more common morphological and sedimentary features that should be noted while in the field are discussed.

### 3.1.2.1 Introduction

A detailed field investigation of the watershed is extremely important in assessing channel stability because the physical characteristics of the stream are indicators of the dominant geomorphic processes occurring in the basin. Reconnaissance of the watershed by helicopter or small plane prior to the more detailed ground investigation is helpful in determining basin wide characteristics which might otherwise have been overlooked. These visual observations, coupled with the initial analysis of maps and aerial photos, will help to identify problem areas, locate key reaches, and to develop a broad understanding of the general characteristics of the basin so that you can more effectively layout a plan for the detailed ground investigations. Experience shows that watershed perspective developed from these broad overviews allows the investigator to assess bank erosion problems within the context of the wider fluvial system.

Photographs are the cheapest, yet one of the most important products of a field trip. Photographs of channel instability and other problems, as well as photographs of stable sections, are invaluable in presenting observations. It is necessary to record the location, date, and general description of each photo while in the field. Observations made at an unrecorded location are of little value when making a study of this type. Aerial photographs or recent plan surveys are easily carried to the field and provide information that is of sufficient detail to locate key areas or features accurately. When in the field, always keep in mind the maxim that this may be the only chance you have to view the area, for in many instances, time, funding or other constraints may prevent you from returning to the field. Therefore, when you go to the field, it is imperative to obtain complete and accurate field notes and photographic coverage of the study area for use back in the office.

During the stream reconnaissance, it is important to locate and observe areas where the problems associated with the particular study are exemplified. This will give you a perspective into the intensity and spatial distribution of the processes involved. By observing the areas which have the worst problems, you will be able to establish the upper limits of erosion, sedimentation, and/or flooding. It is equally important to visit reaches of the system where these problems are either not as apparent or absent. This will allow you to define a total envelope of values associated with the study area and to understand the variability of the physical characteristics of the various reaches in the stream.

#### 3.1.2.2 Field Equipment for Stream Reconnaissance

Stream reconnaissance involves both qualitative observation and quantitative measurement of key dimensions and geometric parameters of the channel and its morphological features. It is important that the measured dimensions and parameters are representative of the study site or reach but, given the nature of stream reconnaissance, it is not necessary that they are of great precision. Hence, there is no justification for the use of sophisticated instrumentation capable of millimeter accuracy. Also, the need to make observations and measurements in inaccessible locations quickly, and with assistance from only one or two field assistants prescribes the use of equipment that is suited to the purpose and which is convenient to use. As any fieldwork near a waterway carries with it unavoidable hazards, it is essential that individuals performing stream reconnaissance carry basic safety equipment. Finally, the equipment must be portable. Ideally, it should fit into a backpack or rucksack and weigh under 50 pounds.

During development and testing of the approach presented here, a field backpack containing the equipment necessary to perform stream reconnaissance in a wide variety of environments was assembled. The actual contents evolved over time based on the suitability of each instrument or tool, and the need to add or delete some items in order to produce a fieldpack that was versatile, but of manageable size and weight. The final product was a set of equipment that would fit into a single backpack and could easily be carried long distances.

**Rangefinder.** This is an optical distance measuring device based on the 'coincidence' method of rangefinding. It uses a binocular system to produce twin images of a distant object. The observer uses an adjustment knob to make the images coincide and then reads the distance from the instrument to the object on a graduated scale. When properly calibrated and used by an experienced observer, distances can be measured to about two percent accuracy. The range of distances that can be measured varies with the type and cost of the rangefinder. The instrument can be used for distances up to 500 meters. In field tests, accuracy was found to be 3% at 300 meters, rising to 1% at 100 meters.

The rangefinder has the enormous advantages that it is operated by a single person and that it measures distances without the operator having to traverse the intervening landscape. For example, this allows water surface and channel widths to be measured without crossing the stream, saving time and reducing the hazards encountered in field reconnaissance.

**String-operated Pedometer.** This instrument measures the distance walked by the operator by using fine, biodegradable string to spin an odometer as the string is pulled out. The string is broken off at the end of the measured section and left to decompose. The device allows a marked improvement in accuracy over simple pacing, but with no added inconvenience. Like the rangefinder, a single person can measure distance quickly and accurately, but unlike the rangefinder the pedometer does require the operator to traverse the section being measured. Available proprietary brands of string pedometer include "Hip Chain," "Walk-Tax" and "Field Ranger."

**Measuring Tape with Survey Pins and Flagging Tape.** Shorter distances and channel dimensions can be measured using a tape. Usually, two people are required, but if survey pins are used to secure the end of the tape, one person can manage. Banklines and other channel features can be mapped using a compass and tape with surprising accuracy. If repeat reconnaissance surveys are performed, these maps can be useful in establishing rates of bankline migration or channel shifting. Flagging tape is used to mark near-bank objects or vegetation near reference points to aid relocation of repeat sections or transect lines.

**Hand-Level and Pocket Rod or Surveying Staff.** Cross-sections and, on steep streams, longitudinal channel bed and water surface profiles can be surveyed using a hand level and pocket rod. The hand level has a five time magnification and can be used for leveling over distances up to about 20 meters with centimeter accuracy. If shorter distances are acceptable, an Abney Level can be used in place of the hand level.

The pocket rod resembles a 2 meter (or 6 foot) long steel tape which is substituted for a conventional surveying staff. Its advantage is that it retracts into a 50 mm square case that fits easily into the fieldpack. If transportability is not a problem, a telescopic surveying staff may be used instead of the pocket rod.

**Clinometer.** The clinometer is used to measure angles and heights. It works on the principle of a spirit level and can be used to measure the slope or inclination of a bank surface or tree trunk to within one degree. It can also be used together with simple trigonometry to measure the height of objects such as trees, engineering structures or flood marks on buildings.

Estimating bank slopes is notoriously difficult and most untrained observers tend to seriously over-estimate bank angles. The inclinometer can be used in conjunction with the pocket rod or survey staff laid along a bank profile to measure the slope angle of different segments of the bank profile quickly and conveniently and so avoid subjectivity.

**Folding Trenching Tool, Plastic Bags and Marker Pen.** These items constitute the basic equipment for collecting field samples of any sediment finer than coarse gravel. The trenching tool is a type of folding spade used for digging into the bed, bank or bar to extract a sample, examine the stratigraphy or gain access to the substrate. Its advantage over a conventional spade is that it is more compact and fits easily into the fieldpack.

Samples of clay, silt, sand or pea-gravel of sufficient size for particle size analysis can be packed in plastic “zip-lock” bags and labeled using a water-proof marker pen. Bulk sampling of coarse gravels or cobbles requires samples that are too large to carry by hand over all but the shortest distances. Consequently, a size-by-number sampling strategy is preferable when dealing with gravel, cobble or boulder-bed rivers. The necessary equipment is described in the next section.

If it is desired to complete particle size analysis of sand-sized sediment in the field, a field sieve stack and electronic scale can be used, although the sieve stack will not fit into the fieldpack and must be carried separately.

**Probe Rod.** As well as sampling sediments, it is often useful to investigate the thickness of sedimentary units and of loose sediment stored in the bed and in bars, by using a probe rod. A probe rod is also useful to examine the depth and extent of any tension cracks and piping cavities in the banks.

A variety of rods could be used for this purpose, but in the field tests carried out in developing the reconnaissance sheets presented here a jointed, stainless steel gun-cleaning rod was found to be ideal. This rod is strong enough to penetrate most alluvial sediments, including densely packed sands, but can be taken apart to store neatly inside the fieldpack.

**Gravelometer.** The gravelometer is used to measure particle size in the Wolman pebble-count or grid-by-number method of sampling armor layers and other alluvial gravels that are too course to be handled by bulk sampling. It is a light, thin sheet of aluminum alloy with square holes machined in it that follow the Wentworth Scale and range between 2 and 180 millimeters in size.

**Geological Hammer and Rock Identification Charts.** The geological hammer is useful for examination and sampling of rocks and other lithified materials encountered during stream reconnaissance. A pack of rock identification charts, available from most supplier of equipment for petroleum geology, can be very useful in assisting non-specialists in making rudimentary observations concerning geological controls and influences on channel form and stability.

**Miniature Cassette/Voice Recorder.** A miniature cassette or voice recorder is extremely useful for taking notes to supplement the written and photographic records of a reconnaissance trip. The recorder can be used to preserve thoughts or ideas triggered by field observations and may be used during inclement weather and in other situations where writing or taking video would be impossible. Field notes may also be dictated immediately after completing a survey or when traveling between sites, to be typed up later.

**35mm Still Camera.** A compact, automatic 35mm camera is suitable for field reconnaissance. A water proof or, at least weather proof camera should be used. Ideally one equipped with a zoom lense should be chosen for versatility in photographing both wide, panoramic views of the river and floodplain, and detailed shots of bank and sedimentary features that are difficult to access closely.

Slide film is usually preferred over print film because slides are more compact to store and are useful for illustrating and presentations concerning the project. Either prints or slides can easily be scanned into a computer for electronic enhancement and storage, but if computer-based storage is to be used, investment in one of the new, relatively inexpensive, digital cameras will prove cost-effective since the time and costs of processing and scanning are eliminated.

Stereo photographs are especially valuable in that they yield considerably more information than single photographs and can be used to determine morphological changes between successive visits to a site. They can be generated by taking two photographs of the same scene from points separated by a distance of about 2% of the distance to the primary object in the scene. The availability of computer software for analysis of digital-stereo photographs makes this the appropriate technology for the 1990s.

**8mm Video Camera.** A light, compact video camera such as a 'palmcorder' or 'viewcam' is extremely useful in conducting stream reconnaissance because it captures both a moving, visual record of the stream and a verbal commentary. Video can be stored on tape for future reference or is easily transferred to a computer system as part of a multi-media archive.

**Maps and Reference Materials.** Maps of suitable scales to cover the study area are essential tools in stream reconnaissance, both for route finding and identification of landscape features. In remote areas with few reference points in the field, a hand-held GPS will prove useful.

Aerial photographs are also valuable to gain an overview of the channel and its surroundings. Historical as well as contemporary maps and aerial photographs should be obtained if possible because comparison of present and past features adds a valuable time-dimension to the reconnaissance survey.

A geological map yields valuable information on landscape-forming materials and possible geological controls on the fluvial system. Relevant geological maps should at least be consulted prior to setting out on a survey, even if they cannot be taken to the field.

**Miscellaneous Equipment and Supplies.** There is a wide range of auxiliary equipment that can be useful on a stream reconnaissance trip, and the selection of miscellaneous items to be included in the fieldpack is largely a matter of individual choice. Based on experience from trips in the UK, USA and SE Asia, items that have proved useful include a clipboard, umbrella, cagoul or waterproof suit, thigh waders, calculator, magnifying glass, and brush knife or woodsman's axe.

**Safety Equipment.** Safety must be of paramount importance and fieldworkers must assess all potential hazards associated with reconnaissance of a given stream before setting out. Steps must then be taken to avoid hazards where possible and to take all reasonable precautions to minimize the risks where hazards cannot be avoided entirely.

With regard to safety, as a bare minimum, the fieldpack must include a first aid kit, with the contents sufficient to satisfy the relevant safety legislation. Additionally, it would be prudent to include a canteen of drinking water, torch (flashlight), safety matches, sun block cream and insect repellant. If snakes are likely to be encountered and the location is remote from medical aid, then a snake bit kit should be carried. If any boat work is involved, life jackets must be worn.

Given the rapid expansion in the area covered by mobile telephones globally, a mobile phone constitutes an excellent safety item to be carried on a reconnaissance trip.

### 3.1.2.3 What to Look For in the Field

It is not possible to provide an all inclusive list of features that should be recorded in the field that would cover all applications, but it is the intent of this section to list some of the more common types of features that you may encounter in the field.

**Bed Controls**. Channel degradation is the result of an imbalance between sediment transport capacity and supply. One of the most common causes of this imbalance is channelization of the stream which increases the bed slope, causing an excess sediment transport capacity in the channel. Channel degradation occurs as this oversteepened zone migrates upstream, a process referred to as headcutting (see 2.2.2.1). A field indication of the headcutting process occurs in the form of knickpoints and knickzones (Figures 2.17 and 2.18). A knickpoint occurs when the degrading channel encounters resistant bed material and an abrupt overfall is formed. An oversteepened reach of channel representing the headward migrating zone is referred to as a knickzone. Knickzones may consist of a fairly uniform oversteepened reach, or may have a highly irregular profile with numerous small knickpoints. They may extend over several hundred to several thousand feet of channel and, over time often represent 10 to 20 feet of degradation. The shape of a knickpoint or knickzone, as well as the rate with which it migrates up the channel is primarily a function of the composition of the bed material. Therefore, when in the field, it is helpful to document the type of material comprising the knickpoint or knickzone and to assess the amount of drop through this area.

It is also helpful during the field investigation to determine how long a knickpoint/knickzone has been present at its current location. Local residents, supervisors, or soil conservation officers familiar with the area can provide helpful information on the history of the channel. If the knickpoint consists of very resistant material and has been stationary for many years, then it may serve as a geologic grade control that can be relied upon to provide long term grade control. However, geotechnical investigations of the vertical and lateral extent of the material must be performed to ensure that the geologic control will not be undermined or flanked.

**Berms and Terraces.** The formation of berms can indicate an attempt by the channel to establish stability. Berms form after channel incision, widening and slope flattening have progressed to the point where the sediment transport capacity is reduced. This impedes the hydraulic removal of failed bank material at the toe of the bank and also allows sediment deposition to begin (Figure 3.1). The stability of the berms increases after vegetation (particularly woody species such as willow, birch and sycamore) is established. Studies of streams in northern Mississippi and other watersheds indicate that berms often build to an elevation equivalent to the 1- to 2-year frequency flow. Therefore, well established berms not only indicate stability, but also provide insight into the dominant discharge for the stream.

Terraces are another feature that provide information on channel morphology and the history of channel instability. When a channel degrades, it creates a relatively high erosional escarpment which was previously the top bank (Figure 3.2). This is called a terrace or inactive floodplain. Terraces are usually higher than the active floodplain (berms) and may only be overtopped by extreme flood events. In many instances, there may be several different terraces, each the result of separate episodes of degradation. For this reason, a detailed survey of the terraces may yield valuable information about the erosional history of the channel.

**Channel Geometry.** An alluvial stream will size itself in accordance with the magnitude and frequency of the discharges imposed upon the system. This is accomplished through the rearrangement of the bed and bank materials within the channel. Therefore, it is important during the field investigation to observe the dimensions and geometry of the stream, particularly the width and depth. In some instances this may be the only survey information you will be able to obtain, while in others, it may be used as supplementary data for the existing field surveys. The width and depth of the stream should be measured at low water and top bank conditions. If berms or terraces are present, then the width and depth associated with these features should also be measured. As a general rule of thumb, measurements should be made about every 15 to 20 channel widths along the channel. If the stream shows little variation for long distances, then measurements may be made less frequently. Conversely, if the channel geometry is varying widely along the channel, then more frequent measurements may be necessary.

**Bank Stability.** Heights and angles of the channel banks should be field-determined to assist in a bank stability assessment. These data can be determined from surveyed cross sections, but field verification is recommended since surveyed cross sections may not be representative of the entire reach. Field measurements include estimation of bank height with a survey rod or cloth tape, and of bank angle with an inclinometer.

During the field investigation it is also important to observe the bank stratigraphy, mode of bank failures (slab, rotational failures, etc.), and indicators of potential instability such as tension cracks in the upper bank. Proper identification of the bank stratigraphy and its role in channel stability is best determined by an investigator with a background in geology or sedimentology. The classification of the general composition of the observed layers and the percent of the total bank composed by each layer should be recorded.

Figure 3.1 The Formation of Berms Can Indicate a Tendency for the Channel to Re-establish Stability Following a Period of Morphological Change

Figure 3.2 Terrace Formation in an Incised Channel

**Vegetation.** The spatial distribution, size, type, and approximate age of the vegetation existing within and along the channel should be recorded in the field investigation. Vegetation colonizing the channel and along berms should be evaluated with respect to growth and whether or not it may be removed by the next flood event. Not only is substantial in-channel vegetation an indication of lateral stability, but it also impacts the hydraulic efficiency of the channel, and plays an important role in establishing the overall stability of the channel. Simon and Hupp (1992) provide a detailed discussion of the use of vegetative indicators of channel morphology.

**Sediment Data.** Major sediment sources supplying material to the main channel should be recorded during the field reconnaissance. These sources may include the bed and banks of the channel, tributaries, gullies, drainage ditches from roads and highways, and watershed (upland) erosion. In many unstable streams, the bed and banks are a major source of sediment. In this case, the sediment is introduced into the system over a sometimes lengthy reach of channel. Tributaries that are undergoing similar instabilities may be sources of heavy sediment input. During the field reconnaissance, the amount and size of sediment deposited at and just downstream of tributary confluences should be noted.

Sediment sampling provides information on the composition of the sediments derived from each source. In general, channel bed material samples should be taken at the thalweg in order to obtain a representative sample. Analysis of these samples provides information on the spatial variations of grain size within the channel system. Samples of channel bank material, including if applicable, each stratigraphic layer, should be collected. Sediments in tributary mouth bars are used to determine if tributary sediments are radically different from the main-stem channel sediments. It is also helpful to periodically collect bed material samples at several locations across a cross section in order to determine the lateral variability of sediment size in a section.

**Hydrologic Factors.** During the field investigation, estimates of channel roughness should be made for various reaches of the channel. These data are important for calibrating water surface profiles in the detailed assessment phase of the investigation. Roughness (Manning's 'n') should be estimated for the active channel, berms, and the floodplain.

Vegetation and trash frequently preserve evidence of water surface elevations during floods. Debris transported during floods is often trapped in the vegetation. The highwater marks should be recorded, even if the method of measurement is crude. Any evidence of frequent overbank flows such as sand splays, overbank erosion, and crop damage, etc., should also be noted during the field investigation.

**Existing Structures.** The location of all existing structures along the channel should be recorded during the field reconnaissance. A partial list of common man-made features found in streams includes bridges, bank protection, drop inlet structures, culverts, grade control structures, water intakes, and pipelines. An assessment of the structure condition, and the impact on the local channel morphology should be made during the field investigation.

Evidence of scour at bridge pilings and culverts is particularly important as an indicator of the amount of degradation that has occurred since the construction of the structure.

### 3.1.3 CHANNEL STABILITY ASSESSMENT

The third phase of a geomorphic assessment involves an analysis of the channel stability. This is accomplished by the refinement and detailed analysis of all the historical and archive data previously collected, interpretation of the field reconnaissance observations, and the integration of these data to provide an overall assessment of the system.

#### 3.1.3.1 Identification of Geomorphically Similar Reaches

One of the first steps in the channel stability assessment is to divide the channel into geomorphically similar reaches. When establishing reach limits, consideration should be given to: changes in channel slope, tributary locations, presence of geologic controls, planform changes, location of channel control structures (grade control structures, dams, culverts, etc.), changes in bed material size, major sediment sources (gravel mines, sediment laden tributaries, etc.), changes in channel evolution type, or other significant hydrologic or geomorphic changes. Initial reach limits may be made early during the field investigation, but may be refined following more detailed analysis.

#### 3.1.3.2 Specific Gage Analysis

Perhaps one of the most useful tools available to the river engineer or geomorphologist for assessing the historical stability of a river system is the specific gage record. A specific gage record is a graph of stage for a specific discharge at a particular gaging location plotted against time (Blench, 1969). A channel is considered to be in equilibrium if the specific gage record shows no consistent increasing or decreasing trends over time, while an increasing or decreasing trend is indicative of an aggradational or degradational condition, respectively. An example of a specific gage record is shown in Figure 3.3.

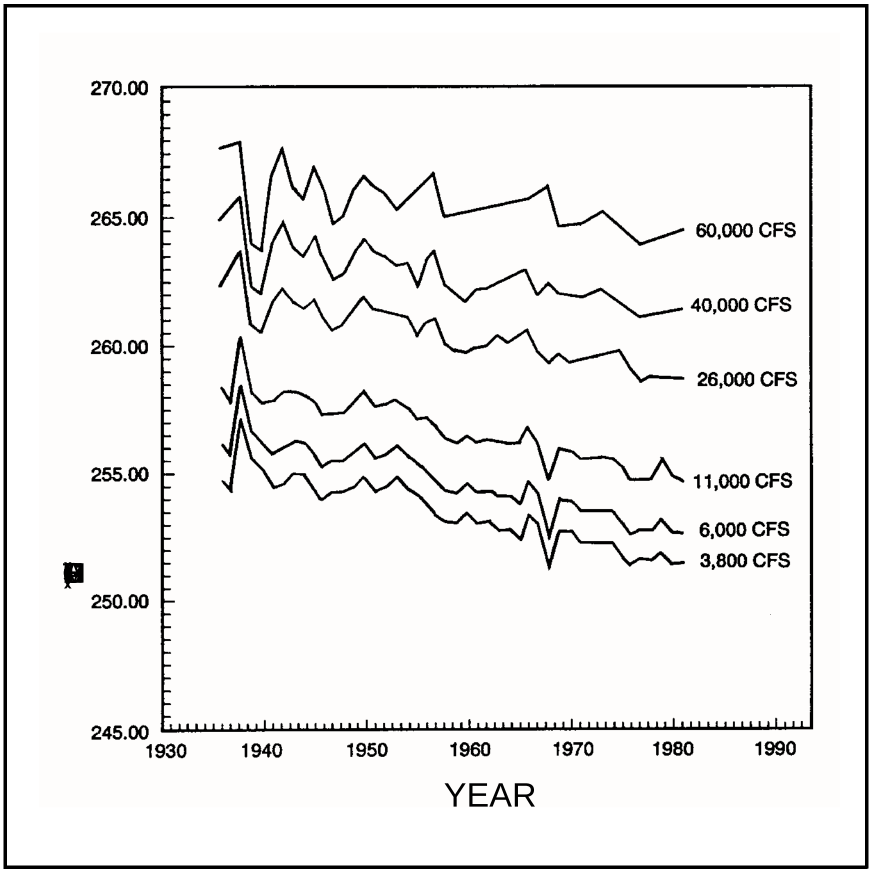



Figure 3.3 Specific Gage Plot for Red River at Index, Arkansas

The first step in a specific gage analysis is to establish the stage-discharge relationship at the gage for the period of record being analyzed. A rating curve is developed for each year in the period of record. A regression curve is then fitted to the data and plotted on the scatter plot. Once the rating curves have been developed, the discharges to be used in the specific gage record must be selected. This selection will depend largely on the objectives of the study. It is usually advisable to select discharges that encompass the entire range of observed flows. A plot is then developed showing the stage for the given flow plotted against time.

Specific gage records are an excellent tool for assessing the historical stability at a specific location. However, specific gage records only indicate the conditions in the vicinity of the particular gaging station and do not necessarily reflect river response farther upstream or downstream of the gage. Therefore, even though the specific gage record is one of the most valuable tools used by river engineers, it should be coupled with other assessment techniques in order to assess reach conditions, or to make predictions about the ultimate response on a river.

### 3.1.3.3 Comparative Surveys and Mapping

One of the best methods for directly assessing channel changes is to compare channel surveys (thalweg and cross section).

Thalweg surveys are taken along the channel at the lowest point in the cross section. Comparison of several thalweg surveys taken at different points in time allows the engineer or geomorphologist to chart the change in the bed elevation through time (Figure 3.4).

There are certain limitations that should be considered when comparing surveys on a river system. When comparing thalweg profiles it is often difficult, especially on larger streams, to determine any distinct trends of aggradation or degradation if there are large scour holes, particularly in bendways. The existence of very deep local scours holes may completely obscure temporal variations in the thalweg. This problem can sometimes be overcome by eliminating the pool sections, and focusing only on the crossing locations, thereby, allowing aggradational or degradational trends to be more easily observed.

While thalweg profiles are a useful tool it must be recognized that they only reflect the behavior of the channel bed and do not provide information about the channel as a whole. For this reason it is usually advisable to study changes in the cross sectional geometry. Cross sectional geometry refers to width, depth, area, wetted perimeter, hydraulic radius, and channel conveyance at a specific cross section.

If channel cross sections are surveyed at permanent monumented range locations, then the cross sectional geometry can be compared directly at different time periods. At each range, the cross section plots for the various time periods can be overlaid and compared. However, it is seldom the case that the cross sections are located in the exact same place year after year. Because of these problems it is often advisable to compare reach average values of the cross sectional geometry parameters. This requires the study area to be divided into distinct reaches based on geomorphic characteristics. Next, the cross sectional parameters are calculated at each cross section, and then averaged for the entire reach. Then the reach average values can be compared for each survey period. Cross sectional variability between bends (pools) and crossing (riffles) can obscure temporal trends, so it is often preferable to use only cross sections from crossing reaches when analyzing long-term trends of channel change.

Comparison of time sequential maps can provide insight into the planform instability of the channel. Rates and magnitude of channel migration (bank caving), locations of natural and man-made cutoffs, and spatial and temporal changes in channel width and planform geometry can be determined from analysis of historical maps. With this type of data, channel response to imposed conditions can be documented and used to substantiate predictions of future channel response to a proposed alteration. Planform data can be obtained from aerial photos, maps, or from field investigations.

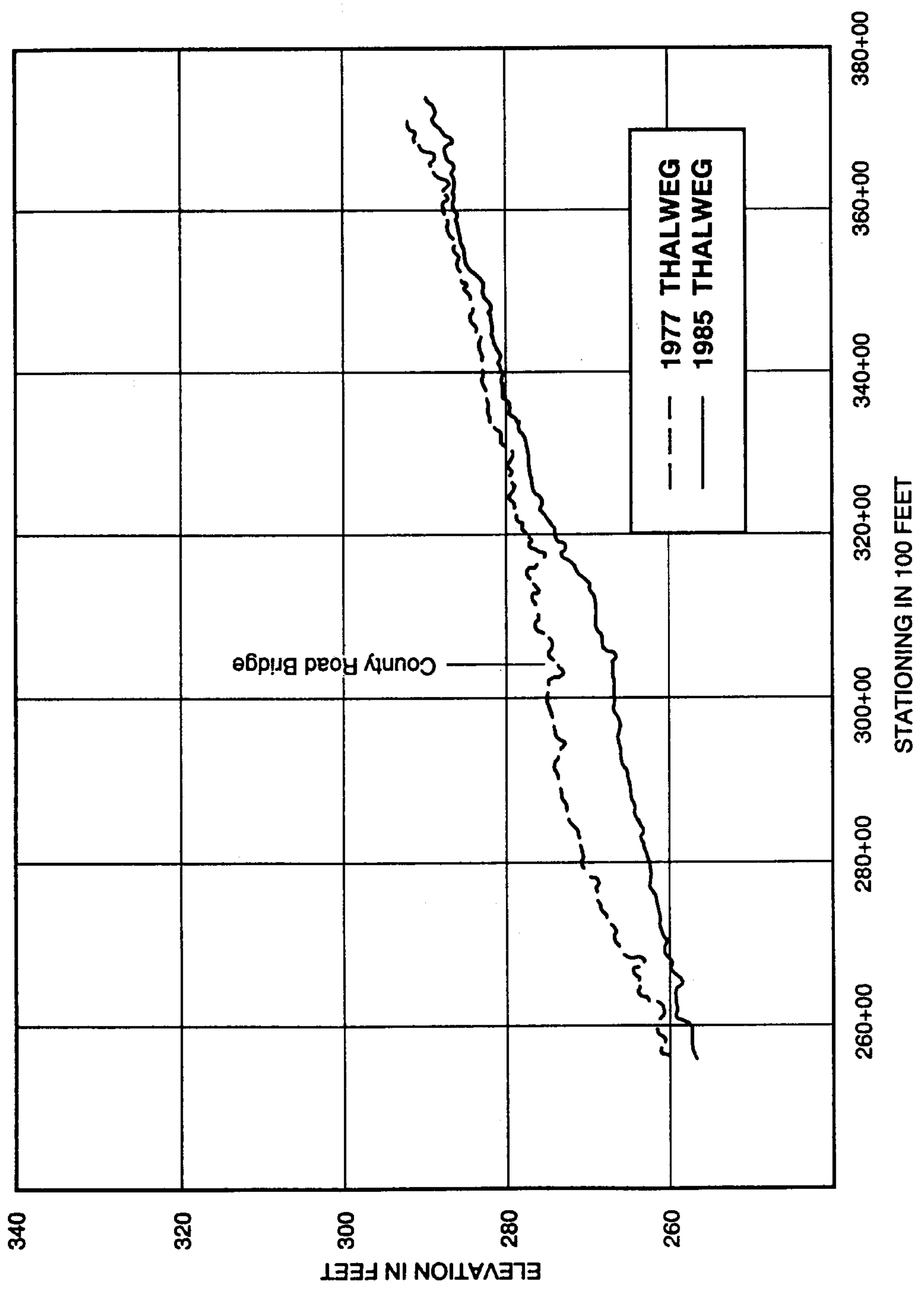



Figure 3.4 Comparative Thalweg Profiles

#### 3.1.3.4 Empirical Methods for Stable Channel Design

Three types of stable channel design methods are reviewed in this section: maximum permissible velocity, tractive force, and regime.

**Maximum Permissible Velocities.** In 1926, Fortier and Scobey presented a channel design method based on maximum permissible velocities for uniform flow. An earthen channel was considered stable if the mean velocity of the channel is less than the maximum permissible velocity for the channel. The USDA (1977) compiled data from Fortier and Scobey (1926), Lane (1953), and the Union Soviet Socialist Republic (USSR, 1936) into a set of design charts. These charts are accompanied by a design procedure found in Technical Release No. 25 (USDA, 1977).

**Tractive Force Design.** Lane (1953) developed an analytical design approach for shear distribution in trapezoidal channels. The tractive force, or shear force, is the force which the water exerts on the wetted perimeter of a channel due to the motion of the water. It is the force exerted over an area of the bed or banks. It is equal to and in the opposite direction from the force which the bed exerts on the flowing water. The average value of the tractive force per unit wetted perimeter (unit tractive force) is given by the following equation (Simons and Sentürk, 1992):

$$\tau_o = \gamma R S_f \tag{3.1}$$

where $\gamma$ is the specific weight of water, R is the hydraulic radius, and $S_f$ is the energy grade line. Simons (1957) provided a detailed process for Lane's tractive stress method.

**Regime Relationships for Channel Design.** In 1895, Kennedy (Lacey, 1931) developed an early regime equation in India on the Upper Bari Doab Canal. The equation is as follows:

$$V_0 = 0.84 \ m \ D^{0.64} \tag{3.2}$$

where: $V_0$ = non-silting, non-scouring velocity, critical velocity;
D = average vertical depth as measured on the horizontal bed of the channel excluding side slopes; and
m = silt factor.

Simons and Albertson (1963) continued regime development by combining data from canal studies in India (Punjab and Sind) and the United States (Imperial Valley, San Luis Valley, and canals in Wyoming, Colorado, and Nebraska). Their motive for additional development of regime analysis was the inadequacy of previous regime methods.

Their data are separated into three groups based on the composition of streambed and streambanks. This eliminates the need for computing bed, bank or silt factors needed for previous equations. Simons and Albertson's (1963) equations are referred to as the Modified Regime Equations.

The U.S. Army Corps of Engineers (USACE, 1994) provides guidance on channel design. Their recommendation is to use locally or regionally developed equations for channel design. However, when this is not possible, relationships are given to provide rough estimates for width, depth, and slope of a channel given the channel-forming discharge and bed material.

#### 3.1.3.5 Summary of Empirical Channel Design Methods

This brief review of empirically-based channel design procedures shows that each of these methods is limited by at least one of the following two constraints:

- the empirical data set is representative of a limited amount of data for the wide range of stream and watershed types, and is not applicable outside the range of data for which each relationship was developed; or

- the concentration of sediment being transported is small, less than 500 ppm, and the method requires that bed and bank materials are static.

While these methods are applicable within the limits for which each was developed, the two primary constraints listed dictate that empirical methods have limited application for natural streams.

### 3.1.4 Computational Design Methods for Channel Design

Two computer programs that can be used to aid in the design of stable channels are SAM (Thomas et al., 1993) and HEC-6 (USACE, 1993). SAM allows for channel design utilizing extremal hypotheses methods, and HEC-6 provides a method for computing bed stability of alternative channel designs.

#### 3.1.4.1 SAM

Thomas et al. (1993) developed SAM, a computer program to calculate the width, depth, slope, and n-value for stable alluvial material. SAM is capable of determining stable channel dimensions, calculating the bed material discharge, and calculating the sediment yield of a stream. SAM is a relatively simple and quick computational procedure that allows preliminary screening of design alternatives, and, in some cases, is suitable for final design or performance monitoring.

#### 3.1.4.2 HEC-6

HEC-6 (USACE, 1993) is a one-dimensional moveable boundary open channel flow numerical model designed to simulate and predict changes in river profiles resulting from scour and deposition over moderate time periods, typically years, although applications to single flood events are possible. A continuous discharge record is partitioned into a series of steady flows of variable discharge and duration. For each discharge, a water surface profile is calculated, providing energy slope, velocity, depth, and other variables at each cross section. Potential sediment transport rates are then computed at each section. These rates, combined with the duration of the flow, permit a volumetric accounting of sediment within each reach. The amount of scour or deposition at each section is then computed and the cross section geometry is adjusted for the changing sediment volume. Computations then proceed to the next flow in the sequence and the cycle is repeated using the updated cross section geometry. Sediment calculations are performed by grain size fractions, allowing the simulation of hydraulic sorting and armoring.

HEC-6 is a powerful tool that allows the designer to estimate long term response of the channel to a predicted series of water and sediment supply. The primary limitation is that HEC-6 is one-dimensional, i.e., geometry is adjusted only in the vertical direction. Changes in channel width or planform cannot be simulated.

#### 3.1.4.3 Integration of Results

The final part of a geomorphic assessment of a channel system is accomplished by integrating the information from all the available analyses. Analysis using each of the geomorphic tools discussed previously may yield a verdict of aggradation, degradation, or dynamic equilibrium with respect to the channel bed, and stable or unstable with respect to the banks. Often the individual assessments produce contradictory results. For instance, the field investigations might indicate that a channel reach is vertically stable, but the empirical relationships and SAM results indicate that the channel should be degradational. In this case you would have to assign a level of confidence to the various components based on the reliability and availability of the data, and your own experience with each tool in order to reconcile these contradictory results. Once again we come back to the fact there is no "cookbook" answer, and that we must always incorporate sound judgement based on insight and experience when making a geomorphic assessment.

# CHAPTER 4

## GENERAL APPROACH TO BANK STABILIZATION

This chapter forms the link between the analysis of channel stability and the selection of appropriate solutions to problems of bank retreat.

"Complex problems often have quick and simple wrong answers" is an apt epigram for problems of riverbank stabilization. River engineers and scientists may be pressured by circumstances beyond their control to plan and construct riverbank stabilization works too quickly, without adequate time or resources for a conceptual evaluation of the problem. The immediate need may be perceived as award of a construction contract when the immediate need may, in reality, be a sound stability analysis of the channel system. The pressure to proceed prematurely to construction should be resisted with logic, while still recognizing that there are practical constraints on data collection and analysis, and that there is often a genuine need for timely corrective action. River scientists and engineers have the privilege and duty to educate the public, representatives of the public, project sponsors, and project managers about river characteristics, and especially the response of rivers to human modification. It must be pointed out that mistakes in detailed design may be only mildly embarrassing and easily correctable, but that mistaking the cause or degree of instability at the outset may doom the entire project to eventual failure.

A more positive perspective is that a system analysis may also identify significant potential benefits from bank stabilization works that might otherwise be neglected. Examples of such benefits are improved water quality and reduction of downstream sedimentation problems. Such benefits are difficult to quantify, and are likely to be achieved only by comprehensive projects, but should be recognized.

Sometimes the obviousness of the bank failure mechanisms obscures the more important underlying causes of bank failure. Without having had either the misfortune of making serious mistakes, or the less traumatic experience of merely observing them, a person may be prone to oversimplify the causes of bank retreat, because a river may take decades to react to imposed changes. Also, the causes and effects of changes are inter-related through complex response. Therefore, the more experienced the observer, usually the more cautious he or she is about making initial judgements.

The salient point is that before beginning the design of bank stabilization works, the concepts and tools presented in the previous chapters on channel stability should be applied systematically and analytically to identify the processes and causes of instability. That analysis may lead to the conclusion that appropriate alternative solutions may involve more than site-specific bank stabilization. Finally, the planner should be aware of other factors which may be peripheral to traditional engineering, but which are essential for a successful project, regardless of scope. This chapter presents a conceptual discussion of those alternatives and factors.

## 4.1 CONSIDERATION OF AVAILABLE ALTERNATIVES

Problems of river instability in general can be addressed by one or more of the following approaches:

- River Basin Management
    - Land treatment
    - Reservoirs
- Bed Stabilization
- Site-Specific Bank Stabilization
- Relocation of Stream or Endangered Facility
- Non-structural Solutions
    - Regulation of navigation
    - Regulation of reservoir releases

The detailed planning and design of the first two alternatives is complex, and beyond the scope of this text. However, the discussion in this section will serve to set them in perspective for the detailed presentation in Chapters 5 through 11 of the selection, design, construction, and maintenance of site-specific bank stabilization, and the presentation in Chapter 12 of concepts of bed stabilization. The last two alternatives are limited in applicability and effectiveness, but consideration of them may be appropriate in some circumstances.

Technical capability to analyze these alternatives is rapidly improving, not only through increased knowledge of river processes, but also through rapid advances in the computational capacity of computers which makes numerical models more powerful. Unfortunately, constraints on time and funds often precludes full application of the more advanced technical tools which are available.

If a problem is purely local, and authority to address a wider project scope does not exist, then the following discussion is academic to a reader searching only for guidance for a conventional approach to bank stabilization. However, if project authority is broader, then the optimum solution may include other components. For example, flood control projects and river restoration schemes may require the evaluation of channel system response to changes in flow or channel characteristics. Bank stability can then be addressed as an integral part of that evaluation.

### 4.1.1 RIVER BASIN MANAGEMENT

This approach is feasible only in comprehensive projects. A large part of the basin must be controlled or managed to be effective in stabilizing the downvalley streams. Significant local cooperation, social and economic constraints, and legal safeguards are involved, and if measures that are comprehensive enough to significantly reduce channel instability are implemented, the project may have complex effects on other aspects of long-term stream behavior.

The operational aspects and effectiveness of basin management depends on basin characteristics such as topography, land use, climate, soil types, vegetation, and rainfall patterns. The two major components of basin management are land treatment and reservoirs.

#### 4.1.1.1 Land Treatment

Major components of land treatment are:

- Riparian greenbelts
- Agricultural practices to minimize runoff and erosion
  - No-till planting
  - Crop rotation
  - Contour plowing and terracing
  - Improved management of irrigation flows
- Improved forestry practices
  - Limits on clear-cutting
  - Careful collecting and hauling practices
- Improved grazing practices
- Stable runoff channels

Benefits of land treatment to channel stabilization are as follows:

> Peak discharges are reduced somewhat, thus reducing streamflow attack on the banks, as well as perhaps providing some flood control benefits.
>
> Sediment supply to the stream system is also reduced, which results in a reduction in channel aggradation and associated flood problems, as discussed by Liu (1989) for the Yellow River, and an improvement in water quality and navigation depths downstream. The precise effects depend on the character of the sediment supply and other basin

and channel characteristics. The drawback of this approach is that a significant change in the sizes of the sediment mixture may induce some channel instability by changing the river regime and triggering some response in channel shape, planform, and/or slope. These are effects which cannot be precisely predicted.

Land loss from overland erosion and gullying is reduced.

#### 4.1.1.2 Reservoirs

The primary purpose of reservoir construction is usually flood control or water supply, but reservoirs also may be designed specifically to induce channel stability. Reservoirs may also be designed specifically to trap sediment. These are sometimes called "debris basins," and require periodic removal of sediment until the basin is stabilized (U.S. Army Corps of Engineers, 1991).

The effect of reservoirs is to reduce peak discharges and sediment supply to the downstream channel. In this regard, their impacts can be viewed qualitatively as the same as land treatment, but the effect is of much greater magnitude. The potential benefit to channel stability is, therefore, much greater, but so is the risk of induced instability due to a change in river regime.

A reduction in peak discharge often reduces bank instability by inducing deposition at the channel margin in the form of berms. In effect, the channel adapts to a lower effective, or dominant, discharge by shrinking. However, reducing the sediment supply to the stream also often induces channel degradation downstream, which can actually lead to mass instability by increasing bank heights. Reducing peak discharge and lowering the flowlines in the downstream channel may also induce tributary instability by lowering their effective base level. This may trigger a reversal of main channel response and lead to its eventual aggradation due to increased sediment supply from tributaries (Biedenharn, 1983).

The effect of reservoirs on rapid changes in river stage can also be significant. The nature of the effect on bank stability depends on the shape of the stage hydrograph of the stream before reservoir construction and the manner in which discharges from the reservoir are regulated, as discussed in 4.1.5.2.

If there is consumptive use or interbasin transfer from reservoirs, for irrigation or water supply, the potential for channel response is further complicated by the reduction of total volume of flow as well as peak discharge.

Bank failure upstream of reservoir impoundments will be decreased by the reduction in flow velocities and bank shear stresses for the length of channel affected by the impoundment. However, associated raising of flowlines due to channel aggradation may create flood control and environmental quality problems.

The preceding account illustrates the fact that, even though reservoir construction may change a stream's water and sediment supply in a relatively straight-forward manner, reliable predictions of the ultimate effects on bank stability and stabilization procedures are elusive.

### 4.1.2 BED STABILIZATION

Assessing the need for bed stabilization measures requires not only a quantification of the active processes of degradation, but also knowledge of the erodibility of bed and substrate materials throughout the entire system, because the rate and magnitude of degradation is very sensitive to bed erodibility. This presents a difficult task if the geologic and morphologic history of the basin is complex. Even with ample data, the erodibility of cohesive soils and weak rocks cannot be accurately predicted. Numerical models do not account for cohesive materials well, and often the best approach is an empirical one, based on the known historical behavior of the particular system in question. If a proposed project will significantly change either the inputs of water or sediment, or the channel slope, then even channel history is not a reliable guide, and design safety factors should be large.

If significant bed degradation is occurring or is expected, then a project should include bed stabilization measures. The only exception is if the requirement or authority for bank stabilization is limited to a very few sites. Local stabilization can be achieved without bed stabilization by designing the toe of the bank protection to function despite general bed degradation. However, this protects only the immediate area of the project. Also, when applied to several sites, the cost of heavy toe protection can exceed the cost of bed stabilization measures without yielding the broader benefits of bed stabilization. A detailed discussion of bed stabilization techniques and design guidance is given in Chapter 12.

### 4.1.3 SITE-SPECIFIC BANK STABILIZATION

This approach is a simpler concept to implement, carries a relatively low risk of induced channel system instability, and is the most immediate and tangible solution. However, unless properly planned and designed, the risk of failure is high. It will be the only component required if project scope is limited to a particular site or local reach of stream, or if the initial conceptual analysis has determined that the stream is in dynamic equilibrium and the predominant cause of bank failure is of local origin. Chapters 5 through 9 discuss the planning and design of site-specific bank stabilization works in detail.

### 4.1.4 RELOCATION OF ENDANGERED FACILITY OR STREAM CHANNEL

These alternatives may require the smallest initial expenditure to "solve," or at least postpone, bank stability problems. They usually have little else to recommend them.

#### 4.1.4.1 Relocation of Endangered Facility

This approach may be dictated by public policy or private preference if it is the least costly alternative. It obviously has no impact on stream characteristics, which can be a positive or a negative factor, depending upon the particular circumstances. It would be most likely to be a positive factor in environmentally sensitive areas where construction of works in the stream channel is to be avoided if possible. The relocation of a structure to a place of safety requires an accurate prediction of the rate and direction of channel migration.

Factors which affect the feasibility of structure relocation are:

Cost;
Degree of safety required to be provided by relocation; and
Social and political impacts of relocation.

#### 4.1.4.2 Relocation of Stream Channel

Factors which affect the feasibility of stream channel relocation are:

Cost ;
Potential for induced damages;
Stream regime; and
Potential environmental benefits of the abandoned channel.

Channel relocation does not always induce instability upstream or downstream, but adjacent landowners are likely to believe otherwise, and legal actions may result. Stream regime and local site conditions determine whether relocation of the channel will cause serious problems. Stabilization of the relocated channel itself may be necessary to avoid problems caused by it migrating after it is constructed. Environmental benefits may result from the formation of a new and abandoned channel, but such benefits are often difficult to maintain permanently unless the sediment load of the stream is relatively small.

### 4.1.5 NON-STRUCTURAL SOLUTIONS

Two situations where a non-structural alternative may be considered are:

Erosion due to navigation traffic in confined channels
Bank instability caused by varying reservoir releases

#### 4.1.5.1 Regulation of Navigation

Regulation of vessel size and speed to reduce erosion from boat or ship passage is a preferable solution and has major environmental benefits, but it may be legally or politically impractical, depending on the local situation. Prediction of the reduction in erosion from regulation is also difficult, which compounds the legal and political problem. Also, public perception of the cause of erosion tends to overemphasize the actual effects of vessel traffic.

#### 4.1.5.2 Regulation of Reservoir Releases

This approach presents much similar benefits and problems as regulation of navigation. Reservoir releases are usually dictated by multiple purposes, and their impact on bank stability is likely to be well down the list of priorities. Also, the public perception of damages from reservoir operation may far exceed the actual damages. In fact, as discussed in Section 4.1.1.2, the net effect of reservoirs is often to improve bank stability. Two exceptions may exist:

> If discharge from a reservoir is frequently and rapidly reduced from close to bankfull to no flow or low flow, geotechnical instability may be increased compared to natural conditions.
>
> If reservoir operation increases the duration of high in-bank flows, the total amount of bank erosion associated with those flows will obviously increase. However, an increase in long-term erosion will be difficult to prove, because the accompanying reduction in peak flows and their associated erosion may more than compensate.

### 4.2 CONSIDERATION OF OTHER FACTORS

A project which performs adequately by traditional engineering standards can nevertheless create public dissatisfaction, because the public takes good engineering for granted, and sometimes focuses on the project's negative aspects. This negative focus may result from inadequate consideration by the project planners of factors which may be beyond what is traditionally considered to be included in the engineering of a project.

The degree of importance and the particulars of these factors are site-and-time specific. It is impossible to address them completely in this text, but they are discussed sufficiently to allow their consideration in the context of proposed bank stabilization, and to raise awareness that further investigation may be appropriate for a specific project.

The factors to be considered are:

Legal and regulatory matters;
Broad environmental issues;
Economic factors; and
Coordination with other interested parties.

### 4.2.1 LEGAL AND REGULATORY MATTERS

Public concerns which are most likely to require permits or conformity to law or formal regulatory procedures are:

Navigation;
Environmental restrictions;
Cultural resources;
Rights of way; and
Other consequential project-induced effects.

Proposed work where commercial *navigation* exists has obvious constraints. Less obvious is the possibility that work may be proposed where there appears to be no actual navigation, but an official classification of the river as a "navigable" waterway exists, requiring the same regulatory procedures.

Public or interagency review of *environmental* aspects of the project may be required. Environmental requirements are rapidly becoming more stringent and complex, and all proposed projects, no matter how small or innocuous, should be critically examined early in the planning stages. Section 4.2.2 presents a broader view of this matter.

*Cultural resources* affected by river stabilization projects are usually archaeological or historical sites. The impact may be positive, as when sites are protected from potential destruction by bank failure, or negative, as when sites are subject to damage from construction activities associated with stabilization projects. Potential impacts are often addressed together with environmental considerations, but specific procedures vary. Consultation with the project sponsor or other project planning authorities is necessary to define the required coordination.

*Right-of-way* for construction, surveillance, and maintenance is always required in some form. It may vary from a simple temporary easement to a complex fee-title purchase from many parties, all of whom may not welcome the project.

*Other consequential effects* of a proposed project may be less well-defined, but perhaps more troublesome. Potential sources of liability and potential litigation are:

Induced bank instability elsewhere;
Induced flooding; and
Physical injury and property damage due to the project.

The engineering aspects of these factors will have been partially addressed in the analysis of stream stability, and will be further addressed during the selection and design of stabilization work. However, the legal aspects are another matter, and early examination by the pessimistic eyes of lawyers reduces the probability of subsequent legal problems arising.

### 4.2.2 BROAD ENVIRONMENTAL ISSUES

The previous section stated that environmental factors must usually be addressed by formal legal or regulatory procedures. This section discusses the following broader concepts:

Historical evolution of public perception;
Opportunities and hazards; and
Public and inter-agency cooperation.

#### 4.2.2.1 Historical Evolution of Public Perception

Future generations will likely judge us primarily on how well we protect the environment. Just as society now regrets indiscriminate dumping of toxic wastes and drainage of wetlands - once condoned for the sake of presumed economic progress - it increasingly regrets many of the environmental sacrifices which have been made in water resource projects for the sake of economic benefits. Brookes (1988) provides a comprehensive discussion of riverine environmental concerns, and a history of the translation of public awareness into law, policy, regulation, and practice.

A policy statement by the Commander of the U. S. Army Corps of Engineers, General H.J. Hatch, on 14 February 1990 illustrates the changing public perspective on environmental issues:

> "....the environmental aspects of all we do must have equal standing among....economics and engineering" and "Our commitment must be to environmentally sustainable development in which we do not compromise the future while we meet current needs."

Nationwide competition within the Corps of Engineers for environmental design awards encourages the application of this policy to practice.

Beyond the altruism of economic sensitivity, most river engineering and management agencies are now more attentive to non-traditional methods for quantifying the economic benefits of environmentally preferable alternatives.

**4.2.2.2 Opportunities and Hazards**

Greater opportunities for creativity and innovation exist in the environmental aspects of riverbank stabilization than in the more obvious and traditional aspects of river science and engineering. A creative attitude can be catalyzed by recognizing environmental considerations as worthwhile challenges and opportunities, rather than as burdensome requirements, although this attitude may be difficult to maintain in the face of pressing schedules and funding constraints. The task is made easier, and goals are achieved more effectively, by addressing environmental concerns as early in the planning process as possible.

Although riverbank stabilization does not have the dramatic environmental impact of reservoir construction or channelization of rivers, significant environmental hazards and opportunities do exist. An important distinction, between *channelization* and *stabilization*, must be made here, so that perception of the former does not unjustly condemn the latter. The distinction is often overlooked by the public, and is sometimes blurred in the literature. Channelization implies significant alteration of long reaches of a stream, often to the detriment of channel stability and environmental quality, whereas bank stabilization often provides environmental benefits, as discussed in 5.2.

All riverbank stabilization projects impact the environment, regardless of the means used to stabilize the channel (Henderson and Shields, 1984). Some potential impacts are:

> Flood plain development or increased agricultural activity may be induced when the threat of channel migration is reduced or removed.
>
> Eroding bank habitat, which is more valuable ecologically than one might think, will be reduced, and the growth of bars and successional vegetation will be altered, perhaps at the expense of habitat diversity. Bed material composition, flow distribution, and other in-channel habitat factors may also be affected.
>
> The formation of ecologically valuable abandoned channels ("oxbows") will be prevented, a serious consequence since existing oxbows usually deteriorate with age due to sedimentation.
>
> Some types of maintenance activities may discourage the reestablishment of natural conditions after the project is complete.

The river scientist's task is to recognize the potential impacts, and then to minimize the bad and maximize the good.

### 4.2.3 ECONOMIC CONSTRAINTS

It was pointed out in Section 4.1 that problems generated by bank instability may sometimes be most effectively and economically solved by less obvious alternatives than site-specific bank stabilization works, especially if the problems are a result of system instability. Section 5.3 further discusses economic factors involved in the selection of site-specific bank protection methods. However, it may be appropriate at an early stage in project planning to consider economic constraints beyond the fundamental engineering concept of obtaining the best value for funds expended. Two such factors are:

> Does the viability of the project depend upon a formally calculated favorable benefit to cost ratio, or will the project be constructed to satisfy specific needs regardless of economic calculations, such as to protect a historic site of indeterminate value? The latter provides the most engineering flexibility, but the former is the most likely situation, in which case the procedure for estimating costs and benefits is probably specified either by the engineer's organization or the project sponsor.
>
> Does the project sponsor have the means and commitment to pay for a well-engineered project, including data collection and an analysis of the causes of the problems and alternative solutions? If not, it is the engineer's duty to point out the hazards of not doing so, in the hope that a more thorough analysis will be authorized.

These considerations are often complicated by the fact that the cost of bank stabilization may exceed the economic value of land and facilities to be directly protected. However, the economic analysis for projects based on broad studies can include identification of less immediately obvious benefits, such as the reduction of sedimentation problems downstream, which may have the potential for benefitting flood control, navigation, and environmental quality.

### 4.2.4 COORDINATION WITH OTHER INTERESTED PARTIES

Most project requirements will be satisfied by adhering to the suggestions in the preceding sections and those in subsequent chapters on design, construction, and maintenance. However, there are usually other interested parties who should be notified or consulted, at least informally. Viewing the specific situation from an “outsider's” perspective, or asking someone less involved in the details of the planning to do so, will usually identify those parties, thus identifying potential non-engineering problems early on. Public notices that may be required by law can also be used as opportunities in this regard, rather than grudgingly satisfying only the letter of the requirement for the notice.

# CHAPTER 5

## SELECTION OF SITE-SPECIFIC STABILIZATION TECHNIQUES

This chapter presents a rational approach to the solution of site-specific bank stabilization problems. It is assumed that application of the insights and analyses presented in the first three chapters has led to the identification of the causes and mechanisms of bank instability. It is also assumed that the conceptual analysis described in Chapter 4 has determined that site-specific bank stabilization is to be a project component, perhaps the only component. Selection of a stabilization approach logically follows that determination, and logically precedes preparation of the detailed project design discussed in Chapters 5 through 9.

The reader might logically question why detailed descriptions of alternative techniques, and their advantages, disadvantages, and typical applications, are not discussed in this chapter rather than later in the text. The author's dilemma was that those topics are even more integral to the design concepts discussed later in the text than to selection of techniques. The discussion, therefore, follows what seems to be the most efficient and comprehensible format overall. Because it is impossible to neatly segregate all topics, some redundancy is unavoidable, but an attempt was made to minimize it. Hopefully the dilemma proves to be less painful in practice than it is in concept, because all pertinent material should be examined by the reader, regardless of its location in the text. Also, some readers will already be familiar with many techniques, and even those readers who are not experienced in bank stabilization will find that one can intuitively grasp many concepts of selection without a tedious search through the text for supporting material.

A framework for selection can be expressed by “Three E's”:

Effectiveness of alternative approaches;
Environmental considerations; and
Economic factors.

The rationale for using this framework is that inherent factors in the properties of a given bank stabilization technique, and in the physical characteristics of a proposed worksite, influence the suitability of that technique for that site. It is essential here to distinguish *suitability*, which is governed by those inherent factors, from *adequacy*, which is governed by design decisions. In other words, the selection phase focuses on suitability, while the design phase focuses on adequacy. Both of these then determine the **effectiveness** of the technique. Many techniques can be designed to adequately solve a specific bank stability problem by resisting erosive forces and geotechnical failure. The challenge to an engineer is to determine the most suitable, the most effective solutions to a specific problem, to recognize which technique matches strength of protection against strength of attack, and which therefore performs most efficiently when tested by the strongest process of erosion and most critical mechanism of failure. **Environmental** and **economic** factors are integrated into the selection procedure, but the chosen solution must first fulfill the requirement of being effective as bank stabilization, otherwise environmental and economic attributes will be irrelevant.

Application of the concepts discussed in this chapter can be enhanced by considering the following philosophical suggestions:

> Be **innovative**. Old concepts can be adapted to specific situations in creative ways, but only if a problem is approached with a creative attitude. Brainstorming with others is helpful in this regard. However, while maintaining a creative attitude, **do not reinvent** the wheel. Learn from others' experience, then reexamine previous practice creatively.

## 5.1 EFFECTIVENESS OF ALTERNATIVE APPROACHES

The following factors of effectiveness influence the selection of a bank stabilization method for a specific project:

- Durability;
- Adjustment to scour or subsidence;
- River depths;
- Foreshore limitations;
- Channel alignment;
- Impact on flowlines; and
- Impact on erosion upstream and downstream.

### 5.1.1 DURABILITY

The following factors should be considered in evaluating the durability of alternative methods:

- Required project lifespan;
- Maintenance requirements and capability;
- Climatic conditions;
- Debris loads, including ice;
- Corrosion and abrasion; and
- Other hazards.

#### 5.1.1.1 Required Project Lifespan

This factor will determine the degree of importance of the durability of alternative methods to a particular project. Required project lifespan is seldom truly quantifiable, even though those projects which require a formal economic analysis must be assigned a specific project lifespan. In practice, the selection of techniques usually involves only a qualitative assessment of required project lifespan, a choice between a “short-term” or a “long-term” lifespan.

Two examples of situations involving only a **short-term** required project lifespan are:

> Emergency stabilization during an unusual flow event, which requires immediate action under conditions not permitting the design and construction of a permanent solution, conditions which may be so rare as to not justify a permanent project.
>
> Local stabilization on a rapidly migrating stream where the attack at the area of concern will be of short duration, and the probability of severe attack occurring at the same point in the foreseeable future is low, or at least acceptable.

The extreme case of required project lifespan being **long-term** is the most common situation. When in doubt, a long-term project lifespan should be assumed, since labor and equipment costs are usually the most expensive part of the project, and a small premium for durable materials will usually provide a cost-effective increase in the factor of safety.

A classification of **intermediate** project lifespan may be appropriate in special cases:

> When an eroding channel is to be stabilized with the expectation that a future project will result in the relocation of either the stream channel or the endangered structure.
>
> When a project for stabilizing a local erosion problem will eventually be endangered from downstream by channel degradation or from upstream by a migrating bend. This factor is, therefore, related to the determination of project components, and to the determination of upstream and downstream limits of the project, discussed in 6.1.1.

#### 5.1.1.2 Maintenance Requirements and Capability

Evaluating this factor involves weighing a lower initial cost or construction expediency against the potential for deterioration of the project as a result of inadequate inspection and maintenance.

If the project sponsor has the capability to monitor the condition of the project, and maintain it as required, a less durable technique may be preferable to a more durable method requiring a higher initial investment. This is particularly likely if inexpensive labor, equipment, and local materials are available. However, the engineer should be cautious about relying too heavily on future maintenance capability, for two reasons:

> The capability of the sponsors to maintain the project may change due to factors beyond their control; and
>
> Human nature may make it easier for the sponsor to agree to an obligation to be incurred in the future than to fulfill that obligation when it comes due.

This is a project-specific determination, but a decision to select a less durable technique should not be made without a frank, even pessimistic, evaluation of the sponsor's long-term commitment and capability, and the durability of the protection methods being considered.

The consequences of failure of the project are linked both to maintenance requirements and to the determination of required durability. However, it is more critical to the detailed design of the project than to the selection of a particular method, since most methods can be designed with a low risk of failure. This is discussed further in 6.6, "Safety Factor."

#### 5.1.1.3 Climatic Conditions

Climatic conditions affect durability through the action of:

Freezing and thawing cycles;
Ice floes;
Heaving;
Wetting and drying cycles; and
Sunlight (ultraviolet deterioration).

The primary vulnerabilities here are the effect of freezing and thawing on stone, the effect of ice floes on armor and indirect protection structures, the effect of heaving on slope armoring, the effect of wetting and drying (with the accompanying damage by bacterial growth and insects, on wooden components), and the effect of sunlight on synthetic materials.

Although stone which is not submerged below the winter freeze line can be damaged by **freezing and thawing**, that is not necessarily a deterrent to the selection of stone as the erosion protection material in cold climates. The primary measure to combat stone deterioration is to insure that high quality stone is used, as demonstrated by local experience or tests. In severe applications, consideration can be given to increasing stone size beyond that which is adequate for hydraulic stability in order to partially compensate for later fracturing.

**Ice floes** uplifting and removing stone and other armor or dike and retard materials may be a problem to be considered in the light of local experience.

Rigid armors are more susceptible to damage from **heaving** than are adjustable armors, flexible mattresses, and indirect techniques.

Permanent submergence greatly reduces deterioration of wooden components from **wetting and drying**, and damage to synthetic materials from **sunlight**. These materials are often used above low water line, but special treatment is usually required. Manufacturers of synthetic materials can provide information for their products.

### 5.1.1.4 Debris Loads

Debris, in the form of uprooted trees or ice carried by the flow, can cause such extensive damage on some streams as to rule out some techniques entirely, and may make the cost of others prohibitive if they are designed to withstand debris loads. However, debris can also affect bank stabilization work positively. Examples of this are:

> Debris can increase the effectiveness of structures in reducing near- bank velocities and accumulating sediment, but debris can also fail structures which accumulate large debris in zones of high velocity, and can make the structures more vulnerable to fire.
>
> Comprehensive stream stabilization will eventually reduce debris loads, but structures must usually be designed for the heavier interim loads.
>
> Debris accumulated by stabilization works may provide habitat for wildlife, but in populated areas the debris may be considered unsightly, and the wildlife it attracts may be pests to nearby residents.

Whether the net result of these interactive events is positive or negative must be determined by applying engineering and environmental judgement to the particular site conditions and project purposes. The sum is often negative, and so the selection of a technique which is likely to accumulate debris, such as permeable dikes or retards, must be approached cautiously if a stream carries large debris loads.

### 5.1.1.5 Corrosion and Abrasion

These mechanisms can greatly reduce the durability of structures which rely on metallic components for long-term structural integrity. The critical factors are water chemistry, air quality, and the concentration and velocity of coarse sediment impinging on the metallic components. A corrosive or abrasive environment does not necessarily rule out the use of vulnerable techniques, but it does dictate that they be approached cautiously and that they be carefully designed.

If metallic components are used the most vulnerable structures are the following:

Flexible mattresses of:
  Concrete blocks
  Gabions
  Used tires
  Wood;
Dikes; and
Retards.

Three ways of avoiding failure in a corrosive/abrasive environment are:

- Select a **technique** that has a high resistance to corrosion and abrasion, such as:

  Stone or other self-adjusting armor;
  Rigid armor;
  Gabions grouted with asphalt or mastic;
  Flexible mattress without metallic components; and
  Stone dikes;

- Use **special components** which are highly resistant to the worst-case agent at the site, such as heavily galvanized or pvc-coated metal and wire, and stainless steel or synthetic fasteners and strand. Even this may not assure success if highly corrosive or abrasive conditions exist, particularly if high concentrations of coarse sediment, high velocities, and highly corrosive water are all present. Even galvanized or coated components are susceptible to “nicking” of the protective layer during construction, which may affect their integrity.

- Use a **“zone”** selection concept, the zones being:

  Below low water;
  Between low water and the permanent vegetation line; and
  Above the permanent vegetation line.

  Materials in the zone below low water must resist only water-borne corrosive agents and abrasive sediments. Materials between low water and permanent vegetation must withstand both water-borne and air-borne agents, and to a lesser degree, abrasion by sediment. Materials above the permanent vegetation line may not need to be highly resistant if site conditions insure that vegetation will become established sufficiently well to function as upper bank protection after the metallic components deteriorate. However, practical considerations of design and construction may make simply using a single design that will withstand the worst case less costly than using the "zone concept."

#### 5.1.1.6 Other Hazards

The following destructive agents are potential problems in some cases:

Vandalism;
Theft;
Animals;
Insects; and
Fire.

Selecting a technique which minimizes temptation will reduce problems with **vandalism** and **theft**. Some materials which are obvious targets for vandals and thieves are posts, boards, concrete blocks and stones of an attractive size and shape, small cables and wire, and easily removable fasteners. Vandals may write graffiti on the smooth surfaces of rigid armor revetments and retaining walls. They may snip readily accessible wires, and cut or build fires on fabric mattress.

Making a potential thief's job difficult will greatly reduce the damage from theft. Increasing the size of components to make their removal or destruction more tedious, peening threaded fasteners, and thinly grouting the surface of vulnerable mattresses, may suffice to keep the work intact. In areas of especially high risk, surveillance arrangements with local law enforcement or security agencies may be worthwhile. The risk will often be reduced by the passing of time, because weathering, the growth of vegetation, and the deposition of sediments serve to make the materials less attractive and accessible to vandals and thieves.

**Animals** are usually considered in the context of environmental impacts of a stabilization project. Occasionally, they can be a problem to consider in the evaluation of the durability of a stabilization method. For example, beavers have a remarkable talent for girdling, felling, or eating vegetation of all sizes and species. This can be disconcerting if the success of the project depends upon quickly establishing a strong vegetative cover. Cows, deer, rabbits, and other animals may also find tender young vegetation on new stabilization plantings to their liking.

To evaluate the potential for problems related to animals, the designer can inspect existing vegetation for heavy browsing, and obtain information from local biologists, resource managers, and agriculturalists. The conclusion will depend on how conducive site conditions are for rapid vegetative regrowth and the nature and density of the animal population. Since many types of vegetation are more effective as erosion protection if pruned to a bushy form, moderate browsing may actually be beneficial to the bank protection function.

In some cases, one or more of the following measures to protect against animal damage may be required:

Providing temporary protection until vegetation becomes well established by using fencing, netting, wrapping, or protective tubes.

Reducing grazing and browsing by using less attractive species of vegetation.

Overwhelm the appetite of the animals by using a species which resprouts vigorously. This often means using a successful native variety of plant.

Obviously, the line between providing beneficial habitat for animals, and risking impairment to the function of the protection project is a fine one. Both goals may not be totally attainable at sites with severe animal depredation.

**Insect** damage can be a problem for wooden components or vegetation. Preservative treatment for wooden components is common practice, and the application of chemicals to vegetation may be effective. However, environmental considerations may rule out these options in some cases.

**Fire** as a factor in selection is highly site-specific. A fire hazard may result from wildfire or from fires set by recreationists. Obviously some materials are liable to catch fire, burn, melt, or be rendered more vulnerable to corrosion if subjected to fire. Relying on fire protection is seldom practical, therefore, if fire is a significant environmental hazard at a site, a material unaffected by it should be selected.

### 5.1.2 ADJUSTMENT TO BED SCOUR AND/OR BANK SUBSIDENCE

All else being equal, a stabilization method which has the ability to adjust to scour or bank subsidence has a significant advantage over those which do not. Completely rigid methods must be carefully designed and constructed, and perhaps even then supplemented by flexible materials at critical points. The property of flexibility reaches its ultimate application in the design of toe protection, discussed in 6.3.

The methods which have this property are adjustable armors, flexible mattresses, and a few types of dikes and retards, and bendway weirs.

### 5.1.3 RIVER DEPTHS

The depth of water expected at the site during the construction period has a significant impact on the range of suitable techniques.

The simplest situation is where a project is to be constructed on a stream which is dry or nearly dry for long periods, so that the entire structure can be built above water. The range of feasible techniques is then very broad and requires no further discussion at this point.

A more difficult situation is that of a stream which experiences flow well below bankfull for long periods, but which has depths of several feet to several meters at the worksite even during base flow. The range of techniques is still broad, but the selection of the material for the underwater portion is critical. The optimum technique frequently will be a "hybrid," involving a design for the subaqueous bank that can be reliably constructed underwater, with a less expensive design for the upper bank. This approach is compatible with the requirement that the toe of bank protection works be functional even when scour occurs, as discussed in 6.3. Examples of this approach are:

> Stone fill placed against and parallel to the bank, with the top elevation of the stone being just above the water surface at the time of construction, with a less expensive treatment used for protection of the bank above the elevation of the stone.
>
> Flexible mattress laid from water's edge out into the channel, with a less expensive treatment protecting the bank slope above the mattress.

The most difficult, or at least the most expensive, situation occurs on large rivers with depths greater than a few meters at the site even at base flow, and the toe of the underwater bank slope far out in the stream, beyond the reach of land-based construction equipment. Two alternative approaches in this situation are to:

"Take the work to the river" by using equipment mounted on barges or other craft, to place stone, flexible mattress, or rigid structures underwater, with the upper bank protection being constructed in a conventional, less costly manner.

"Let the river come to the work" by placing a stone fill on a smooth alignment behind the existing bank. As bank erosion reaches the stone fill, the stone displaces downward until the eroding bank is sufficiently armored to prevent further erosion. This extremely useful technique is known as "trenchfill" revetment when the stone is placed in an excavated trench, and as "windrow" or "falling apron" revetment when it is placed on top of the existing ground.

### 5.1.4 FORESHORE LIMITATIONS

If grading the bank to provide geotechnical stability or to provide a suitable surface for the placement of surface armor is expensive or impractical because of structures near the streambank, or because of restrictions on rights of way, then a technique must be selected which leaves at least part of this "foreshore" or "berm" intact. Restrictions on disposing of excavated material may also create a need to minimize the amount of bank excavation. Two approaches which may be suitable in these situations are:

If immediate stabilization of the bank must be guaranteed, and even a limited amount of bank grading is prohibited, then the bank must be restored, usually with a retaining wall or longitudinal stone bulwark with backfill.

If a limited amount of bank grading can be performed, then a cut-and-fill technique may be adequate and cost-effective. This approach requires careful attention to the protection used on the filled portion of the bank, through the provision of toe protection, a filter layer or fabric, and using an armor material which can adjust to moderate bank subsidence. A noncohesive fill material is best suited to this technique because it can more readily be compacted to prevent subsidence than can a cohesive material. Full compaction is costly, but providing some degree of compaction by traversing the fill with tracked equipment or rollers as the fill is being placed is usually worthwhile, since it greatly reduces future subsidence at minimal expense.

### 5.1.5 CHANNEL ALIGNMENT

Avoiding major realignment of the channel by adapting to the existing general alignment is usually less expensive than realignment, and has the advantage that it changes stream characteristics less. However, navigation considerations, the presence of existing structures, or the unfavorable hydraulic conditions created by an extremely short radius bend or highly irregular bankline, may require a deviation from the existing alignment. If modification is appropriate, a smoother alignment is generally preferable hydraulically and structurally, but an irregular alignment will provide more aquatic habitat diversity.

The most feasible methods if a realignment is necessary are trenchfill or windrow revetment, dikes, and retards.

This topic is discussed further in Sections 5.1.7 and 6.1.2.

### 5.1.6 IMPACT ON FLOWLINES

By altering the channel geometry, and in some cases the channel alignment and length, a streambank stabilization project will change the hydraulics of the flow somewhat. Because the changes are often insignificant, and/or obscured by other factors, such as upstream reservoirs, channelization, or changes in basin land-use, a reliable quantitative assessment of the potential impact of the project on the elevation of the flowline for a given flow may not be possible. However, the sensitivity of potential impacts to the various assumptions that must be made regarding the effect of alternative stabilization methods on channel hydraulics can be examined, and the range of potential impacts can be defined. Fortunately, for most bank stabilization projects which are limited in scope, even pessimistic assumptions will indicate no significant impacts.

Significant lowering of flowlines may have the following undesirable impacts:

Channel degradation on tributaries;
Reduction in amount of aquatic habitat at low flows;
Lowering of ground water level adjacent to the stream;
Decrease in geotechnical stability of channel banks;
Encroachment of vegetation into the channel;
Increase in harbor dredging requirements; and
Disruption in the operation of riverside facilities.

The most severe cases of lowering of stages are usually associated with channelization, and the impacts of bank protection alone may be insignificant, and impossible to determine precisely.

#### 5.1.6.1 Flood Flows

On any stream where flooding is a potential problem, but especially on flood control channels or small streams, the potential impacts of a stabilization project on channel conveyance must be carefully considered.

An armoring approach would not be likely to reduce conveyance. In fact, any armor method is likely to be hydraulically smoother than an existing eroding bank, especially since placing the armor is usually preceded by grading or smoothing the bank. Roughness factors for commonly used materials are given in hydraulic handbooks, and roughness factors for many commercial armors have been determined by the manufacturers.

Dikes and retards, however, could be expected to reduce conveyance, as a result of obstruction of flow by the structures themselves, and as a result of subsequent deposition of sediments and vegetative growth induced by the structures. However, channel adjustments will usually occur, and the ultimate effect may even be a more hydraulically efficient channel if sufficient deepening occurs in the stabilized channel. A precise prediction of the ultimate effect of comprehensive stabilization with indirect methods is not possible. However, the sensitivity for a particular project can be defined by computing flow profiles with and without the project. The most sophisticated approach is to use a coupled flow-sediment numerical model to account for possible deepening of the channel after the initial constriction, but the most conservative approach is to assume that:

> No deepening will occur in the stabilized channel.
>
> Significant deposition will occur within the structures.
>
> Stabilized banks will eventually become vegetated in a manner similar to naturally stable banks, or in the case of works designed to trap sediment, more heavily vegetated than normal.
>
> Protruding structures are significant roughness elements.

Anticipated channel maintenance activities, such as removal of vegetation and sediment deposits from the channel, will have a bearing on the evaluation of ultimate channel conveyance.

#### 5.1.6.2 Low Flows

A less obvious concern focuses on the potential impact of a comprehensive bank stabilization project on the relationship of river stage to low-flow discharge. A lowering of stages at a given low discharge may occur following implementation of the project, if a significant length of channel becomes narrower and deeper, and thus more efficient at low flows. Elliott et al. (1991) discuss the lowering of low flow stages due to complex interrelated factors, including stabilization works, on the lower Mississippi River. The degree of such impact can be estimated, although again not precisely, by the same approach discussed above for flood flows.

### 5.1.7 IMPACT ON EROSION UPSTREAM AND DOWNSTREAM

The final step in evaluating the effectiveness of alternative methods of bank stabilization is to consider potential impacts on channel erosion upstream and downstream of the project.

For comprehensive bank stabilization projects, assessing the long-term impacts of the project on the channel system is a difficult and imprecise task. Projects which also involve bed stabilization, basin management, or regulation of reservoir releases add further complexity, because they are indeed intended to have significant impacts on channel behavior upstream and/or downstream, as discussed in Chapter 4. Defining these impacts is a necessary part of project justification.

Here our focus is much narrower, addressing the topic only as it applies to the potential impacts of alternative site-specific bank protection methods on erosion in adjacent reaches. Fortunately, predicting these impacts is usually less harrowing than predicting the impact of a stabilization project on channel capacity, not because the impacts on erosion in adjacent reaches can be predicted precisely, but because the range of possible responses is more limited, and the impacts are more likely to be local. Therefore, the sensitivity of the prediction to erroneous assumptions is less critical.

A cautious statement can be made that stabilizing a riverbank is not likely to have significant detrimental impacts upstream. It is more likely that the stabilization project itself will be threatened by future channel migration upstream of the project.

The possibility that preventing erosion in one reach will affect erosion downstream is of more concern. Although total erosion downstream is not likely to increase, that may not comfort a landowner who sees a stabilized channel upstream perpetuating the impingement of erosive flows on his or her property, whereas before stabilization, the bar upstream may have been migrating downstream, holding the promise not only of cessation of erosion on the landowners property, but perhaps even having the potential for eventually creating additional useable property by deposition of sediments. That landowner's concern will be even more acute if he or she has no interest in events even farther downstream, where the long-term potential for erosion may be reduced by the project, through a reduction in the rate of meander development.

This dilemma cannot be resolved through the selection of a particular stabilization method, since the very act of arresting channel migration changes future events to some extent, regardless of the method chosen.

In summary:

> Potential problems due to project-induced changes should be acknowledged, and geomorphic concepts should be used to predict the impacts of bank stabilization on channel erosion upstream and downstream. The less a

proposed stabilization approach changes existing channel alignment and geometry, the less complex the potential project-induced changes. If the project will change channel alignment or geometry significantly, or if local interests are likely to be concerned about the impacts of even a simple project, then an expert geomorphic and hydraulic analysis is advisable.

## 5.2 ENVIRONMENTAL CONSIDERATIONS

This section builds upon the broad environmental concepts discussed in 4.2.2. Here we address more tangible factors which influence the selection of a method to solve a site-specific riverbank stability problem.

The ultimate evaluation of the success or failure of some bank stabilization projects may rest on their environmental impacts, because environmental impacts are often judged by a much wider audience than is the project's success as bank stabilization. The weight of public opinion may be unfavorable even if the project is completely effective as bank stabilization.

In spite of many examples to the contrary, bank stabilization projects can effectively address environmental concerns. Because many river engineers and scientists are "environmentalists," and many "environmentalists" recognize the need for riverbank stabilization, progress has been made in transforming the attitude of environmental awareness groups from one of automatic resistance to any river modification, into one of cooperation and dialogue in the development of improved environmental features for incorporation into necessary bank stabilization projects. The selection phase provides more opportunity for this cooperation than does the design stage; therefore, effort expended at this point will provide significant returns.

Environmental considerations and opportunities can be analyzed in the following order:

Potential environmental impacts;
Environmental objectives; and
Identification of environmentally preferable methods.

### 5.2.1 POTENTIAL ENVIRONMENTAL IMPACTS

The difficulty of defining potential environmental impacts of a bank stabilization project is illustrated by the following two scenarios:

Armor protection usually requires bank preparation. This will destroy some vegetation on or near the streambank, although if the bank was eroding, then the bank vegetation would have been destroyed by the erosion anyway. If the

> entire riparian corridor is wooded, though, then neither stabilization work nor continued erosion of the bank would have significantly impacted the character of vegetation adjacent to the streambank. However, if the adjacent land was agricultural, with only a narrow band of native riparian vegetation, then continued erosion would have destroyed that native vegetation. A positive effect of bank protection work could then be claimed, even if some destruction of streamside vegetation accompanied it, particularly if the bank was revegetated with environmentally beneficial plants as part of the work. However, woody vegetation felled by erosion would have provided aquatic food and cover, which will no longer be the case once the bank is stabilized. Aquatic cover can be deliberately added as part of the work, but still the bank substrate is irretrievably altered by the work, as is the input of organic material from fallen vegetation. An ironic climax is sometimes reached when the land adjacent to the stream is developed for man's use once the threat of channel migration is removed.
>
> Indirect protection methods may not change aquatic and terrestrial habitat initially, and often can be considered to improve aquatic habitat initially by the provision of cover and diversity of hydraulic conditions. However, subsequent deposition may destroy some or all of the "improved" aquatic habitat, but the vegetative growth which accompanies the deposition provides terrestrial habitat and a source of organic material. If the deposition becomes relatively flood-free, though, clearing for agriculture may follow, with the end result in extreme cases being the conversion of aquatic habitat to agricultural land.

Dealing with these complexities is best approached by classifying potential impacts, either temporally or by type of impact. One valid approach to the selection of preferred bank stabilization methods is to classify potential impacts into the following categories:

Aquatic wildlife habitat;
Terrestrial wildlife habitat;
Recreation;
Aesthetics; and
Cultural resources.

#### 5.2.1.1 Aquatic Wildlife Habitat

Aquatic habitat may be improved or damaged by bank stabilization work. Potential impacts are as follows:

Water quality

Increase in turbidity during construction.

Decrease in turbidity after construction, since sediment input from bank erosion and failure will be reduced.

Increase in water temperature if significant shade canopy is permanently removed.

Decrease in water temperature if an unvegetated bank is stabilized and vegetated.

Presence of chemical leachates in some materials used for bank stabilization.

Fish habitat

Changes in magnitude and distribution of current velocities.

Changes in amount and type of cover (brush, snags, subaqueous vegetation, and irregularities in the bed and banks). Diversity may be increased or decreased. Evaluating the impact may be difficult because the habitat needs of a species varies with the age of the fish and the season of the year.

Changes in channel depth. Revetments often result in a deeper and narrower channel.

Changes in fish habitat indirectly affecting birds and mammals which prey on fish.

Benthic habitat

Changes in substrate material affecting benthic (bottom-dwelling) organisms. Natural bed and bank material (including brush, snags, and subaqueous vegetation) will be replaced by materials used in the stabilization work. This may be detrimental to some benthic species which are critical to the food chain.

An extreme example of river stabilization work impacting aquatic habitat is provided by some reaches of the Missouri River. The elimination of aquatic habitat as a result of sediment accretions behind indirect stabilization works has been documented by Funk and

Robinson (1974). There is uncertainty over how much of the change is due to hydrologic factors, and how much is man-induced, and further uncertainty over the roles of reservoirs, changing land use, and river stabilization works, but it is certain that stabilization works had a significant impact.

#### 5.2.1.2 Terrestrial Wildlife Habitat

Impacts on terrestrial habitat may be more serious and longterm than is readily apparent. The riparian zone is an extremely important component of an ecosystem, and the ecological consequences of changes there may extend far beyond the immediate vicinity. It provides the essential elements for diverse and productive plant communities (nutrients and water) and for diverse and productive animal communities (food, water, and cover). The riparian zone also serves as migration corridors between isolated pockets of natural habitat in developed areas (Henderson and Shields, 1984). Terrestrial organic matter (vegetative debris and insects) falling into the water is a source of energy for the aquatic ecosystem.

Construction activities may temporarily interrupt wildlife movement in the riparian corridor, and interfere with normal breeding, nesting, and feeding. This is a serious, perhaps unacceptable, impact if the species affected are rare or endangered.

#### 5.2.1.3 Recreation

Recreation may be impacted both indirectly and directly, and favorably or unfavorably, by a stabilization project. Stream-oriented recreation is indirectly affected by aesthetic, aquatic, and terrestrial factors, such as naturalness of the surroundings, water quality, and fishery quality. Potential direct impacts are related to safety and to ease of access to the water. For some projects, particularly those in urban or recreation areas, safety will be an important, even overriding, environmental factor. Judgement applied to specific site conditions will usually adequately identify safety concerns, but consultation with a safety specialist is well-advised if one is in doubt.

#### 5.2.1.4 Aesthetics

Aesthetic impacts are subjective and intuitive, and are usually judged in the context of the specific surroundings. Henderson (1986) suggests that aesthetic impact depends upon the number of viewers, frequency of viewing, and the overall surroundings. For example, the impact may be more important in urban or recreational areas than in an industrialized area, but again this is subjective because an aesthetic setting may be rarer and thus of higher value in an industrialized setting. Smardon (1983) discuss procedures for assessing aesthetic impacts, while Gregory et al. (1992) provide evidence of the predictability of public response to changing channel aesthetics with engineering of urban channels.

#### 5.2.1.5 Cultural Resources

Cultural resources such as archaeological sites may be protected by the installation of bank stabilization works, but may also be disturbed by careless construction activities, particularly if an adequate cultural resources survey was not conducted during the planning phase.

### 5.2.2 ENVIRONMENTAL OBJECTIVES

Philosophically, the potential environmental impacts discussed in the previous section may be viewed as opportunities, and translated into the following general objectives:

Preserve or improve aquatic and terrestrial wildlife habitat.

Avoid disturbance of endangered fish and wildlife during sensitive periods.

Preserve or improve recreational opportunities.

Preserve natural aesthetics.

Preserve cultural resources.

Completely satisfying all of these objectives for a given project may not be possible. They may in fact be mutually exclusive in some cases, especially since the work must foremost be effective as bank protection. Careful planning and expert consideration of the compromises is therefore necessary. The epigram “complex problems have quick and simple wrong answers” was applied in Chapter 4 to selection of project components. It applies equally well to the achievement of environmental objectives, and infers that the advice of environmental professionals is as essential to an engineer attempting to make environmental decisions as the advice of river stabilization engineers would be to environmental professionals attempting to make decisions regarding bank stabilization work. Consideration, preferably by an interdisciplinary team, of the factors discussed below will allow informed decisions on environmental considerations as they affect the selection of a stabilization method for a given project.

This is the appropriate point for discussion of these factors, rather than later under specific protection techniques, because the environmental features which can be incorporated into a protection scheme are likely to be a factor in the choice of the preferred methods.

### 5.2.2.1 Preserve or Improve Wildlife Habitat

As with aesthetics, natural conditions may be viewed as the optimum habitat condition, and as a general concept, work which disturbs natural conditions the least would be favored. However, the degree to which various methods alter existing conditions, and whether the alterations are desirable or not, depends to a great extent on specific geomorphic and biologic site conditions. Still, the following concepts will be generally applicable to the selection of a bank protection method:

> Diversity is preferable to a more sterile, uniform environment, whether the diversity be natural or created by man, as long as critical habitat types are present (Henderson, 1986).
>
> Armoring the streambank usually changes stream geometry and hydraulics less than indirect protection, but alters the morphological characteristics and environment of the bank more, removes more terrestrial and aquatic cover, and provides less diversity. However, stone armor does provide valuable substrate for many benthic organisms, and provides micro-cover for fish, especially if the range of stone size in the specified gradation is large. Deposition within the interstices of some armor materials may to some degree replace in kind the natural bank material.
>
> A "hybrid" or "zoned" approach where different armor materials are used for different elevations on the bank, depending upon the streamflow characteristics and bank erodibility, with vegetation usually being the upper slope component, offers environmental and aesthetic benefits as well as economy.
>
> Indirect protection techniques leave much of the stream bank undisturbed, although by definition, erosion must eventually cease, and deposition will occur in some areas, thus the ultimate condition will unavoidably be altered to some degree. The aquatic habitat provided by the structure itself and by induced vegetation, and the terrestrial habitat provided by induced vegetation will often be superior to natural cover.
>
> Vegetation is almost universally considered to improve both aquatic and terrestrial habitat conditions, although its value and suitability is highly site-and-species specific. It can be used with almost any protection technique.
>
> Providing geotechnical stability by placing fill against the bank, retained with a structure of some type, will disturb less terrestrial habitat than excavating the bank to a stable slope, although the cost may be greater. However, the lower part of the structure will disturb some aquatic habitat, although this may be offset by specifying a "borrow" area configuration which creates new

aquatic habitat. If obtaining and placing suitable fill material is a problem, stone fill can be used in the same way, although the cost may be prohibitive if a large quantity of stone is required to obtain geotechnical stability.

Channel relocation often creates valuable wetland and aquatic habitat in the form of the abandoned channel. However, subsequent deposition usually degrades the aquatic portion of that habitat significantly, even if determined efforts are made to artificially preserve it. The rate and amount of degradation depends upon many factors, as discussed by Gagliano (1984) and Shields and Abt (1989). The relocated channel will be poor habitat initially unless features are deliberately incorporated into the work, and its construction may destroy valuable terrestrial habitat.

Selective clearing and snagging is sometimes used to achieve limited improvement in hydraulic conveyance. In some cases, this concept can be applied to bank stabilization as well, with cleared vegetation being used as armor or indirect protection for the streambank, reinforced by living vegetation, with a limited amount of earthwork as required. This approach can be effective, but its application is limited by site conditions and by available resources, since it is labor-intensive and may require conscientious maintenance. Also, it is difficult to write a performance-type specification for the work.

Selection considerations relating specifically to **aquatic habitat** are:

Protection methods which provide zones of slow currents are desirable. The habits of the endemic species and the hydraulics of the stream will determine how critical this is, and the season of the year when it is most critical. For example:

> Farabee (1986) reports limited sampling on the Upper Mississippi River that found much greater numbers of fish on a revetment of large, loosely placed stones than on a revetment of smaller, tightly packed stones. Large stones were defined as having an average diameter of 2 feet or more, and small stones as having an average diameter of 1 to 2 feet.
>
> Smooth armor materials and stone armor of small stones may create near-bank velocities higher than on a natural bank, which may adversely affect the upstream movement of salmon fry.

Structures which create a wider, shallower cross-section in bends, such as bendway weirs and Iowa vanes, might improve aquatic habitat by increasing the diversity of depth and current velocity available in the bends. The structures would provide cover and a more diverse substrate.

Some species seem to prefer eroding vegetated sites to riprap protected banks, and, therefore would prefer indirect protection to armor. Thus the environmental difficulty of preventing channel migration while still providing optimum habitat for these species may be impossible.

Dike and retard structures provide excellent habitat for some species, and are often productive fishing sites. They are amenable to "notching" (constructing low points in the profile) in order to provide habitat diversity and to reduce longterm adverse impacts on aquatic habitat from excessive sedimentation. Ideally, the notches would be designed so that enough flow passed through the structures to retain high quality aquatic habitat without causing unacceptable bank erosion, unacceptable loss in navigation channel dimensions, or undesirably high velocities within the aquatic habitat itself. However, such a delicate balance is difficult to achieve in practice, even if numerical or physical models are used in design. Design details of the notches may be overwhelmed by the overall geomorphic and hydraulic conditions in the area. Nevertheless, notches may be worthwhile in many cases for providing boat access into the dike field, for allowing movement of fish and other organisms between the main channel and the dike field, and for maintaining water quality in the dike field pools, even if the impact on long-term sedimentation is uncertain.

Materials such as slag may contain chemicals that degrade water quality by leaching, and should not be used if this risk is unacceptable.

Selection considerations relating specifically to **terrestrial habitat** are:

If earthwork is a part of the selected method, diversity can be provided by disposing of the excavated material in an irregular fashion, to create local variability in frequency of flooding and drainage characteristics.

Backfill can be placed over stone and seeded or vegetated with desirable species. This is especially appropriate when stone is placed in an excavation behind top bank, as with stone dike roots.

Backfilling over stone and other irregular armor materials on the upper bank slope will expedite the growth of vegetation, and enhance the natural deposition which sometimes occurs within the interstices of the armor. By backfilling during the construction process, seeding of desirable species at that time may be successful.

Placement of fill on the top of dikes that protrude into the channel sometimes succeeds in inducing the growth of native vegetation that is tolerant to the inundation frequency of the top of the structure. This will of course be unsuccessful if high streamflows scour the fill from the structure.

Sloped banks are conducive to vegetation and to wildlife access to the water, but the act of sloping the bank destroys riparian vegetation. If site-specific wildlife access is critical, then some stabilization methods may require special measures to provide access points.

Other terrestrial habitat measures may be separate from the protection work itself, but may be appropriate as mitigation for the destruction of habitat due to the work. Examples are erecting fallen trees as snags for nesting, feeding, and perching sites for raptors or other birds, and creating artificial mounds for bank-nesting birds such as swallows.

#### 5.2.2.2 Avoid Disturbance of Endangered Fish and Wildlife

If endangered fish or wildlife are present in the vicinity of the project, avoiding disturbance to them, especially during sensitive periods in their life cycle, may impose constraints on the allowable construction period. Since some bank stabilization approaches allow more flexibility in the timing and duration of construction than others, they are more amenable to achieving this objective. For example, construction of a stone windrow on top bank can be accomplished regardless of river stage, whereas a subaqueous armor usually requires low to moderate river stages for successful construction. Therefore, choosing stone windrow would provide more flexibility in the dates of construction.

#### 5.2.2.3 Preserve or Improve Recreational Opportunities

One may view recreation as the link between objectives which are of vital concern to nature, such as the quality of aquatic and terrestrial habitat, and objectives which are of concern only to man, such as aesthetics and cultural resources. All of these influence the quality of recreational activities such as boating, fishing, hiking, hunting, nature study, and swimming.

However, easy and safe access to the stream for pedestrians, boats, or vehicles is a separable factor in the quality of recreational activities, and thus can be clearly weighed in the choice of a stabilization method. The method most amenable to suitable access is armor accompanied by bank preparation in the form of sloping the bank. However, when geotechnical stability is achieved by a wall of some type, then acceptable access may have to be provided as a modification to the standard design. Special features, such as steps, walks, access for the handicapped, fishing points, or boat launching facilities, can be justified in some cases. The cost of such features may be minimal if incorporated into the design and construction of the stabilization work.

Henderson and Shields (1984) suggest that stone dikes can be utilized as boat launching points. However, swift and turbulent flow and sharp dropoffs into deep water are likely to exist adjacent to the dikes, and if provision for boat launching is desirable, then the dike design should take into account the safety aspects. A gently sloping dike profile will be required, which may result in the structure protruding further into the channel than is acceptable from the standpoint of channel alignment and cost. Dikes which are built in shallower water and areas of slower currents may be acceptable from the standpoint of channel alignment, cost, and safety, but since deposition often occurs within the dike field, these dikes may become unusable as launching points.

Dikes are convenient for pedestrian access for fishing, but safety aspects should be considered in design as appropriate for the site conditions.

Other examples of safety considerations are:

An easily traversed armor material will be safer than one which is slippery or jagged. Vertical walls or steep slopes may need guardrails, limited access, or other measures.

If the area is likely to be used by boaters or swimmers, some types of stabilization work, such as “jacks” could be detrimental to safety.

“Drop-offs” (areas where the depth changes suddenly from shallow to deep) may be hazardous.

Consultation with the project sponsor and safety specialists may reveal other advisable precautionary measures.

#### 5.2.2.4 Preserve Natural Aesthetics

The simplest and least subjective approach to comparing aesthetic merits or weaknesses of various methods is to assume that minimizing the visual impact of bank protection work is desirable. Therefore, disturbing the site as little as possible and using natural materials are desirable features for a stabilization method.

Site characteristics obviously vary, and a material such as stone may be aesthetically suitable for most applications, but unsuitable for some. Some armor materials, such as concrete products, are not natural materials, but may be used in a form which gives a somewhat natural visual impression. Other materials, such as rubble or used tires, have little to recommend them aesthetically, although with time, vegetation may reduce the impact.

The aesthetic impact of indirect protection methods is somewhat mitigated by minimizing disturbance to the existing streambank. Still, retards will have a significant impact on site aesthetics, although the degree of impact depends upon the materials chosen, and the impact often decreases with time, as deposition occurs and vegetation is established. Dikes also have a visual impact, but being intermittent, preserve more natural-appearing bank than retards. Other flow deflectors, such as bendway weirs and Iowa vanes, being submerged at least part of the time, have less of a visual impact with respect to the duration of the impact.

Vegetation is aesthetically suitable for almost all applications, but since native species, or species commonly used in landscaping, may not be the most effective erosion protection, a compromise may be necessary.

### 5.2.2.5 Preserve Cultural Resources

Presence of an archaeological site or other cultural resource may influence the choice of a stabilization method in two ways:

> A method which requires bank grading or excavation may not be acceptable if the site would be disturbed.
>
> A method which provides total erosion control may be dictated if the site is very close to a highly erodible bank. Bank filling and armoring, rather than an indirect protection method would then be appropriate.

The hard realities of authority and available resources may preclude a totally satisfactory solution. If no stabilization approach would preserve the cultural resource, and at the same time meet engineering and economic requirements, then relocation of the cultural resource, or in the case of an archaeological site, exploration and salvage, may be the only feasible alternative. If the value of the site is unknown, then exploration and evaluation may be necessary to determine if stabilization is justified. If the bank is failing rapidly and exploration cannot be done immediately, an inexpensive temporary stabilization work may be appropriate in the interim. The temporary work should be compatible with the method likely to be chosen for permanent stabilization if the site proves worthy of preservation.

## 5.3 ECONOMIC FACTORS

The following economic factors influence the selection of bank stabilization measures for a specific project:

- Cost of alternative techniques;
- Available resources; and
- Feasibility of incremental construction.

### 5.3.1 COST OF ALTERNATIVE TECHNIQUES

Because costs vary widely with location and time, discussion here is limited to general concepts, which are universal and timeless.

Suitable methods can be identified using the matrix approach presented in the next section. A preliminary cost estimate can then be used to eliminate cost-prohibitive methods, followed by more precise estimates to be used in final selection.

The final estimate can take into consideration incidental items such as rights-of-way, engineering and design, supervision and inspection of construction, operation and maintenance, and contingencies. Institutional policy may specify that these items simply be estimated as a percentage of construction cost, or a more precise estimate may be appropriate. In the selection phase, it matters only if there are substantial differences in these factors among the methods being considered, which is seldom the case. Some possible exceptions are:

> Significant differences in cost of *rights-of-way* may occur if one method could be constructed with floating plant, but another method would require extensive rights-of-way on the bank in a developed area.
>
> Significant differences in the cost of *engineering and design* may exist if methods are being considered for which standard specifications exist, or for which design assistance is available from the manufacturer. These would require less engineering and design effort than methods for which original specifications must be developed. Also, the data required for analysis and design, may vary between methods. For example, precise riprap design requires hydrologic and hydraulic analysis, and precise geotechnical design requires costly field and analytical work. Protection techniques which involve pile-driving may require borings to determine sub-surface conditions.
>
> Methods requiring a long period of time to construct, such as labor-intensive methods, or methods requiring intensive quality control, such as underwater placement of stone, would have a higher *supervision and inspection* cost than techniques that are quickly constructed with minimal supervision.
>
> In practice, the cost of *operation and maintenance* for well-designed work is usually low, and quantitative comparison of various methods is difficult unless a method is being considered which requires unusually intensive monitoring, maintenance, and reinforcement. A sophisticated analysis would examine "life-cycle" costs, the procedure for which will usually be specified by institutional policy. Some of the environmental features discussed in 5.2 often require more long-term attention than "hard" structures. Vegetative measures and land use management often require monitoring and maintenance in order

to remain effective, and that expense should be accounted for in the life-cycle cost.

*Contingencies* are normally expressed as a percentage of the estimated cost, but if unpredictable changes in site conditions or materials and fuel costs would impact some methods more than others, good practice would be to weight the estimate of contingencies accordingly.

If precise cost estimates to compare techniques cannot be made, and the work is to be contracted, alternative bid items can be used to allow final selection of techniques after bid opening. However, the extra effort required for detailed design of more than one method may not be worthwhile.

### 5.3.2 AVAILABLE RESOURCES

Resources that will be available for the construction of a bank protection project are in the form of funds, labor, materials, and equipment. One of these will be the critical constraint for a specific project.

#### 5.3.2.1 Funds

Funds are usually the greatest constraint on selection of the preferred technique. The sponsor of the proposed work may have a limited amount of funding, and any project that costs more than that simply cannot be built. A requirement that the calculated benefits exceed the project cost may be a similar constraint. Unfortunately, the laws of physics cannot be repealed to fit these constraints. Selection of an inadequate technique simply because it could be built within available funds will obviously be a mistake. Funds spent to construct such a project are wasted, the remnants of the work may create problems, unfavorable publicity may prejudice or preclude future efforts, and the engineer's reputation and credibility suffer.

It is the designer's duty to insist on adequate design, although works built for research or demonstration purposes, or works for which project authority allows speedy reinforcement if necessary, may be designed on the thin edge of adequacy. In this situation, the increased likelihood of failure, and the probable need for rehabilitation work if the project is to remain functional, should be recognized and provided for in the initial stages of project planning.

### 5.3.2.2 Labor

A lack of funds can sometimes be overcome if volunteer or low-wage labor is available. Labor-intensive techniques are:

Hand-placed stone, blocks, or rubble;
Sacks filled with cement mix or other material;
Mattresses of gabions, used tires, lumber, poles, or brush;
Many types of permeable dikes and retards; and
Vegetative treatment.

### 5.3.2.3 Materials

Ingenious use of locally available materials instead of imported materials can sometimes compensate for a lack of funds. Some examples are:

Armor of concrete blocks, sacks, soil-cement, or rubble;
Mattresses of used tires or wooden material;
Gabions filled with stream cobbles;
Dikes and retards of timber or scrap metal;
Dikes with a core of local material capped by armor; and
Vegetative treatment.

A materials-related constraint in urbanized areas, or areas where the terrain is difficult to traverse, is the availability of stockpile and handling areas for bulky materials. A technique which makes efficient use of easily handled material would then be preferred.

### 5.3.2.4 Equipment

Equipment availability will not be a factor for projects advertised for construction on the open market in an area that has general construction contractors. Contractors are usually quite competent to identify equipment requirements. However, the choice of techniques may more be restricted for projects to be constructed by the sponsor's employees, or those of some other specific organization. In that situation, the design engineer should consult with the appropriate construction personnel early in the planning stage to eliminate impractical techniques. Equipment rental or contracting-out those features of the work that require specialized equipment may be feasible. However, problems of coordination and contract administration frequently occur when the work is subdivided, and should not be considered unless significant savings in cost or construction time will be gained.

Equipment-related concerns include access to the jobsite, which in turn is affected by weather, terrain, vegetation, river levels, navigability for floating plant on larger rivers, feasibility of working in the streambed on smaller streams, environmental impacts, and proximity to populated areas. Proximity to populated areas might restrict operations by public objections to dust, noise and vibrations, and concern for potential safety hazards to the public. These problems should be identified early in the selection process so that time is not wasted considering impractical techniques.

### 5.3.3 FEASIBILITY OF INCREMENTAL CONSTRUCTION

The cost of a stabilization project and/or the initial investment can sometimes be reduced by constructing the project in vertical or horizontal increments. It should be noted that the distinction between a planned incremental approach, and having to do additional work later because the original design was inadequate, can become blurred. The distinction is that an incremental approach is planned, and is implemented with the desired result, whereas doing additional, unexpected work is both unplanned and unwelcome.

#### 5.3.3.1 Vertical Increments

This approach consists of utilizing for the first phase a method which will induce deposition of sediment within the protection works, then taking advantage of that deposition to construct the remainder of the work at a reduced total project cost.

This approach requires flexibility, both in funding and in timing of construction, but can be very useful and economical. It can reduce the required height of retards and retaining structures, permit the planting of vegetation at the ideal season for growth, allow the introduction of vegetation at the ideal time to take advantage of induced deposition, and incorporate the use of vegetation to induce more deposition. It also spreads the expenditure for construction over a longer period of time.

This approach is feasible only if:

> An indirect protection technique is suitable for the site. A possible exception to this condition is the use of armor protection on the lower bank, with vegetation to be established on the upper bank during the ideal planting season, after the upper bank has eroded to a flatter slope more conducive to bank preparation and planting of vegetation.
>
> The bank instability is moderate, so that a delay in completion of the work will not endanger the initial work or the object of the work.

A variation of this approach is to plan for reinforcement of the toe of the work after initial toe scour occurs. This approach requires careful monitoring of the work so that the reinforcement can be placed before toe scour progresses to the point of failing the upper bank.

#### 5.3.3.2 Horizontal Increments

This approach consists of initially stabilizing only that length of bank which is the highest priority, then stabilizing the remainder of the project on a delayed schedule. This approach does not decrease total project cost. In fact, the total cost of a project is likely to be higher, but expenditures will be spread over a longer period. This approach is common on comprehensive projects, and it can be used with any technique, but avoiding disaster in the interim requires a reliable forecast of channel migration.

The probable increase in total cost is a result of having to mobilize on the same site more than once, having to tie-in to existing work after the first phase, and perhaps having to repair damage at the ends of the earlier work prior to extending it.

A variation on this approach can be applied to protection work which utilizes vegetation. Several varieties of vegetation can be established initially, then the most successful varieties can be used in a later phase to complete the work.

## 5.4 APPLICATION

This section presents a rational procedure for identifying the preferable erosion protection approach for a proposed project. This procedure provides a means for considering all of the factors which are relevant to the selection, provides a basis for objective decision making, and provides a safeguard against major oversights occurring in the selection process. The concept has been used in the Lower Mississippi Valley Division of the U.S. Army Corps of Engineers to compare standard protection techniques to other potentially useful techniques.

The procedure is flexible in that it can be adapted to either a very disciplined and thorough approach, or to a very informal and rudimentary approach. The procedure is iterative, requiring only the level of effort and number of iterations that are necessary to ensure a competent selection. Estimates of costs need be only to the level of detail that is appropriate for each iteration.

A matrix is the fundamental element of the procedure. The matrix is composed of the factors of effectiveness, environmental suitability, and economics which were discussed in Sections 5.1 through 5.3, along with all of the alternative protection methods which are available for use on a project. The contents of the matrix and the definition of the pertinent factors can be changed to satisfy a particular project. The matrix can also be expanded into sub-matrices as appropriate for a specific project. For example, environmental factors can be listed in detail in a sub-matrix. Also, in some cases it will be appropriate to subdivide the streambank into two or more zones of elevation, to provide for the selection of a composite or “hybrid” protection technique.

A suggested general matrix is shown by Table 5.1. A beginning point for applying the matrix to a proposed project is to first eliminate factors that are irrelevant, and protection methods which are obviously infeasible, for that project. For example, if the proposed project is on a shallow or ephemeral stream, then an irrelevant factor would be "Use In Deep Water." Similarly, a method can be immediately eliminated from further consideration if a severe deficiency in even one relevant factor precludes that method from being effective or environmentally acceptable. For example, if the work is to be constructed in deep water, then some types of retards would be infeasible to construct. The number of factors and the number of alternative methods which will be eliminated in this initial iteration will vary, depending upon the complexity of project circumstances and the experience of the evaluator. An evaluator with sufficient experience may be able to make a competent selection simply by objectively and qualitatively evaluating the basic factors that are pertinent to the project, without further iterations.

The second iteration can consist of assigning qualitative ratings to each remaining protection method for each remaining factor. The form of these ratings can be "+" for a favorable rating, "-" for an unfavorable rating, "0" for a neutral rating or for a factor that is not relevant to a particular technique, and "?" if the rating cannot be determined at this point in the analysis. A qualitative rating of the importance of each factor will be inherent in this iteration. For example, a deficiency in resistance to fire is not usually as serious as a deficiency in ability to adjust to scour. The end result of this iteration will be the identification of seriously deficient techniques, which can then be eliminated from further consideration.

If the optimum technique has not been identified by this point in the process, the final iteration can consist of assigning numerical ratings to the remaining techniques, with each technique being given a rating for each factor of effectiveness, environmental suitability, and economics. The numeric scale for these ratings is a matter of choice, but as a practical matter one is not likely to be able to distinguish more than five levels; for example, the ratings could range from "1" for "poor" or "least favorable," to "5" for "excellent" or "most favorable." In addition, it will usually be appropriate at this time to numerically weight each factor according to its importance to the success of the project, with the weight being based on site conditions and the project sponsor's needs and capabilities. Also, an approximate estimate of costs for each remaining method will probably be appropriate at this point.

The preferred method can perhaps now be identified by summing the scores, and considering the total score for each method along with estimates of cost. If the choice is still not clearcut, more detailed estimates of cost can be prepared in order to make the final determination. If uncertainty still exists at that point, the evaluator can select the protection

Table 5.1 General Matrix for Selection of Erosion Protection Method

| | EFFECTIVENESS | | | | | | | | | | | | | ENVIRON EFFECTS | | ECONOMICS | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | DURABILITY (SUSCEPTIBILITY TO): | | | | | | | ADJUST TO SCOUR | USE IN DEEP WATER | EFFECT ON: | | | | People | Critters | COST | | | SAVINGS BY INCR CONSTR |
| | Inadeq Maint. | Freeze/ Thawing | Wet Drying | Debris Damage | Corros Abras | Fire | Vandals etc. | | | Fore-shore | Chan align | Chan capac | Upstr. down | | | Labor | Mater | Equip | |
| <u>Stone Armor</u><br>Riprap<br>Trenchfill<br>Windrow | | | | | | | | | | | | | | | | | | | |
| <u>Other adjust armor</u><br>Sacks<br>Blocks<br>Rubble<br>Slag<br>Shredded Tires | | | | | | | | | | | | | | | | | | | |
| <u>Rigid Armor</u><br>Concrete<br>Asphalt<br>Grouted Riprap<br>Soil-Cement<br>Ice | | | | | | | | | | | | | | | | | | | |
| <u>Flexible Mattress</u><br>Gabion<br>Concrete Block<br>Fabric<br>Used-Tire<br>Wooden | | | | | | | | | | | | | | | | | | | |
| <u>Dikes</u><br>Permeable<br>Impermeable | | | | | | | | | | | | | | | | | | | |
| <u>Retards</u><br>Permeable<br>Impermeable | | | | | | | | | | | | | | | | | | | |
| <u>Other Flow Defl</u><br>Bendway Weirs<br>Iowa Vanes | | | | | | | | | | | | | | | | | | | |
| <u>Vegetation</u> | | | | | | | | | | | | | | | | | | | |

approach that he or she feels the most "comfortable" with. This may be the one that either the designer or the project sponsor has had success with, or the one which involves the least modification to the stream.

It is often advisable to develop an "environmental sub-matrix" to assist in the selection of the preferred stabilization method. The first step is to consider the five environmental objectives which are discussed in 5.2.2, and then refine them to suit the specific project at hand. Then, list those objectives, or the more specific goals derived from them, in a sub-matrix which also lists those bank stabilization methods which will meet the prerequisite of accomplishing the primary function of bank stabilization. That prerequisite is defined by the "effectiveness" portion of the general matrix.

Numerical ratings and weight of importance for each environmental factor are then assigned to each alternative technique, as discussed above. Adjectives such as "highly detrimental," "moderately detrimental," "no effect," "moderately beneficial," and "highly beneficial" can be used as an aid to visualizing the numerical ratings for environmental factors. The resulting total score for each method can be used to simply identify the environmentally preferable method, or it can be added to the scores for effectiveness and economics to obtain an overall ranking of the alternatives. An example sub-matrix is shown in Table 5.2. In this simple example, stone paving would be more environmentally desirable than a retaining wall.

Table 5.2 Example of Very Simple Environmental Sub-matrix

| Feature | Weight | | Beneficial Attributes of Each Method to Each Feature | |
|---|---|---|---|---|
| | | | Bulkhead | Stone Paving |
| Riparian vegetation | (3) | | 1 | 2 |
| Aquatic habitat diversity | (3) | | 1 | 3 |
| Substrate for benthos | (2) | | 1 | 3 |
| Access to water | (2) | | 1 | 2 |
| Water quality | (2) | | 3 | 3 |
| Aesthetics | (1) | | 3 | 2 |
| SUM OF WEIGHT X BENEFIT | | | 19 | 33 |

One source of environmental information which has an iterative user-interactive format is the U.S. Army Corps of Engineers' "expert system" ENDOW (Environmental Design of Waterways), which operates on a personal computer. Like all approaches, the use of ENDOW involves some subjective judgements, and is subject to the usual hazards associated with attempts to simplify complex situations. It is not intended to remove all uncertainty from environmental considerations, but it does provide useful insights into the selection of environmentally sound protection techniques. ENDOW also contains modules for evaluating environmental features for flood control channels and levee projects.

A troubling irony will become apparent in the course of using the matrix approach. Even though this is a quantitative and objective procedure, one can arrive at different conclusions by changing the weight of the various factors and/or changing the ratings of the different stabilization methods, while still remaining within a reasonable range of values of weights and ratings. This irony illustrates that the procedure is not a substitute for engineering judgement, but is merely a catalyst for engineering judgement, a mechanism for considering all relevant factors and all appropriate alternatives, thus reducing the probability of a major oversight.

# CHAPTER 6

## GENERAL PRINCIPLES OF EROSION PROTECTION

This chapter provides general guidance for the design of the erosion protection component of riverbank stabilization work. The discussion in this chapter will also help in understanding the basis for the approach to selecting the preferred stabilization method, which was presented in Chapter 5.

The great variation in site conditions and project constraints, and the almost infinite possible combinations of materials and design details of the alternative methods of erosion protection, make a "cookbook" approach to design impractical. A committee of the American Society of Civil Engineers reported in 1965 that "Because of the complex nature of alluvial streams, design of channel stabilization works is based largely on experience" and this is still the case. As Simons and Li (1982) state, "...handbook-type analyses and designs [for river training and bank stabilization] usually lead to poor solutions of specific problems." Hemphill and Bramley (1989) similarly state "...good design practice necessarily involves *judgement and experience*, and [we] can only draw attention to the various aspects which need to be taken into consideration or on which *expert advice* should be sought."

This chapter seeks to assist the designer's *judgement* by providing a synthesis of *experience*. Additional information on specialized topics can be obtained from the references cited in this text.

To present guidance in a structured fashion, the chapter is divided into the following six sections:

Applied Geomorphology;
Hydraulics;
Toe Protection;
Surface Drainage;
Manufacturer's Recommendations; and
Safety Factor.

This list, along with environmental considerations (see 4.2 and 5.2) can be used as a "checklist" to insure that the designer has not overlooked any major factors. For some stabilization methods, one or more factors can be quickly dismissed. For example, provision for surface drainage is not usually required for "indirect" protection methods, while manufacturer's recommendations apply only to commercial products.

At this point, a caveat which has been inferred previously in Chapters 4 and 5 must be reiterated: to arrive at the point of designing a site-specific bank stabilization project, the designer must make one of the following judgements:

that the fluvial system is in equilibrium;

that system instability exists, but that channel changes will not significantly affect the bank stabilization project, or that the bank stabilization can be designed to accommodate such changes; or

that system instability exists, but that it will be corrected in conjunction with site-specific bank stabilization if appropriate.

In practice, the problem of making an assumption regarding system instability is not as difficult as might first be imagined. In practice, the first two conditions often apply, and successful bank stabilization projects have been built without the designers taking into account the possibility of system instability. Ideally, however, any element of chance can be removed from project design by first applying the principles discussed in Chapters 2 and 3 to correctly determine the causes and mechanisms of bank retreat.

## 6.1 APPLIED GEOMORPHOLOGY

This section deals with the integration of the understanding of stream characteristics outlined in Chapters 2 through 3 into the decision-making process with regard to the location of bank stabilization works. The specific aim is to describe how best to use "applied fluvial geomorphology" to make the following two basic decisions:

Location of the upstream and downstream limits of work

Alignment of the work with respect to the stream channel

### 6.1.1 UPSTREAM AND DOWNSTREAM LIMITS OF WORK

This initial design decision is based on an analysis of channel migration, a consideration of the minimum requirement for length of bank to be protected, and other detailed considerations.

### 6.1.1.1 Prediction of Channel Migration

The key to success in choosing the upstream and downstream limits of the work lies in the prediction of channel migration. The basic parameters of channel migration, or meandering, are shown in Figure 6.1. Note that both sinuous and straight streams exhibit characteristic patterns and spacings of bars, pools, and crossings. The one particular characteristic of these patterns which is an invaluable aid in a correct determination of the siting of stabilization work is that the movement of bars, pools, and crossings has components both perpendicular to the axis of the meander belt and downvalley. As a rule, the greatest movement is usually downvalley.

While this is a sound general rule, in nature the variability of the bed and bank materials usually distorts the actual pattern from the ideal pattern of movement to some degree, as shown on Figure 6.2. Therefore, it is important to obtain some verification of recent migration trends for each specific location.

There are four potential sources of data which can be used in this verification. Listed in approximate descending order of reliability, they are:

(a) Historical geomorphology based on *documentary information* on channel evolution from hydrographic surveys, topographic maps, and/or scaled aerial photographs. With the position of the stream channel documented at two or more points in time, the length of bank which has been subject to erosion can be identified.

(b) Interpretation of *existing planform* ("process geomorphology"). If data are available only for the present point in time, the principles of downvalley migration and increase in bend amplitude, together with experience derived from similar situations on other streams, can be used to predict likely locations for continued erosion if the bank is not stabilized.

(c) Historical narrative accounts of channel shifting based on *interviews* with local residents, landowners, and interested individuals. While these observers may not be scientists, and may not be completely unbiased in their opinions, they can provide useful information on historical erosion and channel changes.

(d) Numerical or physical *morphological modeling*. Numerical modeling of meandering is a developing science that shows promise, but unfortunately, reliable prediction of future migration requires that the model be verified using past migration trends as documented by one or more of the first three sources of information listed above. Therefore, to some extent, the necessary information must already be available before numerical modeling can be undertaken. The same is true of physical modeling, with the additional disadvantages of requiring

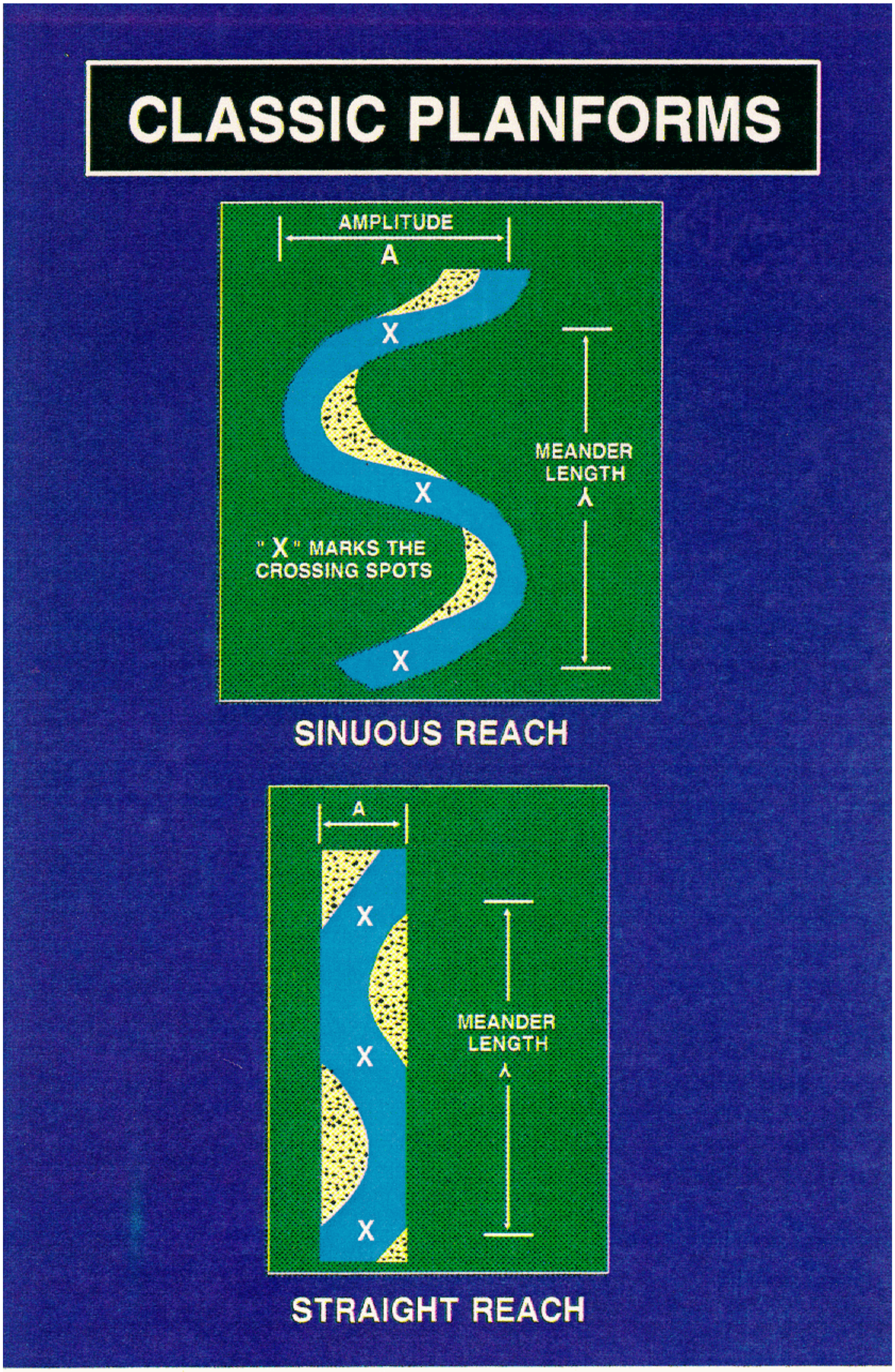



Figure 6.1 Classic Planforms for Straight and Sinuous Channels

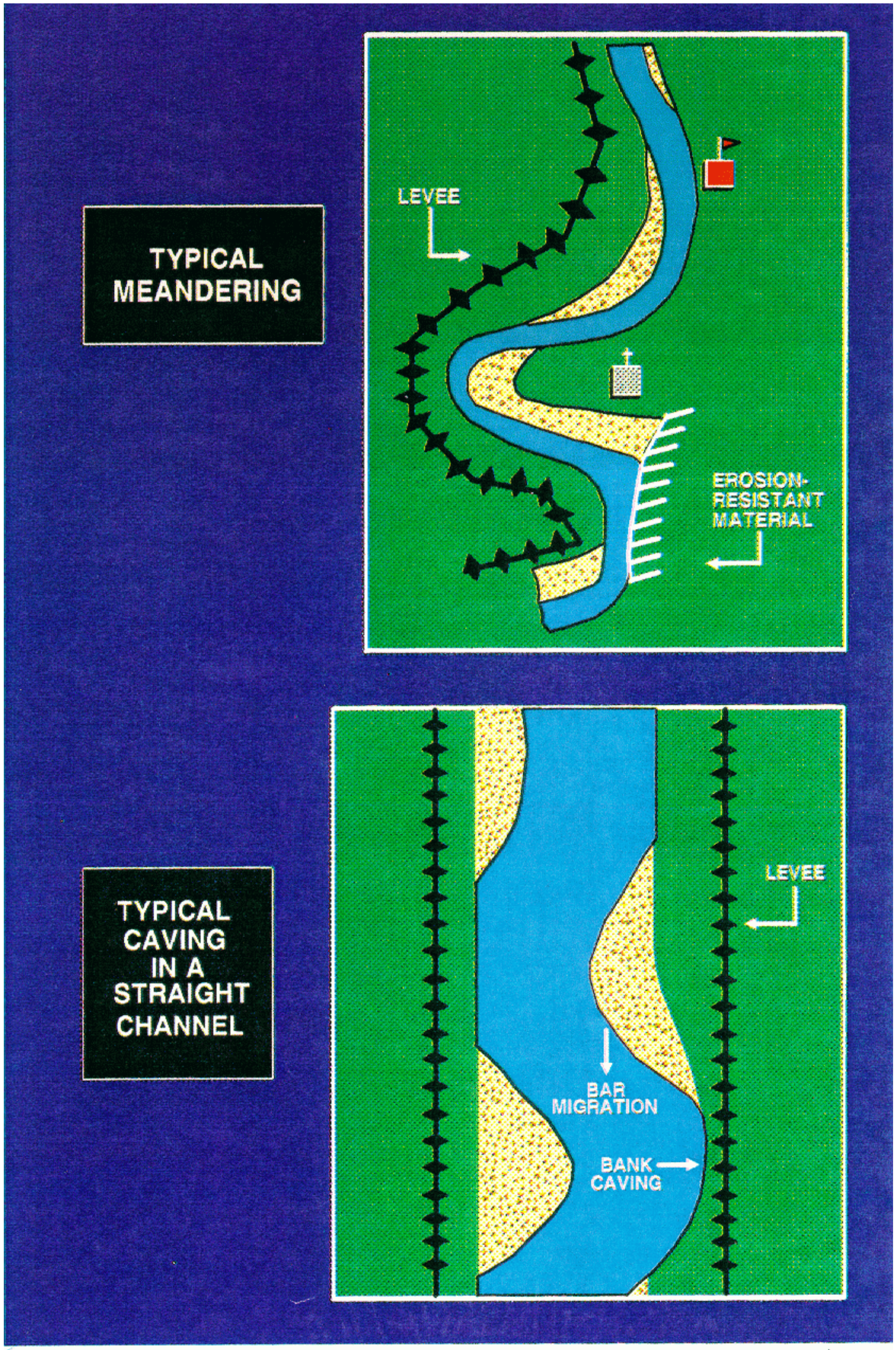



Figure 6.2 Effects of Varying Bed and Bank Materials on Planform Characteristics

> a lengthy time period for completion of modeling, and the high cost of model operation. Numerical and physical models are much more useful for studying hydraulics of flow, changes in the bed, and "generic" meandering than for predicting long-term channel migration for specific locations. The major limitations are that meander evolution includes a random element, and that bed and bank materials are seldom uniform, so that it is impossible at present to predict the migration and evolution of a particular meander bend through heterogeneous flood plain sediments.

Experience in geomorphological interpretation is required for the designer to fully exploit these sources of information. There are at present no "cookbook" solutions.

#### 6.1.1.2 Minimum Length of Protection

The upstream and downstream limits of the work in terms of the minimum length of bank to be protected depends upon the nature of the erosion problem and the scope of the project. Generally, the scale of the solution can be classified as one of the following, listed in ascending order of required length of protection:

> The streambank immediately adjacent to a threatened structure.
>
> The length of streambank which is retreating rapidly enough that localized protection would not guarantee adequate protection for the required project life.
>
> For comprehensive projects which require fixing a great length of streambank in a stable position for navigation, flood control, irrigation, or other long-term project purposes, the minimum requirement is to stabilize the entire bankline which must remain in its present position or in some other predetermined position so that navigation channel alignment, flood control works, irrigation structures, or other project features are not threatened within the required project life.

Figure 6.3 illustrate these situations.

Even if the minimum requirement is stabilization of a single point on the streambank, a geomorphic analysis of channel migration as described in 6.1.1.1 should be performed to provide insight into the stream dynamics that define the instability problem and its solution, and to predict future problems that might arise due to channel migration. Only then can the location of the protection required to meet the specific need be assured.

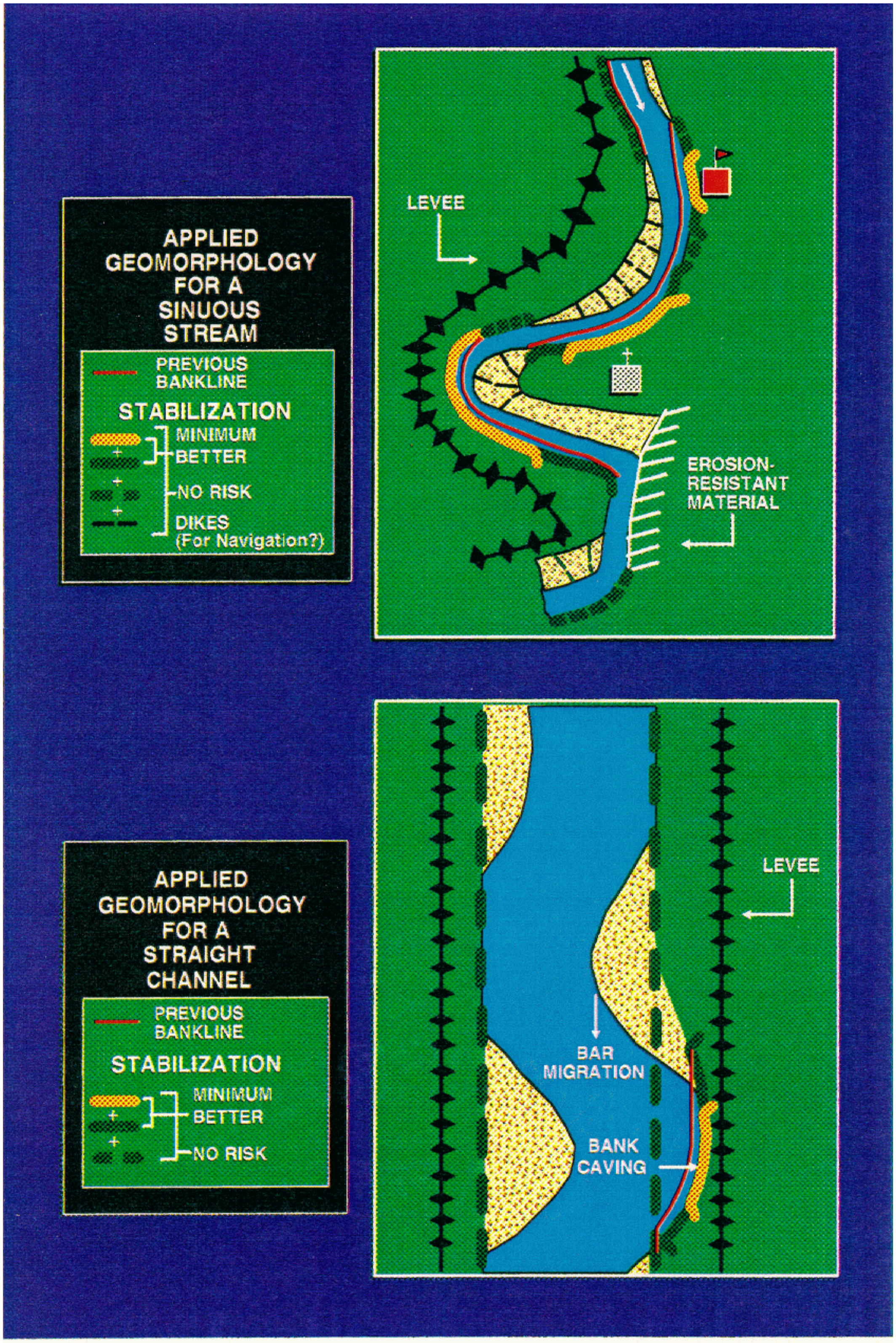



Figure 6.3 Upstream and Downstream Limits of Bank Protection for a Sinuous and Straight Channel

### 6.1.1.3 Other Considerations

Application of the preceding discussion to the selection of upstream and downstream limits of stabilization work should consider the following concepts:

(1) The *downstream limit* is usually more critical than the upstream limit, since the scour pools associated with the normal pool-bar pattern tend to move downstream, and bank failure is often associated with these pools. Although beginning the upstream end of work at the precise point where erosion presently begins carries some risk that erosion will later occur upstream of that point, cost savings may make that risk acceptable, because often a bar will migrate downstream, changing the area of erosion at that point into one of deposition. Conversely, placing the downstream end of the work at the present limit of erosion carries a high risk that the work will be outflanked by subsequent erosion downstream of that point.

(2) Although the preceding concept applies to most situations, a caution regarding the *upstream limit* should be observed: if a serious mistake is made in assessing the future migration pattern of the stream, and the stabilization work is outflanked at the upstream end, then subsequent deterioration of the integrity of the stabilization project may occur rapidly.

(3) *Model tests* under the Section 32 Demonstration Erosion Control Program (U.S. Army Corps of Engineers, 1981) indicated that erosion protection in a bend should be extended downstream from the point of tangency a distance of at least 1.5 times the width of the approach channel into the bend. This can be used for general guidance if data on channel behavior at a specific location is unavailable or unreliable. Studies by Parsons (1960) provide similar insight.

(4) The *transition* from the stabilized bank to the natural bankline can be made rather simply at the *upstream limit*. The details depend on the type of protection being used, but in general a slight increase in the strength and/or a shallow "key-in" will be sufficient for armor-type protection. Indirect protection can simply be turned, or "feathered," into the bank, with a slight recess being good insurance.

(5) In contrast, more elaborate precautions are advisable for the *transition* to the natural bankline at the *downstream limit*. The most important precaution is to insure that the work is not stopped prematurely, as discussed above. Beyond that, it is advisable to "key-in" and/or increase the strength of armor revetment. For indirect bank protection works, a pronounced "tuck-in" should be provided, perhaps with a liberal application of stone if conditions are severe. The alternative is to be prepared to reinforce the work at some time in the future if scour at the downstream limit threatens to outflank the protection. Such an approach may be sound if project authority, rate of erosion, and the potential consequences of miscalculation allow it.

(6) Analysis of the present conditions and prediction of future channel migration should take into account the *magnitude of flows* that occurred prior to the site inspection, or that occurred during the time period spanned by surveys or aerial photos that are being used for analysis. High flows tend to attack the bank further downstream, and low flows farther upstream, because meander wave length is directly proportional to discharge. The stream integrates the total hydrograph over the long-term, but short-term observations may be distorted by extreme events. Therefore, if the period of observation is weighted toward low flows, the long-term attack may be farther downstream and more severe than current observation would indicate. High flows during the period of observation may have the opposite effect.

### 6.1.1.4 Special Considerations for Braided Streams

Protection works on braided streams may need to extend upstream and downstream from the active erosion, because bars, chutes, and pools move more rapidly and much less predictably than in meandering streams. Often bank erosion is associated with meandering tendencies of major anabranches, and the likely pattern of attack can be fairly well predicted using the rules of downvalley and lateral migration for alluvial bends. However, in other cases bank erosion may be caused by less predictable impinging flow in side channels. The most efficient approach on a braided stream where bank erosion is associated with anabranch flow which impinges against the bank at sharp angles may be one of the following:

> Treat problem areas as they arise by constructing spot stabilization as the stream attacks first one spot, then another.
>
> Temporarily divert impinging flows, either by excavating a new channel, building temporary dikes of streambed material, or using floating "breakwaters" to absorb the brunt of the impinging flow. However, environmental aspects of construction activity in the streambed may not be acceptable.

If long-term stabilization is required on a braided stream, and project constraints preclude installing bank protection periodically as the channel changes, an adequate solution may require the construction of continuous protection on both sides of its "braid belt." Trenchfill or windrow revetment is well-suited for this application.

### 6.1.2 CHANNEL ALIGNMENT

The preferred choice for the alignment of a stabilized channel is straight-forward. However, exceptions to the preferred choice are rather common. Following is a discussion of the preferred choice and some exceptions.

#### 6.1.2.1 Preferred Choice

The preferred alignment in most cases is to accept the existing general channel alignment, because significantly changing the alignment makes it more difficult to predict the ultimate equilibrium planform and channel geometry. This uncertainty carries risk not only for the success of the work, but also for assessing the potential for detrimental effects caused by the work.

Relocation of a bank which is to be armored or vegetated requires costly and time-consuming excavation and/or filling. Also, the environmental effects of removing and disposing of large amounts of bank material may be unacceptable. The work would be vulnerable to damage from high flows during construction, causing both contractual and engineering difficulties. Banks constructed totally of fill material would be highly susceptible to settlement and scour even after being armored, unless the fill is well-compacted during construction and a well-designed filter is provided. Both of these measures would add substantial cost and difficulty to the project. Also, vegetation may not provide adequate protection for banks newly constructed of fill material, which would further limit the potential for a cost-effective and environmentally acceptable design.

Indirect protection methods (Chapter 8) can more easily be used to modify the existing alignment, but the same basic principle applies - the existing stream, especially if it is in a condition of dynamic equilibrium and has developed a stable alignment, so changing that alignment may generate system-wide instability and should be approached with caution.

#### 6.1.2.2 Possible Exceptions

Exceptions to the preferred choice of accepting the natural alignment are sometimes justified in situations other than that of limited foreshore. Three potential exceptions are:

(1) At very sharp bends;
(2) Highly irregular banklines; and
(3) Straight reaches with unstable planforms.

Channel realignment in these situations is more likely to be required on projects with navigation aspects than on projects with only bank stability aspects.

Most alluvial rivers have a range of values of radius of curvature, meander wave length, or in straight reaches, pool and bar spacing, within which the planform is dynamically stable. Therefore, any channel realignment project undertaken to alter one of the situations listed above should avoid extreme values of those variables. Determining the preferred range of values for a particular stream will be especially difficult if the stream has recently aggraded or degraded, and the planform is still adjusting, especially if the threshold between meandering and braided has been crossed.

(1) Specific considerations regarding channel realignment in *very sharp bends* are as follows:

Realignment of the channel in a very sharp bend may be justified in order to prevent a major channel avulsion in the future. Such an avulsion, in the form of a natural cutoff or development of a major chute channel through the point bar of the bend, might cause serious bank stability problems in downstream reaches.

The radius of curvature of the bend can be increased, that is, the bend made flatter, either by using indirect protection, or by making a well-planned cutoff. An increase in bend radius may result in a decrease in maximum channel depth in the bend, thus improving bank stability.

Use of an indirect protection method in this situation will ideally result in the maximum depth of scour occurring farther away from the toe of the bank than under natural conditions, thus improving bank stability with respect to mass failure.

The disadvantages of using an indirect protection technique in this situation are that (a) much of the construction has to be done in the deepest part of the existing channel, thus increasing the cost and difficulty of construction; and (b) it may be necessary to excavate the opposite point bar to relieve the initial constriction caused by the stabilization structures. Otherwise, local velocities, and perhaps even backwater effects in extreme cases, may be unacceptably high in the interim period that it takes the channel to adjust to the work.

Realigning the channel by constructing a cutoff across the neck of the bend amounts to the channel relocation approach discussed in Chapter 4. Erosion in the bend can be eliminated by a cutoff, but erosion will continue elsewhere if channel migration is characteristic of the stream. Therefore, even a cutoff may need to be accompanied by bank protection, and the prediction of subsequent long-term channel behavior is more uncertain in the presence of a cutoff.

Detailed considerations for designing a cutoff are presented by Petersen (1986).

(2) If the natural bankline alignment is *highly irregular*, and it is determined that a more uniform flow and planform would best accomplish project purposes, then the bankline can be smoothed either by (a) placing indirect protection on a smooth alignment riverward of the natural bankline, as discussed above for a

sharp bend; or by (b) placing trenchfill or windrow revetment landward of the protruding bankline points, and allowing the stream to erode the irregularities away. This approach offers simple design and construction, since the operation is removed from the active channel. However, the bank retreat which occurs before the channel reaches the stabilization structure may not be greeted enthusiastically by property owners. Conversely, if erosion is slower than anticipated, a navigation project with a schedule to meet may require costly dredging of the uneroded foreshore.

(3) In a *straight reach with an unstable planform*, or on a braided stream, it may be desirable to increase the sinuosity of the main channel in order to stabilize the location of scour pools and bars. This can also provide better channel alignment and a deeper channel for navigation. However, an accurate assessment of channel migration tendencies, the stable range of values for pool and bar spacing, and the ratio of radius to width, is especially critical in this situation. Such a realignment often involves both armor revetments and indirect protection in combination, depending upon the bank and channel topography along the proposed realigned bankline.

## 6.2 HYDRAULICS

Having used applied fluvial geomorphology to decide on the location of bank protection work, the next step is to apply a fluvial hydraulics analysis to decide how deep, how high, and how strong to make the work. The concepts of fluvial hydraulics presented in Chapters 2 and 3 apply to the following factors:

Design discharge;
Tractive force and permissible velocity;
Secondary currents;
Variations in river stage;
Top elevation of protection;
Wave, vessel, and ice forces; and
Prediction of toe scour.

### 6.2.1 DESIGN DISCHARGE

It is important to recognize the distinction between *design discharge* and *dominant discharge*. Design discharge usually refers to an extreme event, and is often used in connection with flood control channel analyses. A tractive force or velocity associated with the design discharge is also commonly used to compute stone size for riprap armor (see 6.2.2), and a similar approach can be used to design many commercially available armor materials. The design discharge can be defined quite precisely using hydrologic analyses.

In contrast, the discharge which governs the long-term behavior of a stream, and thus many aspects of the design of bank stabilization work, is exceeded rather frequently, but there is no consensus on how to best define it. It is variously called the “dominant” or “channel-forming” or “effective” discharge, but is in fact an abstract quantity because in nature no single steady discharge will reproduce the morphological and sedimentary features which are formed by the varying discharge of a natural stream. However, it is often considered to be about equal to bankfull discharge on streams that are neither aggrading or degrading.

In practical terms, the design discharge is the flow which would stress bank stabilization work most severely over a short period of time. It is desirable to quantify it and to use it to size the armor layer if we are using an armor technique for which criteria exist. However, for many other protection methods, determination of a design discharge will be academic because no criteria exists to apply it.

### 6.2.2 TRACTIVE FORCE AND PERMISSIBLE VELOCITY

Tractive force and permissible velocity are two parameters that are commonly used to quantify stress on the boundaries of a channel, whether the boundaries are formed in sediment or consist of a protective armor. The highest stresses usually occur under the design discharge. Except for riprap and some manufactured products, little precise guidance exists regarding the limiting tractive force for erosion protection materials. For materials for which no precise guidance exists, demonstrated performance under comparable conditions is the best guide.

### 6.2.3 SECONDARY CURRENTS

Flow at channel bends in meandering channels and alongside bars and confluences in braided channels is sharply three-dimensional. Velocities in the plane normal to the axis of primary or longstream flow are termed secondary currents and coherent patterns of these currents, termed secondary cells, can influence the distributions of primary velocity and tractive force that erode the banks.

However, the patterns of secondary cells, especially close to eroding banks, are poorly understood. It is known that plunging flow close to the outerbank in natural meanders often promotes deep toe scour and that the sweeping effect of inward directed secondary currents near the bed promotes point bar growth at the inner bank.

These processes are important to the growth of meanders, but more to the point here, they are fundamental to the mechanisms of bank failure, and influence the effectiveness of many types of bank and channel stabilization work. Unfortunately, secondary currents can usually be addressed in design only indirectly, by letting the stream integrate them into its behavior, along with all the other geomorphic, hydraulic, and geotechnical processes.

### 6.2.4 VARIATIONS IN RIVER STAGE

Although this is, strictly speaking, a hydraulic variable, its primary application to design is geotechnical, since the susceptibility of the bank to mass failure, leaching, and piping, is to some degree a function of the rate of drawdown of the water level. The magnitude and timing of the variation also influences the constructability of different techniques.

### 6.2.5 TOP ELEVATION OF PROTECTION

A major design parameter concerns the determination of the elevation to which erosion protection works should be constructed. The most conservative approach for armor revetments is to set the top elevation at design flowline plus a margin for freeboard. This equates to the top of the levee for leveed channels, or to the top of the riverbank where there are no levees, or where they are well protected by vegetation or by distance from the channel. In many situations, this criteria is too conservative and would result in excessive cost and reduced environmental suitability. Unless erosive velocities are believed to exist at high elevations, and the consequences of even minor erosion are unacceptable, consideration should be given to designing the top elevation of protection at a more frequently occurring flowline. Other factors that should be considered in order to decide on the lowest and least costly, yet effective, elevation are shown in Figure 6.4 and are listed below:

Stage duration;
Severity of overbank flow;
Erodibility of upper bank material;
Type of protection and slope of the bank; and
Consequences of failure.

These factors also influence the top elevation of indirect protection, although the most conservative elevation for indirect protection is normally considered to be the elevation of top of the river bank rather than the elevation of the design flood flowline.

Little quantitative guidance is available for applying these factors. An exception to this occurs in situations where the primary purpose of the work is protection against wave action. In those situations, fairly rigorous procedures have been developed to compute wave height and run-up for use in designing top elevation of protection. This is usually not the critical condition for streambank protection, but the references provided in 6.2.6 provide specific guidance for situations where it is the critical condition. In other cases, merely considering qualitatively the factors listed above and discussed below is an aid to intelligent decision making.

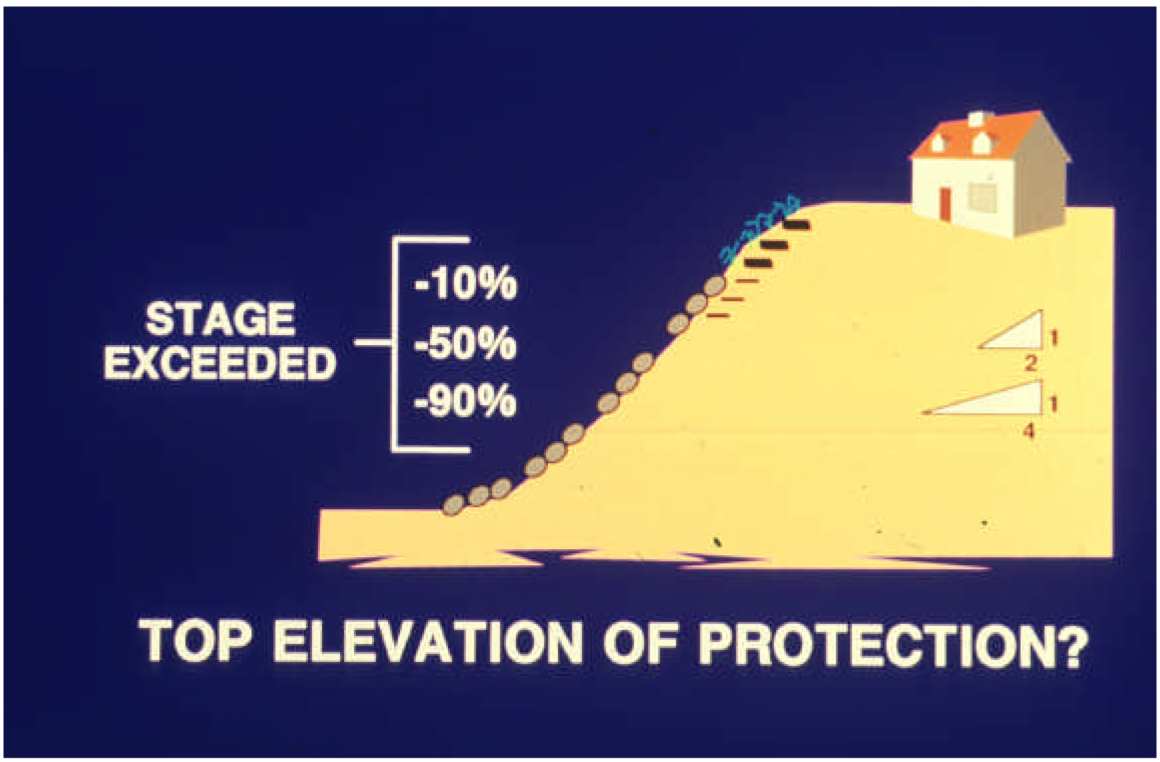



Figure 6.4 Top Elevation of Protection

#### 6.2.5.1 Stage Duration

The period of time that river stages exceed a given elevation is important for two reasons:

> It determines how long the upper bank will be subject to potentially erosive current.
>
> It determines the lowest elevation at which riparian vegetative growth will not be impeded by inundation. This is a factor in determining the minimum top elevation for armor revetment when it is used in combination with vegetation for upper bank protection.

Unfortunately, there is no published guidance that relates the top elevation of protection works with stage duration. However, an example of terminating armor protection below the elevation of the design flood flowline is provided by riprap upper bank protection on some portions of the Lower Mississippi River. It is routinely terminated at a flowline elevation exceeded 5 to 10 percent of the time. On small streams in north Mississippi, the top of the rock in longitudinal stone toe protection (see section 7.1.4) routinely corresponds to a stage that equaled or exceeded about 1 to 5 percent of the time.

#### 6.2.5.2 Severity of Overbank Flow During Floods

The velocity of flow at the interface of the protection and the unprotected bank during floods is influenced by channel alignment, local variations in the elevation of the bank, and the extent of vegetative cover on the upper bank and overbank. Local bankline irregularities which cause a convergence of streamlines, accompanied by higher velocities, can also play a role. The most severe conditions occur at the necks of sharp bends with relatively low bank elevations and little overbank vegetation. Also, the downstream half of sharp bends is where highest velocities against the bank and in the overbank usually exist. Therefore, a higher elevation of protection may be prudent there.

#### 6.2.5.3 Erodibility of Upper Bank Material

This factor is best evaluated by on-site observation. The rate of historic bankline recession alone is not a totally reliable indicator of erodibility of the material in the zone being considered for the top elevation of protection. The material in the upper bank may be very erosion resistant, but still fail from toe scour and subsequent mass failure. General guidance on the erodibility of different bank materials is available, and can be used if experience with particular site conditions is lacking. The most erosive soils are fine sands and silty sands, and the least erosive are clay and coarse gravels.

Erodibility of the upper bank material is also an important factor in selecting the top elevation of indirect protection structures. If the upper bank is highly erodible, then the structures should be high enough to reduce near-bank velocities during most flows. If the upper bank is more erosion-resistant, then the structure needs to be only high enough to induce deposition in front of the bank, especially if the duration of high flows is short, the stream carries a large suspended sediment load, and/or if vegetation can be expected to colonize the area of induced deposition.

#### 6.2.5.4 Type of Protection and Slope of the Bank

These two factors act together to affect the near-bank velocity at the top of armor protection. The rougher the armor and the flatter the bank slope, the lower the velocity at the top of the armor will be, and the lower down the bank the armor can be terminated. Also, flatter slopes are more conducive to vegetative growth, thus upper bank erosion in the form of “shelving” behind the armor will be more likely to be arrested by subsequent volunteer vegetative growth, if climate and soils are favorable.

Since rigid armor cannot adjust well to local scour, it should normally be carried to a higher elevation than adjustable or flexible armor. The alternative to carrying it to a higher elevation is to use adjustable or flexible armor above the rigid armor as a means of transitioning to a non-erosive elevation or to the elevation where vegetative protection will begin.

To some degree, the type of protection determines the amount of design effort that can reasonably be expended to optimize the top elevation, and the potential savings from doing so. If the protection material is relatively inexpensive, the potential savings from a lower elevation will be relatively small, and taking a considered risk in terminating it at a lower elevation will be less attractive.

#### 6.2.5.5 Consequences of Failure

This may well be the dominant factor in setting the top elevation of your protection work, especially if the other factors are poorly defined. Since this is an integral component in determining the overall “safety factor” against failure of the work, it is discussed in 6.6.

### 6.2.6 WAVE, VESSEL, AND ICE FORCES

These forces are seldom the dominant cause of bank failure on most streams. Therefore, they do not usually present the critical design case. However, in some situations the erosion induced by these forces, and the effects of these forces on bank protection works, can be highly visible and significant. There are several references that address design of protective works in detail, and numerous more specialized papers and publications. Two general references that provide an excellent introduction to the topic are U.S. Army Corps of Engineers (1984) and Hemphill and Bramley (1989).

### 6.2.7 PREDICTION OF TOE SCOUR

Degradation is a long-term, large scale process, but toe scour is usually associated with the impacts of high flows over short reaches of channel.

This chapter deals with situations where the design of bank stabilization proceeds under the assumption that the channel will not suffer significant future degradation. Similarly, the various methods for predicting toe scour deal with local scour as a separate process from degradation.

The general approaches to predicting toe scour are:

Analytical, using one or more of the relationships that have been proposed by various researchers;

Empirical, using experience from similar situations; and

Modelling (numerical and/or physical).

Illustrating the uncertainty of analytical approaches, Copeland (1983) cites seven different equations proposed by as many researchers for the specialized case of predicting scour at spur dikes. There is disagreement even as to the significant factors involved, and certainly disagreement in the results.

As a result of the limitations of analytical approaches, an empirical approach is often considered to be more reliable. However, adequately documented experience may be unavailable in particular situations. In such cases, the designer must apply one or more of the analytical or modelling approaches.

The limitations to modelling that were discussed with respect to channel migration also apply to the prediction of toe scour, although not to as great a degree. Two-dimensional numerical models are required, and three-dimensional numerical models would be preferable if available. Physical models must be large scale.

The degree to which these three techniques can be applied is a matter of judgement, and will depend on time and funds available for the analysis, the scope of the project, the consequences of an inaccurate estimate of toe scour, and upon the personal and institutional experience which can be brought to bear on any specific problem.

## 6.3 TOE PROTECTION

Toe protection is essential to the success of bank protection work, although it may not be a massive element of the work if the exposure to scour is relatively mild. Once a prediction of the amount of toe scour to be expected has been made, a variety of methods are available to accommodate it in the design. In this section, general guidance for all types of protection is provided.

### 6.3.1 BASIC OPTIONS

There are two basic options. First, to *"dig it in"* by extending the toe of the protective works into an excavation at or below the predicted scour depth, or at the elevation of a non-erodible material, if such material is present within the practical limits of excavation; or second, to *"let it self-launch"* by designing the work so that, as scour occurs, the protective material can launch or flex downward sufficiently to prevent the scour from moving inshore and causing geotechnical instability of the bank.

The "dig it in" approach is most often used with an armor revetment. Its primary disadvantage is that excavation in a flowing stream, and precise placement of an armor material in the excavation, is often difficult and costly, and sometimes impossible. Ten feet is sometimes used as a rule of thumb for the limit of conventional excavation techniques underwater. Beyond that depth, either dredging or dewatering with a cofferdam may be required for excavation and armor placement.

Sheet-pile retaining walls and pile-supported indirect protection structures which are designed to withstand maximum scour can be considered special cases of the "dig it in" approach.

The "self-launching" approach offers economy and ease of construction by allowing the stream rather than the contractor to perform the excavation. However, it does require a larger volume of material in the toe section than if the toe is placed in an excavation, since the launching process may be irregular. As a result, the cost of material may in some cases negate savings in operational cost. Therefore, if site conditions permit easy mechanical excavation to the predicted scour depth, the "dig it in" approach may be the least costly overall.

The "self-launching" technique also offers the considerable advantage of providing a built-in scour gauge, particularly if the top of the launching section is visible above water. If it is underwater even at low stages, it can be surveyed by accurately located soundings. If it appears that the toe section is launching more than expected, it can be reinforced simply by placing additional material at the riverward edge of the remaining section.

With either approach, stone is often chosen for the toe protection material, even if another technique is selected for the remainder of the armor or structure, because stone toe protection can be precisely and confidently designed for almost any application.

## 6.3.2 SPECIFIC GUIDANCE FOR VARIOUS TECHNIQUES

Application of the basic approaches to specific types of work is discussed in the following paragraphs.

### 6.3.2.1 Stone Armor

Guidance can be found in Section 7.1.1.

### 6.3.2.2 Other Self-adjusting Armor

Either dig it in, or use the self-launching technique. The required toe volume for self-launching can be computed the same way as for stone armor. If the self-launching approach is to be used where predicted scour is more than a few feet, then stone is recommended for the toe material.

### 6.3.2.3 Rigid Armor

Rigid armor should either extend to the predicted scour depth, be combined with a self-launching technique with required toe volume computed the same way as for stone armor, or incorporate a flexible mattress at the toe.

### 6.3.2.4 Flexible Mattress

Similarly, a flexible mattress can be installed to the predicted scour depth, or extended riverward of the toe of the bank by a horizontal distance at least twice the predicted depth of scour. Where appropriate, these guidelines may be superseded by the manufacturer's recommendations. If more than a few feet of scour is predicted, then the use of a self-launching stone toe should be considered, particularly if it will not be feasible to monitor toe scour frequently, or to reinforce the toe if required.

### 6.3.2.5 Dikes

The provision of toe protection for dikes is a more complex problem than that for armor revetments. The complexity arises when trying to distinguish among three cases:

(1) General toe scour which immediately endangers the overall stability of the bank and threatens to flank or fail the dike system.

(2) Local scour which can threaten the integrity of part of a dike, and which ultimately may fail local portions of the bank.

(3) Local scour which may cause minor damage to a dike or minor bank instability, but which can be accepted.

In practice, it is difficult to separate these three processes. Conceptually, however, case (1) must be prevented, and case (2) must be addressed if the consequences of it occurring are high. Acceptance of case (3) is inherent in the choice of dikes as the method of erosion control.

The alternative toe protection treatments, which can be used separately or in combination, are to:

Extend the dikes into the channel to move general scour far enough away from the bank to prevent major geotechnical instability.

Provide separate protection at the toe of the bank with an adjustable armor or flexible mattress. With this approach, the dikes will limit the velocity and associated general scour near the bank, theoretically allowing a less substantial toe protection than without dikes. This approach may not be cost effective for preventing general scour, since the effect of dikes on general scour cannot be reliably predicted, requiring a more conservative design for the separate toe protection than is theoretically necessary. However, it is often used to protect against local scour induced by the dike itself.

Provide separate protection riverward of the bank toe, perhaps along a line connecting the ends of the dikes. This is a conservative, but costly approach, which may negate the cost advantage that dikes might otherwise provide. In the extreme case, this approach would more properly be termed a type of retard. In this case the dikes would simply serve as tiebacks, and would be the secondary component of the work.

Some **permeable dike** designs, such as tire-posts and “Palisades” (a commercial product) allow components of the structure itself to be displaced downward, maintaining contact with the bed as scour occurs. With these designs, the same cautions that are stated below for flexible retards are applicable.

If **impermeable dikes** are constructed of a material such as stone, which will launch into a developing scour hole, the size of the scour hole will tend to be self-limiting. However, impermeable dikes are often less effective in inducing deposition than permeable dikes, and are likely to produce more concentrated flows and higher velocities locally, which tends to offset the positive effect of self-launching.

#### 6.3.2.6 Retards

For **rigid retards**, such as non-adjustable fencing or piling, some designers assume that the impedance to flow provided by the structure, and subsequent landward deposition of sediment, will prevent toe scour from endangering bank stability. Thus they make no specific provision for limiting toe scour. This approach is sometimes successful, but it is not recommended unless the following conditions are met:

Pile penetration and size are designed for the condition of maximum predicted scour;

The structure will be monitored frequently and reinforced if necessary;

Suspended sediment load of the stream is large, so that deposition behind the structure is likely to occur; and

The distance from the toe of the bank to the structure is at least twice the predicted scour depth.

One of the following approaches may be more efficient and effective:

Locate the retards in a trench excavated to the predicted scour depth;

Install a self-launching toe section of stone or other material; and

Secure a flexible mattress to the riverward side of the retards

**Flexible retard** designs, such as jacks, tire-posts, and trees, allow part; or all of the retard structure itself to displace downward as scour occurs. Use of this technique requires secure connections between retard units and between the retard structure and the landward anchors, or extra penetration of piling, depending on the specific design. Allowance must be made in selecting retard height so that the downward displacement will not leave the upper bank exposed to significant erosion during high flows. Otherwise, it may be necessary to maintain the work by placing additional units on top as the originals units displace downward.

#### 6.3.2.7 Other Flow Deflectors

**Iowa vanes and bendway weirs** are similar to dikes and retards in that they function by inducing deposition at the bank toe rather than permitting scour to occur. Since they significantly alter secondary currents, rather than simply relocating the secondary currents, they should be less demanding of toe protection than dikes or retards. However, since these are relatively new techniques, long-term field experience is not yet available.

#### 6.3.2.8 Vegetative Bank Protection

The importance of toe protection for successful bank stabilization using vegetation cannot be overemphasized. Vegetation alone is unlikely to be successful as toe protection unless velocities during design flows are so low that little toe scour is predicted, and climate, inundation conditions, and soils are conducive to a vigorous growth at the toe.

Selection of a toe protection technique should assume that the vegetated portion of the bank is in effect a rigid armor, which dictates that either a self-launching material or a flexible mattress be used at the toe. In practice, vegetation is usually used as a cost-saving or environmental feature in conjunction with a structural technique, and appropriate toe protection will be an integral part of the design of the structural technique. Typical examples are vegetative plantings between dikes, behind retards, and on the upper bank slope above one of the many armor materials.

#### 6.3.2.9 Retaining Wall

If a retaining wall is part of the solution to geotechnical instability, then the approach to toe protection should be the same as for rigid retards. The alternative of designing the wall to be stable under maximum scour is likely to be more costly than limiting the scour. It also introduces the risk of a sudden, and, perhaps, catastrophic mass failure in the event of miscalculation of the maximum scour depth, since underdesign of toe protection is more likely to manifest itself gradually and is more easily detected in time for remedy than is excess scour during high flows in the absence of toe protection. Since retaining walls are often used in situations where consequences of failure are high, increasing the safety factor by using toe protection as well as extra structural strength is advisable.

## 6.4 SURFACE DRAINAGE

Inadequate provision for surface drainage seldom results in complete failure of the work, but it should not be neglected as it can be a major concern to adjacent property owners. Inadequate design with respect to surface erosion gives the appearance of incompetent design, affecting public perception of the success of the work. Mistakes occur easily, because the designer's primary focus is usually on overall channel stabilization, and proper design for overbank drainage flow outlets can be a tedious process, especially if rigorous design procedures are to be followed.

Attention to surface drainage is even more important if the stream is degrading, and flowline lowering is anticipated, since rills, gullies, and channels draining from the floodplain will similarly degrade if not adequately protected.

The amount of design effort which is appropriate is determined by the:

project purpose;

susceptibility of a site to surface erosion, which depends on topography, rainfall, vegetation, soil characteristics, and the type of bank stabilization to be used;

engineering, environmental, and political consequences of erosion; and

feasibility of collecting sufficient data to permit a rigorous design.

The potential for surface erosion is best determined by identification and observation of pre-existing problems. However, the construction of bank stabilization work can make the problem worse, as well as more noticeable, since gullies leading into the stream will no longer be periodically destroyed by streambank caving. Freshly graded banks are particularly susceptible to surface erosion, and natural levees and existing drainage patterns and vegetation may be disturbed by construction operations.

The basic steps in preventing erosion from surface drainage are to:

Protect all bare ground;
Collect the overland flow; and
Provide controlled outlets into the stream.

In the simplest situations, surface drainage occurs by sheet flow which is directed away from the stream into a natural interior drainage channel. In this case, protection of bare ground on unarmored bank slopes and in areas disturbed by construction activities is all that is necessary. This is usually provided by vegetative treatments. Various types of chemical soil stabilizers are also available and are often effective.

If topography is such that significant amounts of surface drainage enter the channel in the vicinity of the work, it is necessary to collect the overland flow. This can often be accomplished by small unlined ditches if drainage areas are small, slopes are flat, and the soil is erosion resistant. The ditches should usually include vegetative treatment or soil-stabilization. If grading of the bank is part of the stabilization work, the natural levees should be rebuilt using material from bank grading or ditch construction, as shown in Figure 6.5.

The provision of controlled outlets into the stream is sometimes a simple matter of leaving a natural outlet undisturbed, if the flow carried by the outlet is not increased by alterations to the topography during construction. Otherwise, or if the natural outlet shows signs of instability, a lined outlet or culvert should be provided. Steep drops can be accommodated by a drop culvert, or by providing energy dissipators at the ends of lined outlets or culverts. The detailed design will be site-specific. Schwab et al. (1981) treat the subject thoroughly, and Schiechtl (1980) provides information on successful techniques. More specific guidance based on site conditions can usually be obtained locally.

Rigid armor is more susceptible than most armors to undermining by surface drainage or destabilization by excess hydrostatic pressures due to the trapping of sub-surface water behind the armor. Therefore, special care should be taken in collecting surface water and providing outlets into the stream. “Keying in” the top of the armor, or providing a “collar” of adjustable armor, is a common practice.

When indirect bank protection methods are used, surface drainage is often not a consideration, since the work usually does not significantly alter existing drainage conditions. A reduction in erosion from surface drainage may be an incidental benefit of the work if deposition behind the bank protection structure raises the base level of existing outlets. This may, in fact, present a problem if deposition is high enough to block local drainage outlets. Usually, however, the only drainage treatment necessary when using indirect protection is to treat areas disturbed during construction. Treatment for local surface erosion can be designed separately if it is a significant problem to be addressed under the project.

## 6.5 MANUFACTURERS' RECOMMENDATIONS

Manufacturers and distributors of the various patented or commercially available erosion protection products may not be completely objective, since they have a vested interest in their product. However, they also want their product to perform well, and their experience with it is likely to be extensive. While their design methods should not be accepted uncritically, when they are supported by a service record under comparable conditions they may obviate the need for a duplication of effort by the designer. However, the procurement policies of some governmental agencies may make it difficult to specify a particular product by brand name. Adding the phrase “or equal” to the specification may

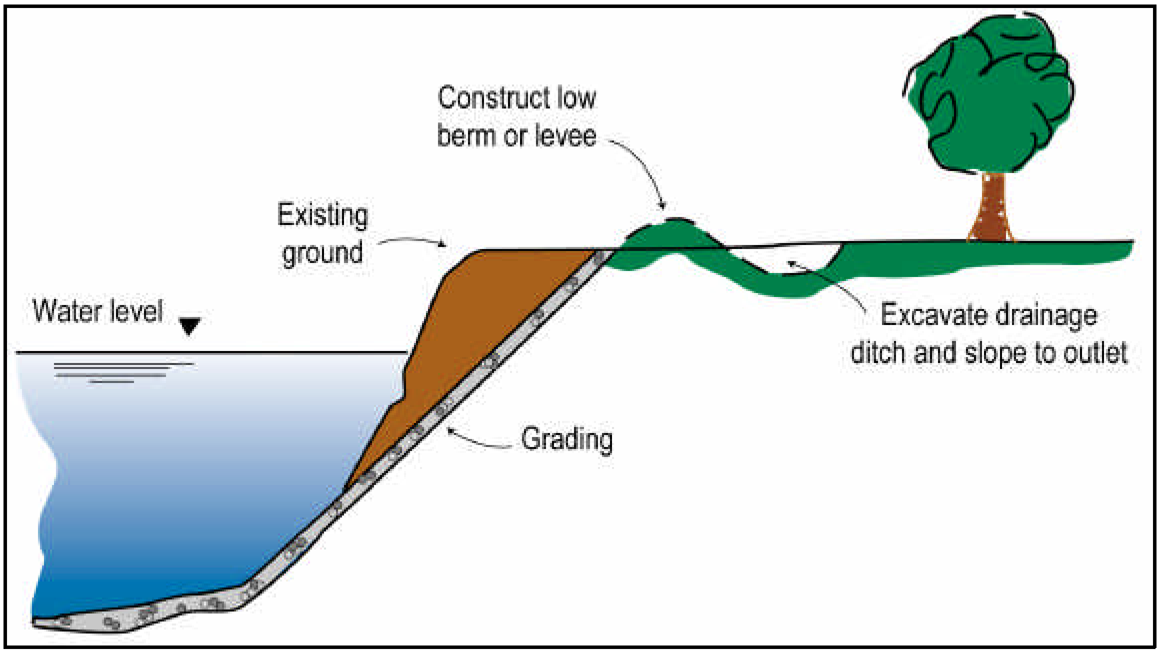



Figure 6.5 Construction of Berm or Levee to Control Overbank Drainage

alleviate that difficulty, but even if a contractor's proposed substitute is considered by engineering personnel to not be "equal," documenting that to the satisfaction of administrative personnel may be difficult.

## 6.6 SAFETY FACTOR

Most engineering analyses for the design of structures such as buildings and bridges provide for a "safety factor," even when the physical laws governing the behavior of the structure are well-defined and readily quantifiable. Because rigorous design procedures are lacking for many aspects of streambank protection, the need for a safety factor is even more apparent. However, there is a significant difference in the analogy, in that failure of buildings and bridges invariably carries the risk of loss of life, whereas that is often not the case with streambank protection works. Also, the safety factor for buildings and bridges is usually governed by statutes and by codes of practice, which is also not usually the case for streambank work.

The safety factor is influenced by the:

- Designer's level of experience
  - with the protection method being used
  - with the stream itself or comparable streams;
- Difficulty of constructing the work to specifications;
- Sponsor's capabilities
  - to perform routine maintenance
  - to perform emergency reinforcement; and
- Consequences of failure.

The designer's experience with the protection method and with the stream itself is a measure of the confidence that can be placed in a prediction of the performance of the work and in a prediction of the consequences of failure of the work. The reliability of available data on the stream also affects the level of confidence which can be placed in the designer's assessment of the causes of bank instability.

The difficulty of constructing the work to specifications affects the possibility that undetected construction flaws will leave vulnerable points. The timing of construction affects the likelihood that bad weather and high flows will extend the work period, making incomplete portions of the work more vulnerable, especially if vegetative treatment is an important component. Also, construction delays which result in changes in the channel may make the design itself unsatisfactory in extreme cases.

The competence of the construction personnel and the capabilities of their equipment are also factors. However, these factors may be unknown at the time the project is being designed.

The sponsor's capability and commitment to perform routine maintenance will determine the probability of minor failures becoming catastrophic. The sponsor's ability to perform emergency reinforcement under difficult conditions will affect the safety factor for work which protects important facilities such as levees. The availability of sources of assistance during emergency conditions is also a factor. Project documentation should emphasize the importance of monitoring and maintenance, if appropriate.

The consequences of failure will probably be the most important single element in determining the safety factor. This hinges on an assessment of the likelihood of loss of life, significant property damage, or severe stream channel instability if the work fails. When faced with design decisions that cannot be resolved analytically, the consequences of failure must be the overriding consideration.

# CHAPTER 7

## SURFACE ARMOR FOR EROSION PROTECTION

In this chapter, descriptive information is generally followed by a discussion of advantages, disadvantages, typical applications, and design considerations as appropriate. In order to minimize redundancy, these topics are discussed at the broadest possible level in the hierarchy of the text; in other words, aspects which are shared by all techniques are discussed at the beginning of the chapter; aspects which are shared by a group of techniques are discussed at the group level; aspects that are peculiar to a smaller category of techniques, or to a single technique, are discussed at the appropriate level of specificity.

The extent of the discussion of specific techniques ranges from the detailed design guidance presented for riprap to a brief description for some specialized techniques. Therefore, a complete understanding of a specific technique requires perusal of all material at a broader level in the text, as well as material peculiar to that technique.

The following paragraphs outline the general description, advantages, disadvantages, typical applications, and design considerations for **most** surface armor used in bank stabilization methods:

> Armor is a protective material in direct contact with the streambank. Armor is often simply called "revetment," but the more specific term "armor" is used here because other forms of bank stabilization, such as retards and retaining walls, are also referred to in some regions as revetments. Armor materials can be categorized as follows:
>
> Stone;
> Other self-adjusting armor;
> Rigid armor; and
> Flexible mattress.
>
> Advantages are: Armoring the surface of the bank is a proven approach which can be precisely designed for a given situation, and which provides immediate and effective protection against erosion. Also, existing or potential problems from erosion by overbank drainage can be effectively addressed integrally with the design of the streambank armor work.

Disadvantages are: Preparation of the bank slope is usually required, either for geotechnical stability or to provide a smooth surface for proper placement of the armor. This may result in high cost, environmental damage, and disturbance to adjacent structures. The extent of earthwork associated with an armor revetment will be especially significant if the existing channel alignment is to be modified either by excavation or by placing fill material in the channel.

Effective subaqueous placement of armor material in deep water or when current velocities are high is often difficult and costly.

Some armor materials may require special measures to mitigate undesirable aesthetic and biological characteristics.

Design considerations are: Armor must have sufficient weight and/or strength to remain in place when subjected to hydraulic forces and impact from objects carried by the stream. It must also prevent significant loss of bank material from beneath it due to turbulence of flow or movement of groundwater.

All armor protection requires careful consideration of the geotechnical stability of the bank, and sometimes a granular or fabric underlayment is required for proper interior drainage of the bank material, or to prevent loss of fine grained material through the armor.

## 7.1 STONE ARMOR

The following paragraphs outline the general description, advantages, and disadvantages for **most stone armors** used as a bank stabilization method:

Stone armor can be placed in four general configurations, the most common being a “riprap blanket.” Other forms, known as “trenchfill,” “*longitudinal stone toe*,” and “windrow” (referred to in some regions as “falling apron”), can be very useful in certain situations.

A stone armor usually consists of “graded” stone, which is a mixture of a wide range of stone sizes; the largest sizes resist hydraulic forces, and the smaller sizes add interlocking support and prevent loss of bank material through gaps between the larger stones. Hand-placed stone in a smaller range of sizes is occasionally used.

Advantages are: Because its performance has been so thoroughly analyzed by research and practical application in a wide range of conditions, stone armor can be designed with an especially high degree of precision and confidence. The American Society of Civil Engineers' Task Committee on

Channel Stabilization Works stated in 1965 that "Stone is the most commonly used material for upper bank paving for revetment works, and in most cases has proved superior to other materials because of durability and ability to conform to minor irregularities in the slope" (ASCE, 1965). Since that time, further development and application of manufactured proprietary armor materials, and increasing emphasis on environmental considerations and the use of vegetation for erosion control, has tempered that observation to some degree. However, the favorable attributes of stone armor are not diminished by the increasing availability of alternative materials. Furthermore, well-graded stone can often be placed without a separate underlayment material, because it provides permeability without exposing bank material. This characteristic may be a crucial factor when comparing the economics of alternative armor materials.

Disadvantages are: Stone may be more costly than other materials, depending on its availability. It requires heavy equipment for efficient placement on large projects. It may be considered unaesthetic for some locations, and may not compare favorably with other materials in some environmental circumstances.

### 7.1.1 RIPRAP BLANKET

Detailed discussion of and design guidance for this most common form of stone armor is provided in Appendix A. Environmental considerations pertinent to the use of riprap armor are discussed in 5.2.2.

### 7.1.2 TRENCHFILL

#### 7.1.2.1 Description

A trenchfill revetment, shown in Figures 7.1 and 7.2, is simply a standard stone armor revetment with a massive stone toe. It is normally constructed in an excavated trench behind the river bank, in anticipation that the river will complete the work by eroding to the revetment, causing the stone toe to launch down and armor the subaqueous bank slope.

Material other than stone, such as broken soil-cement, has been used successfully and may be less costly than stone, but careful design of the soil/cement mixture, and careful monitoring of the material mixing, breaking, and placing operation is required.

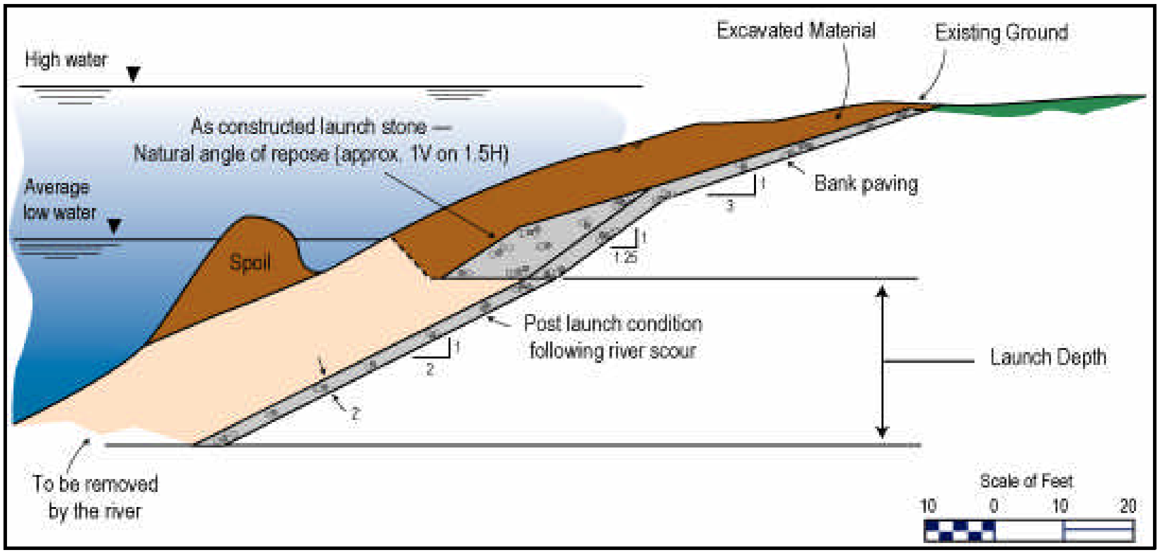



Figure 7.1 Typical Cross Section of a Trenchfill Revetment

Figure 7.2 Aerial View of Trenchfill Revetment With Foreshore Material Still in Place

#### 7.1.2.2 Advantages

A trenchfill revetment allows stabilization along a predetermined alignment, and is often simpler to design and construct than a revetment placed on the active stream bank.

#### 7.1.2.3 Disadvantages

Trenchfill allows erosion to continue unabated until the stream reaches it, and its construction requires heavy equipment. Large areas of rights-of-way are usually required.

#### 7.1.2.4 Typical Application

Trenchfill's most powerful use is in the following circumstances:

Where a smooth alignment of the stabilized channel is required (usually to meet navigation criteria); and

Where rapid erosion rates, high velocities, large depths of flow, or rapid fluctuations in river stages make construction within the stream channel very difficult.

Trenchfill has been extremely useful where these conditions exist on the Arkansas, Red, Missouri, and Mississippi Rivers. The key to successful performance is a relatively uniform rate of launching at any given point, although uniformity of bank recession along its length is not a prerequisite to successful performance. Therefore, it is most successful when bank materials are predominantly noncohesive. Otherwise, additional stone may be necessary, either during construction or in later reinforcement operations, to compensate for inefficient launching where the underlying bank material fails by slab or rotational slips.

#### 7.1.2.5 Design Considerations

Special design considerations are as follows:

The required thickness of the stone armor on the upper bank slope can be computed according to Appendix A or can be based on successful experience under similar conditions.

Stone gradation can likewise either be computed according to the guidance provided in Appendix A, or based on successful experience in similar applications. A gradation which has a significant amount of fine stony material has been shown by experience to be effective in many cases without a filter or underlayment, because the fines fill the voids between the larger stones, while still allowing the armor layer to retain adequate permeability. Such gradations are sometimes called “quarry run” because little sorting is required after the blasting operation in the quarry.

The required volume of stone in the trench can be computed according to guidance provided in Appendix A, after the design depth of toe scour is either computed or estimated based on previous experience.

Design of the trench is a compromise between economics and performance. A higher trench bottom elevation reduces the volume of excavation and is less likely to require expensive dewatering or difficult underwater excavation. Unfortunately, it also requires a greater total volume of stone because allowing the stone to launch is less efficient than placing it to the required thickness on a prepared slope above water. Thus, a higher trench elevation requires a larger volume of stone to protect a given height of bank, because non-uniform launching of the toe stone must be allowed for. The guidance for required quantity of toe stone presented in Appendix A allows for this, but the fact remains that pre-placing stone closer to its final position (that is, to a lower elevation) carries less risk than allowing it to launch, particularly if the bank contains cohesive material which may retreat by mass failure rather than eroding uniformly.

Because placing the stone in the trench to the lowest practicable elevation is desirable, the elevation of the bottom of the trench is sometimes specified to be as much ten feet below the river stage expected during the construction season, based on the assumption that groundwater level in the trench will be about the same as the river stage. Ten feet of underwater excavation is the most that is usually feasible with standard equipment without dewatering. Careful supervision during construction is required, and the underwater trench should be filled with stone in a continuous operation immediately behind the excavation finishing operations.

A useful design refinement is to provide for a variable depth of trench, keyed to the actual river stage during construction. This permits taking maximum advantage of low river levels by lowering the trench so that the stone can be placed at a lower elevation. It also allows the trench bottom to be raised if river levels are unexpectedly high. Construction can then continue in spite of higher stages, without putting the contractor in an untenable position by requiring more underwater excavation or dewatering than was anticipated in the original bid. The specifications should set an upper limit of river stage, above which operations will be suspended. Setting this upper limit is a subjective decision, determined by the urgency of completing the work, the hydrologic characteristics of the river, and the experience of design and construction personnel. The extreme case, if quick completion of the work is mandatory in spite of high river stages, is to allow for substituting a stone windrow revetment, constructed without excavation, in place of the trenchfill.

The design slopes of the trench are established by the most critical geotechnical condition, depending on bank materials. This will usually be the fully launched condition. The configuration of the riverside slope of the trench is governed only by construction considerations, the only requirement being that the trench remain stable long enough for the stone to be placed, without creating a hazardous condition for construction personnel in the interim.

Environmental or land use considerations may limit the area available for the disposal of material excavated from the trench. Within those limitations, excavated material can be placed either riverward or landward of the trench. If it is placed riverward of the trench, it will be eroded away as the river channel migrates toward the revetment, although it should be placed so that it does not cause geotechnical bank failures that might affect the integrity of the stone in the trench before it launches. If it is placed landward of the trench, the geotechnical design of the work should account for its presence, and proper routing of surface drainage should be provided for. Unless it is certain that natural revegetation will occur on the disposal area within a short time after completion of construction, vegetation should be established as part of the construction operation. Since the disposal area will be built up higher than the adjacent ground, habitat diversity can be improved by establishing species of vegetation that are less tolerant of flooding than the existing species.

A portion of the excavated material can be placed so that it becomes an extension of the bank slope, in order to provide a greater degree of control over the direction of flows at river stages which would otherwise overtop the natural bank. This is more likely to be desirable in cases where the channel alongside the revetment will be used by navigation traffic than in cases where prevention of channel migration is the only project purpose. In such cases, the material should be semi-compacted as it is placed, and then protected from erosion as if it were part of the original bank.

### 7.1.3 WINDROW

#### 7.1.3.1 Description

A windrow revetment is simply an extreme variation of a trenchfill revetment. A windrow revetment consists of rock placed on the floodplain surface landward from the existing bankline at a pre-determined location, beyond which additional erosion is to be prevented as shown in Figures 7.3 through 7.6.

#### 7.1.3.2 Advantages

A windrow has the same advantages as a trenchfill, and is even simpler to design and construct.

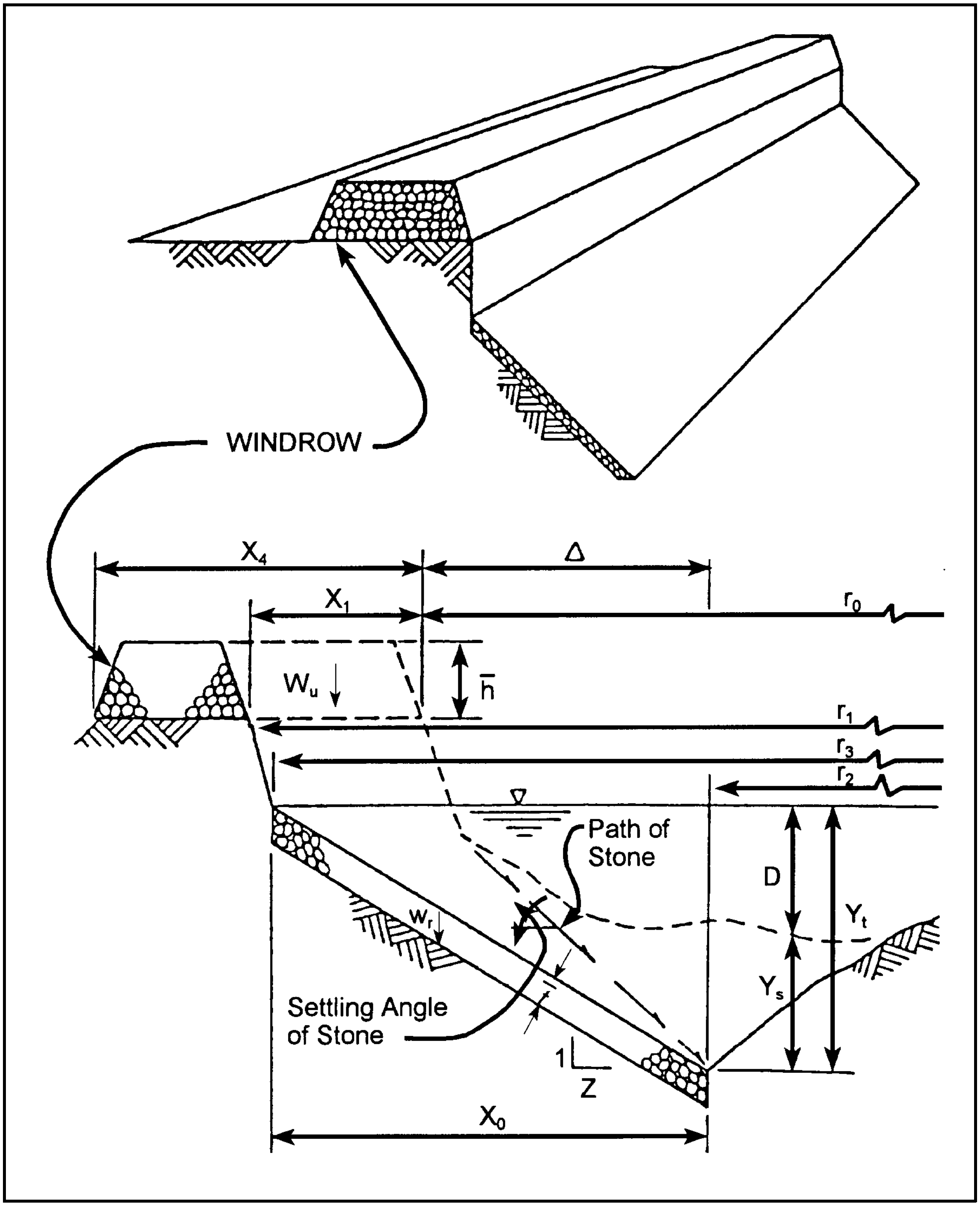



Figure 7.3 Schematic Diagram of Windrow Revetment

Figure 7.4 Conventional Windrow Placed on Top Bank

Figure 7.5 Placement of Windrow Rock in Excavated Trench on Top Bank


Figure 7.6 Launched Windrow Rock

### 7.1.3.3 Disadvantages

A windrow also has the same disadvantages as trenchfill. Also, it is rather wasteful of stone when it is placed on top of the stream bank, because the self-launching process is not as efficient when the stone must launch down the entire bank height rather than only below the bottom of a trench excavated to a lower elevation.

### 7.1.3.4 Typical Applications

Where stone or other suitable windrow material is relatively inexpensive, construction of a windrow behind the existing bank may be cost-effective, if the simplicity of design and construction offset the relatively inefficient use of material. As with trenchfill, the key to efficient performance is a relatively uniform rate of launching at any given point. Therefore, sites with predominantly non-cohesive bank materials are the most suitable.

Windrow may be appropriate for emergency situations, where urgency overrides cost, there is limited time for detailed design, and high river stages and velocities prevent normal construction operations. The site conditions, availability of materials, equipment, and labor, in practice dictate the design, which must be performed concurrently with mobilization of resources and the beginning of construction. The approach is to quickly feed into the stream a resistant material at the critical points, continuing the operation until the crisis passes and a well-designed, permanent solution can be engineered.

### 7.1.3.5 Design Considerations

The design of windrow is approached in the same way as trenchfill, except that no trench design is required. Geotechnical analysis is recommended to determine if the risk of mass bank failure during or after launching is acceptable, although it is impossible to obtain the same degree of geotechnical safety with windrow as with more conventional methods, so that some risk is unavoidable.

Based on laboratory model studies conducted at the U.S. Army Waterways Experiment Station, a rectangular shape for the windrow was found to be the best windrow shape (USACE, 1981). This shape supplies an initial surge of stone which counters the thinning effect of the scour in the toe zone of the forming revetment. The remaining portion of the windrow then provides as ready supply of stone to produce a uniform paving. However, this shape does require the excavation of a trench for placement of the stone. The second best windrow shape was the trapezoidal shape. It has one advantage over the rectangular shape in that no trench is needed to contain the windrow stone. This shape supplies a steady supply of stone similar to the rectangular shape. The triangular shape was the least desirable shape. This shape supplies more stone initially, but the quantity of stone diminishes as the windrow is undercut.

The velocity and characteristics of the stream dictate the size of stone used to form the windrow revetment. The stone must be large enough to resist being transported by the stream. Results obtained from windrow revetments constructed on the Missouri River indicated that small gradation stone (200-pound top size with $D_{50}$ of 7 to 8 inches) was more effective than large gradation stone (500-pound top size with $D_{50}$ of 9 to 10 inches) because the smaller gradation forms a more dense, closely chinked protective blanket layer than the larger gradation. A well graded stone is important to ensure that the revetment does not fail from leaching of the underlying bank material.

### 7.1.4 LONGITUDINAL STONE TOE

#### 7.1.4.1 Description

Longitudinal stone toe is another form of a windrow revetment, with the stone placed along the existing streambed rather than on top bank. The longitudinal stone toe is placed with the crown well below top bank, and either against the eroding bankline or a distance riverward of the high bank. Typical crown elevations may vary but are commonly between 1/3 and 2/3 of the height to top bank.

The success of longitudinal stone toe protection is based on the premise that as the toe of the bank is stabilized, upper bank failure will continue until a stable slope is attained and the bank is stabilized. This stability is usually assisted by the establishment of vegetation along the bank.

#### 7.1.4.2 Advantages

A longitudinal stone toe has the same advantages as a trenchfill and windrow. It also allows for the preservation of much of the existing vegetation on the bank slope, and encourages the growth of additional vegetation as the bank slope stabilizes. An additional advantage is that the treatment is amenable to the planting of additional vegetation behind it.

#### 7.1.4.3 Disadvantages

A longitudinal stone toe also has the same disadvantages as trenchfill. By definition, longitudinal stone toe protection only provides toe protection and does not directly protect mid and upper bank areas. Some erosion of these mid and upper bank areas should be anticipated during long-duration, high energy flows, especially before these areas stabilize and become vegetated.

### 7.1.4.4 Typical Applications

Longitudinal stone toe protection is especially suitable where the upper bank slope is fairly stable (due to vegetation, cohesive material, or relatively low flow velocities), and erosion can be arrested by placing a windrow along the toe of the bank. This avoids the wasted effort of disturbing, then rearmoring, an existing stable slope. Small or ephemeral streams are especially suited to this approach.

The longitudinal stone toe technique may be appropriate where the existing stream channel is to be realigned, although for maximum effectiveness the top elevation of the stone must be high enough that it is not overtopped frequently. In this application, it actually functions as a retard.

### 7.1.4.5 Design Considerations

There are basically two variations of the longitudinal stone toe. These will be referred to as **longitudinal peaked stone toe protection,** and **longitudinal stone fill toe protection.** Design consideration for these two stabilization measures are discussed below.

**Longitudinal Peaked Stone Toe Protection.** An efficient design for a longitudinal stone toe is to simply specify a weight or volume of stone to be placed per unit length of streambank, rather than to specify a given finished elevation and cross-section dimensions. This basically results in a triangular shaped section of stone placed along the toe of the streambank. This type of protection is commonly referred to as a longitudinal peaked stone toe protection (Figures 7.7 and 7.8). A primary attraction of this treatment is its simplicity. Extensive surveys and analysis during design and construction would reduce that attraction. Since the volume of stone required at each section is determined by the estimated scour depth, simply specifying a volume or weight is all that is required. In the small streams of north Mississippi, longitudinal peaked stone toe protection placed at a rate of 1 to 2 tons per linear foot of streambank has proven to be one of the most successful bank stabilization measures used in that area. This generally results in a height of stone between 3 and 5 feet high above the streambed. A "typical" cross-section can be specified on the drawings, along with a relatively smooth alignment to fit site conditions. During construction, the selected alignment for the structure is flagged, and increments of length are measured as appropriate for the size of delivery vehicles or placement buckets. Design, bidding, and supervision of construction is, therefore, greatly simplified.

With longitudinal peaked stone toe protection, the establishment of vegetation landward of the structure is a critical component for a successful project. Consequently, it is important to maintain as much of the natural vegetation as possible. If at all possible, the construction site should be approached and the construction work accomplished from the riverward side of the bank to leave the existing upper bank vegetation undisturbed.

(a) One Ton Per Foot Immediately After Construction

(b) Same Site One Year Later

Figure 7.7 Typical Longitudinal Peaked Stone Toe Protection

Figure 7.8 Typical Longitudinal Peaked Stone Toe Protection With Tiebacks

Longitudinal peaked stone toe protection is easily combined with vegetative treatments for a composite design (Figure 7.9).

The centerline of the longitudinal peak stone toe protection should be constructed along a smooth alignment, preferably with a uniform radius of curvature throughout the bend. The upstream and downstream ends of the structure should be protected against flanking and eddy action.

Where the bank materials are highly erodible, and the adequacy of an unsupported stone placed along the toe of the bank may be marginal, stone dikes can be placed at intervals as "tiebacks" to prevent erosion from forming behind the structure. A spacing of one to two multiples of channel width can be used between tiebacks. At the very least, a tieback at the downstream limit of the structure is recommended.

**Longitudinal Stone Fill Toe Protection.** With longitudinal stone fill toe protection, a top elevation and crown width for the stone are specified, along with bank grading and/or filling to provide for a consistent cross-section of stone. The finished product could just as easily be classified as a thickened stone armor to provide a launchable toe, with the top elevation of the armor being well below top bank elevation. In fact, this method is sometimes referred to as **reinforced revetment**. There are two basic configurations of longitudinal stone fill toe protection. One method is to place the toefill stone adjacent to the high bank with the tieback stone fill placed in trenches excavated into the high bank as shown in Figure 7.10. In some instances it may be necessary to place the toefill stone riverward of the high bank as shown in Figure 7.11. Longitudinal stone fill toe protection is often used as the toe protection with other methods for upper bank protection.

Longitudinal stone fill toe protection can be "notched" in the same manner as a transverse dike or retard in order to provide an aquatic connection between the main channel and the area between the structure and the bank slope.

## 7.2 OTHER SELF-ADJUSTING ARMOR

Some armor materials other than stone which have the ability to adjust to scour, settlement, or surface irregularities are:

Concrete blocks;
Sacks filled with earth, sand, and/or cement; and
Soil-cement blocks.

Materials which have been occasionally used in the past, but which have serious shortcomings, are:


Figure 7.9 Longitudinal Peaked Stone Toe Protection In Combination With Willow Post Upper Bank Protection

Figure 7.10 Longitudinal Stone Fill Toe Protection Placed Adjacent to Bank With Tiebacks

Figure 7.11 Longitudinal Stone Fill Toe Protection Riverward of High Bank With Tiebacks

Rubble from demolition of pavement or other source;
Slag from steel furnaces; and
Automobile bodies.

## 7.2.1 CONCRETE BLOCKS

### 7.2.1.1 Description

The discussion here will focus on armor revetments composed of blocks which are placed as individual components. Additional discussion of concrete blocks fastened together in flexible mattresses is provided in 7.4.1.

A wide variety of block shapes and placement techniques can be used. Some have evolved from engineering analyses, some from observation and empiricism, and some from improvisation using readily available materials.

Blocks designed specifically for bank armor are commercially available. Forms for casting concrete blocks locally are often available from distributors, and may be an economical alternative to purchasing and transporting precast blocks.

### 7.2.1.2 Advantages

Concrete blocks are durable, provide permeability for bank drainage, and are amenable to complementary vegetative treatment. Most designs provide easy pedestrian access to water's edge, and may be more aesthetic than other materials. Channel boundary roughness is less than with many other techniques. Hand-laid blocks will fit irregularly shaped areas, and do not demand access by heavy construction equipment.

### 7.2.1.3 Disadvantages

A fabric or granular underlayment (“filter”) is often required. Successful performance of the underlayment is more critical than with a riprap armor. In areas of high turbulence or waves, displacement of one block can lead to successive displacement of adjacent blocks.

If blocks are cast on-site, delays from inclement weather may be a problem.

At sites that are subject to theft or vandalism, blocks of an attractive size and shape may suffer serious attrition.

#### 7.2.1.4 Typical Applications

In addition to typical application as bank armor, blocks can be used effectively for special features such as ditch and spillway linings, culvert outlets, walkways. They are suitable for areas to be vegetated which are subject to erosive forces which vegetation alone could not withstand.

Manufactured blocks are sometimes the least-cost alternative for self-adjusting armor. This is usually in regions where riprap must be transported long distances at great expense, or at sites of high erosive forces where a thick armor of riprap can be replaced by a thinner armor of concrete blocks.

They are well-suited for projects where labor-intensive hand placement is acceptable. Efficient mechanized placement is an option when the blocks are fabricated into mattresses.

#### 7.2.1.5 Design Considerations

Manufacturers' recommendations and/or guidance from laboratory tests and field experience, should be followed in determining block thickness and other details.

### 7.2.2 SACKS

#### 7.2.2.1 Description

Sacks as an armor material can be considered to be artificial “rocks” of uniform size and shape. The sacks may be made of paper, burlap, or a synthetic material. The fill material may be soil or aggregate of various types, with or without cement.

#### 7.2.2.2 Advantages

Sacks can be placed on a steeper slope than stone.

Materials are often available locally. The hydraulic roughness is low, and they form a walkable surface. The “cobblestone” effect may be more aesthetic than some other materials.

#### 7.2.2.3 Disadvantages

A sack armor may tend to act monolithically on steeper slopes, therefore small failures can lead to large ones. The characteristic of being “stackable” may lead to their use on slopes too steep for long-term geotechnical stability, although this is a flaw in design rather than an inherent flaw of sacks themselves.

Synthetic bags, which are sometimes marketed as being suitable for filling with soil or sand rather than a cementious mixture, may be vulnerable to environmental hazards such as fire, ice, vandalism, livestock traffic, floating debris, and rupturing by the roots of vegetation.

A fabric and/or granular underlayment ("filter") is usually required, whereas that may not be the case with a riprap blanket. Successful performance of the underlayment is more critical than with riprap.

A sack armor may not be as likely to support vegetative growth as readily as some other armor materials, especially if a cementious filler is used, or if the sacks are placed on a steep slope. However, in situations where vegetative growth is not desirable, this would be an advantage.

### 7.2.2.4 Typical Applications

Sacks are especially suitable for use on transitions to steep slopes, or in areas where they are aesthetically desirable. If low-cost labor is available, they may be the most cost-effective method, especially on small projects.

### 7.2.2.5 Design Considerations

If commercial bags are used, then the manufacturer's guidance should be followed. Otherwise, the following guidance should be used:

Sack material selection is not critical if the sacks are to be filled with a cementious mixture, as long as they are strong enough to withstand the stress of handling, and will not degrade before the cement sets up. The choice of sack material can then be based on economics, considering the total operation of filling, closing and placing. Some commercial bags have ingenious provisions to speed filling and closing, thus reducing labor costs. Prefilled bags are available in some areas. An alternative to specifying a particular sack for work to be contracted out is to allow bidders a choice of sacks, within broad guidelines.

Sack size should be small enough for laborers to handle. General purpose sacks such as burlap bags or sandbags should have a capacity larger than the desired in-place volume, so that the open end can be folded under the bag as it is placed.

The usual filler material is a sand/cement mixture. Since labor costs are high regardless of fill material, use of a non-cementious filler should be considered only if significant savings would result, and a long life is not required. One such case would be for slopes where vegetation will be established for permanent protection, and permanent toe protection is provided by some other material. Otherwise, a cementious filler is recommended. A common mix is 5 parts aggregate to 1 part cement by volume. Ideal aggregate characteristics are discussed in 7.2.3, but streambed sands are usually suitable.

A typical sack revetment is shown in Figure 7.12. Placing the bags flat on the bank slope is recommended only if the slope is flatter than 1V on 2.5H. The practice on steeper slopes is to provide an overlap, which adds to structural stability as well as allowing some adjustment to scour and settlement without exposing bare bank. On slopes of 1V on 2.5H or 1V on 2H, the bags should be overlapped by placing with the long dimension pointing toward the bank, while on slopes steeper than 1V on 2H, the bags should be overlapped with the short dimension pointed toward the bank. This produces the most efficient bank coverage while still providing the desired overlap between bags. The bags should be placed with staggered vertical joints, as in laying bricks.

Filling of bags is usually done with a portable concrete mixer when a soil-cement mix is used. For maximum convenience in handling, the bags can be filled with dry material rather than adding water during the mixing process. After placement, the bags can be sprinkled with water to speed hydration. Ambient moisture, rainfall, and/or stream flow will complete the hydration process.

There are two alternative approaches to bonding between adjacent sacks. “No bonding” permits individual sacks to adjust to scour and settlement, whereas “bonding” provides greater overall structural strength. The designer must decide which is preferred for a particular application. Generally, bonding is desirable only if design velocity is so high that individual bags might be displaced. Otherwise, adjustability is desirable. Bonding can be discouraged by using tightly woven sacks or placing heavy paper between adjacent courses. Bonding can be encouraged by using porous sacks, placing cement between cold courses, or driving rods through adjacent bags.

## 7.2.3 SOIL-CEMENT BLOCKS

### 7.2.3.1 Description

Soil is mixed well with sufficient cement to provide a durable bond between soil particles. The resulting monolith is broken into blocks of various sizes, which are used to armor the bank.

Figure 7.12 Typical Sack Revetment

### 7.2.3.2 Advantages

Besides the general characteristics of adjustability to bank irregularities and self-healing properties, soil-cement blocks allow the utilization of locally available materials.

### 7.2.3.3 Disadvantages

Soil-cement blocks have a lower specific weight than riprap, and obtaining acceptable gradation and durability are highly dependent on closely controlled construction operations. Construction operations are adversely affected by wet or cold weather.

### 7.2.3.4 Typical Application

Soil-cement blocks are most often used when stone is prohibitively expensive, suitable soil for aggregate is available at or near the job site, and personnel experienced in making the blocks are available. Cost savings over alternative methods are more likely on larger projects which amortize the cost of operations set-up.

### 7.2.3.5 Design Considerations

Since soil-cement blocks are simply man-made rocks, the general principles of effective riprap design apply. However, the lower specific weight of soil-cement requires larger block sizes for equivalent protection, and size criteria as precise as those for riprap do not exist.

For other aspects of design, extensive research and field experience has resulted in detailed recommendations by the Portland Cement Concrete Association and others. The following points are especially important:

> Specifying a suitable soil as aggregate is critical. Although soil-cement can be made from almost any soil, soil with at least 55 percent sand and no more than 35 percent fines is recommended. A "graded" soil of mostly sand, but with some non-clayey fines and gravel provides the optimum combination of workability, strength, durability, and minimum cement requirements.
>
> Blocks with a low cement content may be vulnerable to damage from waves, impingement by high velocity streamflow, and abrasion from transported sediment.
>
> A controlled gradation of finished blocks is best obtained by spreading mixed soil-cement in slabs of varying thicknesses, then scarifying the upper portion of each slab early in the curing process. Following curing, the slabs can be

broken into blocks by driving heavy equipment over them. Sizes of the broken blocks will vary according to the thickness of the slabs and the distance between scarification lines.

Careful quality control during construction is vital to insure that specifications are met.

## 7.2.4 RUBBLE FROM DEMOLITION

### 7.2.4.1 Description

The ideal rubble for erosion protection is a dense, durable material such as concrete or asphalt with a size gradation similar to riprap.

### 7.2.4.2 Advantages

Rubble is economical, and recycles material that otherwise might be wasted.

### 7.2.4.3 Disadvantages

Even dedicated advocates of economy and recycling are likely to view rubble on a stream as unesthetic at best. Leachates from some rubble may pose a water quality problem.

Since rubble is usually available only on a “take it or leave it” basis, it may be too small and/or too large. Losses of finer material due to piping, overbank drainage, and streamflow is likely. Conversely, larger rubble precludes attaining a uniform and efficient layer thickness.

### 7.2.4.4 Typical Applications

Rubble would be considered where the justification for a more sophisticated but expensive armor does not exist, suitable rubble is available, and the environmental shortcomings are acceptable. It is often used in windrow form.

### 7.2.4.5 Design Considerations

Although precise control is likely to be impossible, the same general principles as for riprap will apply to weight, gradation, and durability requirements for rubble. The layer thickness should be equal to at least 1.5 times the maximum block size, although controlling the placement of larger blocks may not be practical, and their in-place orientation may depend more on chance than on design specifications.

When rubble contains large amounts of fines and/or oversize blocks, the layer thickness should be increased generously over the theoretical riprap thickness that would be required for the same site conditions.

A granular or fabric filter can be used to improve performance, but at the sacrifice of economy. Some risk in performance is inherent in rubble, and the additional risk of using it without a filter is usually accepted.

## 7.2.5 SLAG FROM STEEL FURNACES

### 7.2.5.1 Description

Slag is a granular material which is a by-product of steel-making. It is most commonly known for its use as railroad track ballast.

### 7.2.5.2 Advantages

Slag may be relatively inexpensive when available locally, and its use recycles material that might otherwise be wasted. It is dense, durable, and angular, and is often available in a range of sizes, which gives it the same basic properties as stone riprap.

### 7.2.5.3 Disadvantages

Leachates from slag may affect water quality, and some displacement of slag by persons searching for scrap steel has been reported. At one site on the Ohio River, some spalling from weathering and subsequent erosion of the fines has been observed, but this has not occurred at other sites.

### 7.2.5.4 Typical Applications

Slag would be a suitable choice where it is the least costly effective armor material, and where site conditions and chemical tests of the slag indicate that there would be no detrimental effects on water quality.

#### 7.2.5.5 Design Considerations

Principles of design are the same as for stone riprap. Slag from oxygen or electric furnaces is denser than that from blast furnaces, and may even be denser than stone. Therefore, the riprap design criteria in Appendix A would be applicable. The designer may have a choice of different gradations if slag is commonly used locally for construction. The size gradation is sometimes enhanced by the addition of scrap refractory brick.

Slag has been used both with and without an underlayment. On the Ohio River, an 18 inch blanket without underlayment was as successful as a 12 inch blanket on top of engineering fabric.

### 7.2.6 AUTOMOBILE BODIES

Automobile bodies are included is this listing only because they have been used occasionally for erosion protection. No redeeming features beyond low cost can be claimed. Environmental considerations make their use as streambank protection objectionable.

## 7.3 RIGID ARMOR

The following paragraphs outline the general description, advantages, disadvantages, typical applications, and design considerations for **most rigid armor** used as a bank stabilization method:

> Rigid armor is an erosion-resistant material which has little or no flexibility to conform to bank irregularities occurring after construction. Typically, the armor is placed directly on the bank slope in a fluid or chemically reactive state, then hardens.
>
> The most common rigid armors are:
>
> > Asphalt;
> > Concrete;
> > Grouted riprap (or other grouted armor material); and
> > Soil-cement.
>
> Materials which have a more restricted use, but which can be classified as rigid armors, are chemical soil stabilizers, and clay.
>
> Advantages, disadvantages, typical applications, and design considerations for rigid armor are discussed collectively, followed by a discussion of distinctive characteristics of each type and sources for additional information on each type.

Advantages are: The most common rigid armors will withstand high velocities, have low hydraulic roughness, and prevent infiltration of water into the channel bank. They are practically immune to vandalism, damage from debris, corrosion, and many other destructive agents. The most common rigid armors are easily traversed by pedestrians.

Disadvantages are: A rigid armor requires careful design and quality control during construction, and unfavorable weather conditions can cause construction delays. Chemical soil stabilization, clay, and ice have a limited range of effectiveness.

Provision for draining groundwater and preventing the buildup of excess positive pore water pressures, in the form of a filter or subsurface drains, must usually be provided for impermeable armors, which may significantly increase the cost of the project.

Most rigid armors are difficult or impossible to construct underwater, although this difficulty can be alleviated for concrete by using one of the commercially available fabric mattresses (see “Concrete” below). Asphalt has been placed underwater in some cases (see “Asphalt” below).

Rigid armor, being inflexible, is susceptible to breaching if the bank material subsides or heaves. Increased wave runup on a smooth rigid armor may be a concern for some projects.

Some of these materials have little to recommend them environmentally, being biologically sterile and perhaps unacceptable aesthetically, depending on the surroundings.

Typical applications are: Rigid armor in the form of concrete, asphalt, or grouted riprap is often considered for use in situations where high velocities or extreme turbulence make adjustable armor ineffective or very expensive. Typical uses are in conjunction with hydraulic structures or in artificial channels on steep slopes.

Rigid armor may be the preferred alternative in flood control or drainage channels where low boundary roughness is mandatory, or in water supply channels where prevention of water loss due to infiltration into the bank is important. It is suitable for bank slopes which must be easily traversed by pedestrians or recreational users, if the slope is not too steep for safety.

Rigid armor is sometimes the least costly alternative, typically where adjustable armor is not available locally, especially if a geotechnical analysis of the bank material indicates that elaborate subsurface drainage work is not necessary.

Design considerations are: Careful attention to geotechnical stability of the bank, provision for overbank and internal bank drainage, and toe protection is especially critical for rigid armor. Flexible or self-launching toe protection is appropriate in many cases, such as in larger channels where dewatering during construction is impractical and significant toe scour during high flows is expected.

### 7.3.1 ASPHALT

Asphalt is available in three forms: Pure asphalt, which can be mixed with soil or other aggregate and spread on the bank; cutback asphalt, which is pure asphalt mixed with solvent; and asphalt emulsion, which is pure asphalt mixed with water and an emulsifying agent. The generic term "asphalt" applied to bank stabilization usually infers pure asphalt. The other two types can be used in the same manner as chemical soil stabilizers; that is, by being sprayed directly onto the bank and allowed to penetrate the soil before hardening into a cohesive mass. The properties of an emulsion can be varied by using various emulsifying agents.

Asphalt mixes with a high sand content are sometimes used to retain some permeability to relieve hydrostatic pressure. However, these mixes have been reported to become more brittle and less permeable upon long exposure to the elements, and weathering may result in a slow loss of thickness.

The use of asphalt placed underwater on the Lower Mississippi River was discontinued because of problems with placement control and inconsistent performance, and as a result of the development of an efficient and effective articulated concrete mattress. However, it should be noted that the Lower Mississippi River presents extremely difficult construction conditions, with high velocities, great depths, and steep underwater slopes.

### 7.3.2 CONCRETE

On slopes above water, concrete can be placed in the conventional manner with forms, or can be pumped into fabric mattresses which serve as forms for a fine aggregate concrete. Prefabricated slabs may be the least costly alternative for some sites. An armor of relatively small slabs would assume some of the characteristics of concrete block armor (see 7.2.1).

Fabric mattresses are the preferred method for underwater placement, and are available in various configurations. The appropriate design for a given application will depend on the need for relief of hydrostatic pressure, the design velocity, and the preferred roughness characteristics. Some mattresses are described as being flexible by the manufacturer, although this description should be objectively examined by the project engineer if flexibility is a critical factor for a specific project. Section 7.4.2 below provides further discussion under "Fabric Mattresses."

### 7.3.3 GROUTED ARMOR

Grouting of an armor layer with asphalt or concrete enables the armor to withstand higher flow velocities, provides a smooth surface for pedestrian or vehicle access, and reduces the hydraulic roughness of the armor. Grouting is also sometimes used with gabion armors or structures to increase the resistance of the gabions to corrosion and abrasion.

Grouting allows the use of locally available stone or cobbles which are not large enough to withstand design flow velocity if used alone. A grouted armor of streambed cobbles with the surface of the cobbles exposed is more aesthetically pleasing than most other armor materials.

When applied to a riprap armor, grout which thoroughly penetrates the riprap enables a smaller stone size and thinner layer to be used for a given velocity of flow. If grouting is used only to reduce hydraulic roughness or to improve trafficability, thorough penetration of the armor layer is not necessary. However, in that case, stone size and layer thickness should be designed as if the grout were not present.

### 7.3.4 SOIL-CEMENT

Soil-cement will withstand relatively high velocities and is usually less expensive than concrete, asphalt, and grouted riprap. It is more durable than chemical stabilization, clay, and certainly ice, but usually somewhat less durable than concrete, asphalt, and grouted riprap, assuming that sound design and construction procedures are followed for all. A typical soil-cement application is shown in Figure 7.13.

General factors affecting the use of soil cement were discussed under soil-cement blocks in Section 7.2.3. Its use as a rigid armor is usually an economic decision. However, an additional consideration is that, when mixed in a batch plant rather than mixed in-place on the bank slope, it can be placed as a rigid armor in stair-step fashion. This allows it to be used on steep slopes where permitted by geotechnical considerations, and provides the capability to construct an armor of great thickness if required to resist high flow velocities, abrasive sediment transport, and wave attack. Use of a batch plant has the further advantage of providing consistent quality control.

In-place mixing is an alternative if a relatively flat bank slope is provided. However, the thickness of the armor is then limited by the mixing capability of the mixing vehicle, and quality control is not as assured.

Figure 7.13 Typical Soil Cement Application

### 7.3.5 CHEMICAL SOIL STABILIZATION

A number of commercially available products, including lime, can be used to increase cohesion of soil particles or to provide a hard film at the soil surface. Under favorable conditions, even those products which eventually break down upon exposure to the elements may be effective in providing erosion protection until vegetation becomes established.

Because specific site conditions can greatly affect performance, the feasibility of this approach and appropriate design guidance for a particular project can be determined only by obtaining evidence of satisfactory performance under similar conditions from previous users or from the manufacturers.

### 7.3.6 CLAY BLANKET

When the upper slopes of a bank are exposed to small erosive forces, but the existing soil has insufficient cohesion to resist them, it may be effective, environmentally beneficial, and economical to utilize a clay blanket instead of a structural armor. The cohesive properties of the clay provide resistance to erosion, and its moisture holding properties may enhance vegetative growth. This approach would be prudent only on projects where the consequences of failure in the event of unfavorable streamflow or weather conditions are low, or where adequate monitoring, and reinforcement if required, are assured.

## 7.4 FLEXIBLE MATTRESSES

The following paragraphs outline the general description, advantages, disadvantages, typical applications, and design considerations for **most flexible mattresses** used as a bank stabilization method:

> The basic concept of a flexible mattress is that material or objects which cannot resist erosive forces separately can be fastened together or placed in a flexible container to provide adequate resistance to erosive forces, while partially retaining the desirable characteristics of adjustable armor, especially that of flexibility.
>
> The most common flexible mattress materials are:
> Concrete blocks;
> Fabric; and
> Gabions.
>
> Materials which have a more limited use are:
> Grids (for confining earth or other fill material);
> Used tires; and
> Wood.

Advantages are: Flexibility to adjust to scour or settlement and still remain in contact with the bed and bank is the most obvious shared trait. Most mattress materials which are sold under trade names share another advantage - they are available in various configurations, thus can be applied to a variety of situations.

Flexible mattresses can be placed underwater with a relatively high degree of confidence. If properly anchored to a geotechnically stable bank, they can be placed on steep slopes.

They can be walked upon easily, thus are suitable for slopes used by pedestrians.

Disadvantages are: Mattress components are subject to deterioration from the elements and vandalism. However, the damage is often within acceptable limits, and, since the various types are affected differently, identification of the hazards enables the designer to select an appropriate mattress for a given application. The construction of some types of mattress is labor intensive, and may require skills not commonly available. However, the labor intensive aspect may not be a disadvantage in all cases, and may be an advantage in some cases.

Typical applications are: This compromise between adjustable armor and rigid armor is most attractive when economical materials can be used for the mattress. In fact, the origin of some variations can be traced directly to creative use of local materials where no protective material of local origin was adequate to withstand the erosive forces in a given application, and where the most suitable method was the one which required the least amount of costly imported material, a requirement which is often met by a flexible mattress.

Some types of mattress are suitable for use where erosive forces are so severe, or construction operations are made so difficult by great depth and/or high velocity of flow, that other types of armor are not effective or cannot be placed reliably. An example is the articulated concrete mattress developed by the U.S. Army Corps of Engineers for the Lower Mississippi River over the last 60 years. The ACM has evolved into a highly efficient product placed by specialized floating equipment adapted to operation under severe conditions of velocity and depths.

Some types of mattress are suitable for use in areas which are to be used by pedestrian or vehicular traffic.

Design considerations are: Beyond the general considerations discussed below for the various types of mattresses, the manufacturers of commercial mattresses have developed very detailed design guidance for their products.

### 7.4.1 CONCRETE BLOCK MATTRESS

The advantages, disadvantages, and typical applications of concrete blocks as armor were discussed in Section 7.2.1. Some additional considerations which apply to the use of concrete blocks in mattress form are as follows:

> Mattresses provide a higher degree of safety from progressive failure of the armor due to displacement of individual blocks from hydraulic or geotechnical forces or vandalism.
>
> Placing of mattresses is more mechanized and less labor intensive than placing individual blocks.
>
> Some commercial mattresses incorporate an engineering fabric, which will eliminate the need for a separate filter layer under some conditions.

Precast concrete blocks can be formed into a flexible mattress in several ways: by fastening them to engineering fabric, by fastening them together with cable or synthetic rope, or by forming them in ingenious shapes which are then interlocked. All of these varieties are commercially available.

#### 7.4.1.1 Design Considerations

Concrete block mattress will usually withstand hydraulic forces greater than an equal thickness of riprap. However, all designs are not equal, and manufacturers being considered as a source for a specific project should be asked to furnish evidence of adequacy.

The most conservative design approach, which would be especially appropriate for areas of high turbulence and areas where waves create the critical loading, is to ignore any extra uplift resistance which is provided by the blocks being attached together. This extra resistance would be assumed to be a safety factor, rather than being taken into account when selecting a block size for hydraulic loading. The rationale is that the “pumping” action created by even a small amount of uplift of the blocks might result in loss of bank material or failure of the mattress connections or bonding system.

Anchoring the mats to the bank slope is usually recommended. This should not be considered as adding to the geotechnical stability of the bank, but rather as providing a margin of safety from mat displacement if small slope movements occur.

### 7.4.2 FABRIC MATTRESS

#### 7.4.2.1 Description

Fabric mattresses made of synthetic material and filled with concrete grout, other cohesive mixtures, or sand are available from various manufacturers. Tubular-shaped bags are also available; these can be filled and placed either parallel to the streambank as a bulkhead or perpendicular to the streambank as a dike, or can be used to fill scour holes or undermined slopes.

#### 7.4.2.2 Advantages

A fabric mattress is relatively easy to place, and fill material is often available locally. Some designs have a low hydraulic roughness.

#### 7.4.2.3 Disadvantages

Some designs provide only limited permeability and flexibility to conform to irregularities in the bank.

#### 7.4.2.4 Design Considerations

Many different designs are available. This allows the designer to discuss particular site conditions with manufacturer's representatives in order to select a mattress which emphasizes particular requirements, i.e., stability under hydraulic forces, filter and permeability properties, flexibility, hydraulic roughness, resistance to deterioration, or compatibility with vegetation. One form, intended primarily for filling with concrete, integrates cables into the mattress to provide flexibility without separation even if the bag deteriorates.

Potential subsurface drainage problems must be identified, and the installation designed and monitored accordingly.

Use of a non-permanent fabric and fill material may be acceptable on the upper bank if vegetation for permanent protection is planned. This approach has also been used on lower banks and bed where the fabric is permanently underwater, and not subject to atmospheric deterioration, vandalism, or impact from debris or vessels. Obviously some degree of uncertainty exists when using perishable materials, so site conditions, expected project life, and the consequences of failure must be carefully evaluated.

Polyester fabric has been reported to be subject to deterioration from the high pH of concrete curing.

Anchoring the mattress to the bank slope is usually recommended by manufacturers.

### 7.4.3 GABION MATTRESS

#### 7.4.3.1 Description

A gabion mattress consists of a mesh container filled with cobbles or quarried stone. Several firms market the containers and furnish technical assistance. Specialized equipment or accessories are sometimes used on large jobs for efficiency, or on jobs requiring underwater placement.

A form of gabion which is a hybrid between flexible mattress and adjustable armor is the “sack” or “sausage,” which can be filled faster than mattress or box shapes, making it suitable for use in emergency situations. However, it makes less efficient use of material, and is less common than traditional mattress or boxes.

#### 7.4.3.2 Advantages

Since relatively small stones are used to fill gabion mattresses, a filter underlayment is often not required. The hydraulic roughness is fairly low, especially if the gabions are carefully filled or grouted. The appearance is more natural than some other materials, and gabions are conducive to vegetative growth.

A gabion mattress is often used in conjunction with gabion dikes or retaining walls, since the same construction practices can be used. A gabion mattress can be tailored to irregular shapes in transitions from one type of protection to another, or around drains and other structural features.

#### 7.4.3.3 Disadvantages

A gabion mattress is less flexible than some concrete block mattresses. The mesh is not immune to deterioration from the elements, although corrosion-resistant coatings or grouting can be used to significantly alleviate potential problems of deterioration.

#### 7.4.3.4 Design Considerations

Manufacturers have developed detailed guidance for every feasible application, and this guidance should be obtained early in the planning process. Some general factors to consider in design are discussed in the following paragraphs.

For given hydraulic conditions, a gabion mattress can be substantially thinner than a riprap blanket. Recent model tests, as reported by Simons et al. (1984) provide guidance for mattress thickness related to shear stress and velocity. Two conditions were analyzed:

Initial movement of stones within the mattress; and

A condition where the mattress shape has been deformed by stone movement, but the mattress is still functional.

The tests indicated that ungrouted mattress thicknesses of nine inches (23 centimeters) or less could withstand significantly higher velocities than previously believed. Grouting would increase the allowable velocities even more. It was noted, however, that wire mesh strength may be a major factor controlling mattress stability.

Filler stone sizes must be more uniform than for typical riprap. The smallest size must be larger than the mesh openings, but the largest size must be small enough to eliminate large voids between stones in the filled mattress. Streambed cobbles are sometimes used to reduce cost where they are locally available.

Corrosion, abrasion, and vandalism can be minimized by grouting the gabions with a sand-asphalt mastic or concrete. However, the accompanying loss of permeability may require that special provision for hydrostatic pressure relief be provided. Also aesthetic and environmental aspects of the project may suffer.

For corrosive or abrasive situations, previous users or manufacturer's representatives should be consulted for information on measures used to ensure successful application under similar conditions. Even in areas of good water quality, water chemistry may be such that galvanized wire will corrode. For this reason, a polyvinyl chloride coating on the wires is often specified.

Care in handling and filling is necessary to avoid damage to protective coatings, especially with plastic coated gabions in cold weather. For work above water, filling in place is preferable to filling before placing. Otherwise, extreme care must be taken during handling to avoid deforming or damaging filled gabions.

Construction must be carefully supervised. Some crucial points, such as care during filling, and complete lacing of the mattress components, are costly to a contractor's operation and present a temptation for short cuts. Some handwork is usually necessary for proper filling, and this in particular may be resisted by a contractor unless it is clearly specified.

On steep slopes, keying-in or anchoring the mattress at the top of the slope is recommended.

For large jobs, a manufacturer may offer custom-sized gabions for optimum design and construction efficiency.

## 7.4.4 GRID CONFINEMENT

### 7.4.4.1 Description

This approach uses a grid several inches thick, resembling a "honeycomb," to confine soil or other material on the bank slope. It provides a level of protection which is less resistant to erosion than conventional armoring, but more resistant than unsupported soil, granular fill, or vegetation.

### 7.4.4.2 Advantages

By using locally available materials, grid confinement may offer a cost savings where erosive forces are moderate. When filled with soil, it is highly compatible with vegetative treatment.

Grid confinement also enhances the resistance of the slope to shallow failure. The grid can also serve as a form for bituminous or similar armor material on steep slopes, in which case some beneficial increase in flexibility of the armor can be expected, since the grid material acts as joints in the armor.

### 7.4.4.3 Disadvantages

When filled with a non-cohesive material, grid confinement will not withstand as high velocities as some other flexible mattresses. When filled with asphalt or concrete, it assumes to some extent the unfavorable characteristics of rigid armor discussed in 7.3.

### 7.4.4.4 Design Considerations

Some manufacturers have developed guidance for allowable velocities and other hydraulic factors, and can furnish specific recommendations for particular applications based on laboratory tests and field experience.

The manufacturer may recommend a geotextile underlayment, and, if the grid is filled with a non-porous material, filter points to allow drainage should be incorporated into the design.

The grid should be anchored to the bank slope according to the manufacturer's recommendation.

### 7.4.5 USED-TIRE MATTRESS

#### 7.4.5.1 Description

Used-tire mattress consists of tires fastened together with bands, cable or rope. Whole tires are normally used, but tires sliced in half or tires with pieces removed are sometimes available.

#### 7.4.5.2 Advantages

Tires are often available at low cost, and use of tires in erosion control may be more environmentally sound than landfill disposal. A tire mattress is conducive to the establishment of woody vegetation.

#### 7.4.5.3 Disadvantages

No formal guidance is available for determining limits of hydraulic forces. A tire mattress is not suitable for severe conditions unless an underlayment and multiple layers of tires are used, which negates the cost advantage. Vulnerability to hydraulic forces, vandalism, and theft is greatest immediately after construction, before exposed areas become vegetated.

Environmental regulations may prohibit the use of tires in many areas. Also, a tire mattress is not aesthetic, although if site conditions permit heavy vegetative growth and deposition of sediment, the appearance improves with time.

#### 7.4.5.4 Design Considerations

To combat vandalism and theft, and to reduce buoyancy during high flows, if whole tires are used, then one or more of the following measures should be employed:

Stout and durable synthetic or galvanized connections;

Backfilling with earth over the completed revetment; and

Cutting or burning a hole in the upper sidewall of each tire.

Less durable connections can be used if the quick establishment of woody vegetation is certain, and vandalism is not expected to be a problem. However, the savings in cost are not likely to be significant.

The mattress should be anchored on the slope with screw anchors, driven anchors, or buried anchors. If little toe scour is expected, and the outer edge of the mattress is not placed underwater, the outer edge can be anchored in the same manner as the slope. An alternative for little toe scour and moderate velocities is to fill the outer few rows of tires with concrete. A more conservative approach is to use one of the toe protection methods discussed in 6.3.

Tire diameters should not be allowed to vary greatly, otherwise it will be difficult to make good connections consistently. A simple way to minimize this difficulty is to specify that only standard tires of nominal 13-inch to 16-inch wheel diameter be used.

## 7.4.6 WOODEN MATTRESS

### 7.4.6.1 Description

Wooden mattress is one of the oldest techniques of bank stabilization, even though it is seldom used now in developed regions. The mats may be made of poles, brush, or lumber. The material can be fastened together by weaving, binding, cabling, clamping, or spiking. The mattresses are sunk by ballasting with stone or other heavy materials. Some types of mat may be so buoyant that the ballast is a significant component of the protection, as well as a large part of the cost.

On navigable rivers during periods when current speed is slow enough that the mats can be safely maneuvered in tow, mats with sufficient buoyancy can be assembled near the materials supply point or near a source of labor, then towed to the project site. Individual tows of as much as 150,000 square feet of mat were reported on the lower Mississippi River.

At least one marine construction firm has adapted modern technology to the construction of wooden mattress, while still retaining traditional skills for use where appropriate. They have also extended new technology to the point of developing synthetic materials for use in mattresses, in order to overcome some of the inherent problems of wood.

### 7.4.6.2 Advantages

Wood is usually available locally, and is a renewable resource. If inexpensive labor is available, a wooden mattress may be the least cost alternative. Wood is relatively durable when permanently submerged in freshwater.

### 7.4.6.3 Disadvantages

Near-site availability of material is usually required for wooden mat to be competitive with other methods. Assembling and placing the mattresses are labor-intensive operations. Design and construction is surprisingly complex, requiring skills which have become rarer as other methods have become more popular.

In most climates wood will deteriorate quickly if exposed to alternate wetting and drying. Therefore, it is not a suitable material for use above low water unless treated lumber is used (which may affect water quality), or unless frequent maintenance or the establishment of vegetation is planned.

The durability of metallic components may be poor underwater. This is a significant shortcoming, since the mat must remain intact to function properly.

Construction is difficult if currents are swift, depths are great, or the flow carries large amounts of floating debris.

The designs that use lumber or long poles woven into a mat are stiff, which limits their capacity to conform to bank and bed irregularities. Severe erosive forces require thick mats, which reduces flexibility in proportion to thickness, and loss of permeability greatly increases the difficulty in sinking in swift currents. In fact, the stiffness of sturdy woven pole and lumber mats led to them being replaced on the lower Mississippi River about 1900 by willow fascines, or bundles, cabled together into mats. The fascine mat was more flexible. However, the high labor cost and diminishing willow supply, as well as sometimes ineffectual performance, led to the fascine mat being replaced in turn about 60 years ago by the much more successful articulated concrete mat.

### 7.4.6.4 Design Considerations

The major causes of failure of wooden mattresses on the lower Mississippi River, as discussed by Elliott (1932). The disadvantages of this technique listed above provide a basis for defining the most critical elements of design. The most serious shortcomings were found to be:

> Rotting of the mattress where it was alternately wet and dry;
>
> Inability of the mattress to adjust to scour at its toe (riverward edge); and
>
> Failure of fasteners and connecting components from corrosion, abrasion, or fatigue.

Design of a wooden mattress should address these points of vulnerability by utilizing the following measures:

> A secure, durable interface between the wooden mattress and whatever more durable material is to be used to armor the upper bank should be specified. Since this interface will likely be underwater at the time of construction, unless the work is done at extremely low river stages, a material which is suitable for reliable placement underwater is dictated. Stone is an excellent choice, although many other adjustable armors or flexible mattress materials

would also be suitable. Simplicity and economy of construction will be enhanced if the same material is used for the connection as for the rest of the upper bank. An overlap should be provided to ensure that any downslope movement of the wooden mattress after placement will not result in an unprotected area of bank.

If significant toe scour is expected, then a wooden mattress should be supplemented by separate toe protection measures.

Fasteners and connectors should be of materials which are resistant to corrosion, abrasion, and failure from fatigue due to flexing of the mattress when subjected to hydraulic forces. Synthetic materials, stainless steel, or heavily coated metallic components are therefore advisable.

Other major considerations for design are:

An overlap should be provided between adjacent mattresses in order to compensate for uncertainties in underwater placement and future differential displacement of the mattresses by hydraulic or geotechnical forces. As an example, individual wooden mattresses on the lower Mississippi River were overlapped from a minimum of 5 feet to a maximum of 15 feet with the adjacent mats. The individual mats were laid from downstream progressing upstream so that the downstream edge of each mat lay over the upstream edge of the adjacent mat, so that the upstream edges were not exposed to the flow.

Because wooden mattresses are relatively inflexible, and because shaping them to irregularities in the bankline is difficult, protruding points and other irregularities should be removed or smoothed as much as possible during bank preparation operations, and sunken debris that would interfere with the mattress making contact with the underwater slope should be removed. This requirement presents a dichotomy which is a major obstacle to the use of wooden mattress, since the fact that wooden mattress is durable only when permanently submerged restricts its use to the subaqueous bank, where removal of bankline irregularities and debris is most difficult, and in fact is likely to be impractical at depths greater than ten feet with standard construction equipment, even if barge mounted.

# CHAPTER 8

## INDIRECT TECHNIQUES FOR EROSION PROTECTION

As in the previous chapter, descriptive information for most techniques presented in this chapter is generally followed by a discussion of advantages, disadvantages, typical applications, and design considerations as appropriate. In order to minimize redundancy, these topics are discussed at the broadest possible level in the hierarchy of the text; in other words, aspects which are shared by all techniques are discussed at the beginning of the chapter; aspects which are shared by a group of techniques are discussed at the group level; aspects that are peculiar to a smaller category of techniques, or to a single technique, are discussed at the appropriate level of specificity.

The extent of the discussion of specific techniques ranges from detailed design guidance to a brief description for some specialized techniques. Therefore, a complete understanding of a specific technique requires perusal of all material at a broader level in the text, as well as material peculiar to that technique.

The following paragraphs outline the general description, advantages, and disadvantages for **most** indirect techniques used in bank stabilization methods:

> Indirect protection structures extend into the stream channel, and redirect the flow so that hydraulic forces at the channel boundary are reduced to a non-erosive level. Indirect protection techniques can be classified as follows:

- Dikes and Retards
  - Dikes
    - Permeable dikes
    - Impermeable dikes
  - Retards
    - Permeable retards
    - Impermeable retards
- Other Flow Deflectors
  - Bendway weirs
  - Iowa vanes

Advantages are: Little or no bank preparation is involved for indirect protection. This reduces costs and riparian environmental impacts, simplifies the acquisition of rights-of-way, eliminates material disposal problems, and usually allows existing overbank drainage patterns to remain undisturbed.

Existing channel alignment and/or geometry can be modified, although the changes may not always be beneficial or predictable.

Indirect approaches usually increase geotechnical bank stability by inducing sediment deposition at the bank toe, although this process may not be rapid or reliable enough to meet project goals.

Disadvantages are: Where geotechnical bank instability or erosion from overbank drainage is a major factor, the fact that indirect protection does not immediately relieve these problems can be a serious and often unacceptable shortcoming.

Because significant changes in flow alignment, channel geometry, roughness, and other hydraulic factors often result from indirect protection structures, special attention must be given to the stream's morphological response.

Some types of indirect protection structures may be a safety hazard if the stream is used for recreation or navigation, and the aesthetics of some types often leave much to be desired, although vegetative growth may ultimately reduce the visual impact in most regions.

Since indirect methods extend into the stream channel, their construction may be difficult, especially during high flow. Also, the structures may be subjected to severe hydraulic conditions throughout their lifespan, and should be closely monitored to insure that maintenance is performed as necessary.

## 8.1 DIKES AND RETARDS

The following paragraphs outline the general description, advantages, disadvantages, typical applications, and design considerations for **dikes and retards** used in bank stabilization methods:

“Dikes” are defined as a system of individual structures which protrude into the channel, generally transverse to the flow. Other terms which are often used are “groins,” “jetties,” “spurs,” “wing dams,” and if they protrude only a short distance into the channel, “hard points.” The term “dikes” is also used in some regions to refer to earthen flood-containing structures, which are also called “levees,” but that usage is not relevant here.

"Retard" is defined as a continuous structure approximately parallel to the streamflow. It can be a single structure or two, or more, adjacent and parallel structures, in which case the space between may be filled with various materials. Other terms that are sometimes used are "longitudinal dikes," "parallel dikes," "jetties," "guide banks," and "training walls." Most designs have occasional "tiebacks" extending from the bank out to the main structure. These tiebacks have the appearance of dikes. In fact, many retard designs can be viewed as being a dike system with a longitudinal component connecting the ends of the dikes.

Advantages are: Dikes and retards provide a means to modify the channel alignment if that is a project requirement. They are also well suited to the incremental construction approach and are amenable to the establishment of woody vegetation. Also, many designs use locally available material.

Dikes and retards offer the opportunity for incorporating a wide variety of environmental features. They may increase the diversity of aquatic and terrestrial habitat, although subsequent sediment deposition may be detrimental to shallow water habitat. The reduction of water surface area due to deposition within the dike or retard system will reduce evaporation rates, which may be considered to be a benefit in semi-arid areas.

Disadvantages are: Those designs which involve "perishable" materials or mechanical connections are susceptible to gradual deterioration and to damage by debris, fire, ice, and vandals.

Channel capacity at high flow is decreased initially when dikes or retards are constructed, although the channel will usually adjust by forming a deeper, though narrower, cross-section, and the ultimate result may even be an increase in conveyance capacity. However, the extent of the adjustment cannot be always be predicted reliably, even with physical or numerical models. Since conservative assumptions on future deposition and vegetative growth would be necessary, extensive use of dikes or retards must be approached with caution on projects where channel flood conveyance is a concern.

Typical applications are: Dikes and retards can be applied to a wide range of conditions. However, the most common use is on shallow, wide streams with moderate to high transport of suspended bed material, because shallow channel depths reduce the required height of structures, a wide channel provides room for the channel alignment and geometry to adjust, and a heavy supply of suspended bed material accelerates the rate of induced deposition.

Where long-term funding is provided, they are often built in increments in order to reduce costs by modifying the river's form gradually, and taking advantage of subsequent deposition to reduce total project cost.

Dikes and retards are often used on large rivers to increase depth for navigation, in addition to improving the alignment and stabilizing the banks. They can be used to stabilize the channel alignment upstream and downstream of armor revetments in bends, since the shallower depths, moderate velocities, and less concentrated drift loads upstream and downstream of bends are more suitable to in-channel structures than is the bend itself.

Dikes and retards can be used where establishment of riparian vegetation is a high priority. Initial plantings and natural establishment of native species can be supplemented by later plantings on sediments deposited within and behind the structures, or by sloping and vegetating the upper bank slopes once lower bank stability has been attained.

No formal and widely tested design criteria for dikes and retards exist, although design concepts based on experience and model tests have been developed for some applications. A study performed for the U.S. Federal Highway Administration and reported by Brown (1985) is one of the most comprehensive analyses of dikes. That report is based on model tests, a literature review, and a survey of several hundred field installations. Studies by the U.S. Army Corps of Engineers (USACE, 1981) also provide observations on design parameters. Some findings from these and other studies, and from practice, are discussed later under specific headings. The following general concepts apply to the design of both dikes and retards:

(a) Because there are so many variations in design, one must be cautious of becoming so engrossed in the details of materials and construction that the importance of the basic layout is overlooked. If the basic principles in 5.1 and 6.1 are followed, then there are many specific designs that will work equally well, but if basic principles are neglected, the most painstaking attention to detail will be in vain.

(b) Simplicity should be a design goal. The principles of value engineering are particularly applicable for dikes and retards. Other factors being equal, a design with fewer components and mechanical connections will be more durable and less costly than a more complicated design.

(c) Basic decisions on materials and structural design for a specific project are inherent in the selection process discussed in Chapter 5. Other aspects are covered below under more specific headings. An exhaustive investigation by the engineer of all design alternatives for a specific project is neither practical or necessary. Many of the overwhelming number of possible variations are described by California State Department of Highways (1960) and FHWA (1985). Beyond their practical value, these publications provide testaments to the wide variety of river stabilization problems encountered in practice, and to human imagination in problem solving.

(d) The need for toe and local scour protection may be less obvious than for armoring techniques, but is still important (see 6.3). Using a permanent scour protection material, such as stone, in conjunction with dikes or retards of a less durable material will allow the designer to be less concerned about dike and retard durability, if woody vegetation will eventually provide the same erosion protection to the middle and upper bank as the dikes or retards provided in the beginning.

(e) Since mechanical connections cannot be made underwater, river stages during the construction season will affect some aspects of design, dictating that prefabricated elements or a launchable material such as stone be used for the portion of the structure which will be built underwater.

### 8.1.1 DIKES

#### 8.1.1.1 Advantages

Advantages of dikes as compared to retards is that they will usually be less expensive for a given situation, and will not interfere with access to the stream. Also, after the stream has adapted to the initial project, dikes can be extended farther into the stream if necessary to fully achieve project objectives, whereas with retards, modification of the initial alignment is likely to be much more expensive.

#### 8.1.1.2 Disadvantages

Disadvantages of dikes as compared to retards is that they will usually be less effective in eliminating bank erosion. Dikes are more vulnerable to floating debris than are retards, since dikes present abrupt obstacles to flow, whereas retards, being approximately parallel to flow, will allow much of the floating debris to pass through the project reach. Also, erosion between the dikes in a system will often be more severe and of longer duration than erosion within a retard system.

### 8.1.1.3 Typical Applications

Typical application of dikes is in straight reaches and long radius bends, since as bend radius decreases, spacing must decrease, and the required number of dikes soon reaches a point where a retard could be built for the same cost or, if channel realignment is not required, an armor technique could be used.

### 8.1.1.4 Design Considerations

Design considerations for dikes beyond the general factors discussed in 8.1 is one of the most complex issues in design of erosion protection works. There is general agreement on some aspects, but considerable diversity, even controversy, on others. A complete reading of the Federal Highway Administration report is recommended to obtain full understanding of the complexities involved in dike design (Brown, 1985).

Design involves the following major parameters:

(a) Permeability;
(b) Length;
(c) Spacing;
(d) Angle with respect to flow;
(e) Height;
(f) Bankhead design; and
(g) Structural scour protection.

(a) Since permeability affects some of the other design parameters, it is appropriate to discuss it first. Permeability is defined as the ratio of the area of openings in the dike to the total projected area of the dike, and is expressed as a percentage. If the stream carries only a small amount of debris, or the dikes are low enough that debris will pass over them during most flows, the permeability can be assumed to be the as-built condition. However, if debris loads are moderate to high, then some reduction in permeability with time should be assumed.

FHWA (1985) suggests that where a large reduction in at-bank velocity is required, such as in sharper bends, permeability should not exceed 35 percent. Where a moderate reduction in velocity is sufficient, such as in bends with mild curvature and less easily erodible bank material, permeabilities up to 50 percent can be used. In mild exposures such as straight reaches with low erosion potential, permeabilities up to 80 percent may be successful. However, permeabilities greater than 50 percent are not recommended unless success under conditions similar to the project at hand can be documented.

The U.S. Army Corps of Engineers (USACE, 1981) suggests that permeability should decrease with decreasing size and quantity of sediment carried by the stream in suspension. That is, greater permeability is allowable if a large amount of bed material sediment is carried in suspension, whereas less permeable structures are required if small amounts of sediment, or predominately fine sediment sizes, are transported in suspension.

Permeability and the choice of materials used to construct dikes are interrelated. To achieve a given permeability, there will be more than one possible combination of materials; conversely, a given choice of materials can be used for a range of permeabilities by altering the design details (see 8.1.1).

(b) The length of individual dike structures (from the existing bankline to the riverward end of the structure) is dictated by the desired alignment of channel if the channel is to be realigned. Where stabilization of the existing bankline is the only requirement, then determining the proper length is not so simple, and there is wide variation in practice.

FHWA (1985) states that dike length affects the local scour depth at the tip of the dike, the angle of flow deflection induced by the dike, and the length of streambank protected by each individual dike. Optimum dike length is to some extent a function of dike permeability. Selection of an appropriate dike length is site-specific. However, the following general guidance is provided:

| Permeability (percent) | Recommended Projected Length of Dike (percent of channel width) |
|---|---|
| 0-35 | 15% or less |
| 80 | 25% or less |

For permeabilities between 35% and 80%, linear interpolation between 15% and 25% of channel width can be used to determine maximum allowable length. Channel width is defined as bankfull width, and projected length of dike is measured perpendicular to the main flow direction.

If the dikes are being used to change the channel alignment, then the dike lengths will often exceed these limits, and the length of individual dikes in a system will vary widely depending upon the location of the realigned channel with respect to the existing bankline. These limits basically represent values beyond which additional length is no longer cost-effective, if stabilizing the bank in its present position is the only objective, since difficulties associated with increased scour at the end of the dikes, and other flow anomalies, may more than offset the additional length of bank protected by each dike.

General practice is to define length as the original constructed length, not including any length dug into the bank for scour protection (see “bankheads” below). A very conservative approach for design would be to assume that deposition after construction would effectively move the bankline riverward, and to compute design dike length from that point. Since design dike length would be shorter with this approach, design spacing would be closer (see “spacing below”). The logic for this approach is that the dikes must ultimately protect the newly deposited bankline. The weakness in the logic is that if the dikes ultimately form a new bankline, then they will by definition, also protect it. Therefore, the cost-effectiveness of this very conservative approach may be questionable.

(c) Spacing and length are usually considered to be related, thus much of the literature addresses the ratio of the two rather than separate values. In the absence of a need to construct dikes to a predetermined channel alignment, the optimum length/spacing ratio becomes a site-specific economic determination, involving a trade-off between shorter dikes at a closer spacing against longer dikes at a greater spacing.

FHWA (1985) states that although spacing is a function of the length, angle, and permeability of the dikes, as well as channel curvature, a parameter called “expansion angle” may be used to better understand the relationship of these variables. In a straight channel, for short dikes with permeabilities less than 35%, the expansion angle is the same as for impermeable dikes, about 17 degrees. For permeabilities of 35% or greater, the expansion angle increases as permeability or dike length increases.

FHWA (1985) also shows a method of determining dike spacing in a bend by using a projection of a tangent to the thalweg at each dike tip. This procedure gives the maximum allowable spacing, which should be decreased for a more conservative design, particularly if short dikes or highly permeable dikes are used, if the banks are easily erodible, or if the consequences of failure are high. They suggest that the expansion angle be used to determine a prudent decrease in spacing from that which would be used in a straight reach.

USACE (1981) and Copeland (1983) report a range in practice varying from a spacing equal to dike length to a spacing of 6.3 times dike length, and describe USACE model tests at the Waterways Experiment Station, Vicksburg, Mississippi, indicating that the optimum spacing of impermeable dikes in a bend was between 2 and 3 times dike length. However, they caution that those tests should not be applied verbatim to practice, stating that “Spacing-to-length ratios for specific projects are best determined by previous experience in similar circumstances or site-specific model studies.” USACE (1981) describes USACE model tests at the Missouri River Division's Mead Hydraulic Laboratory, Nebraska, of very short impermeable dikes (“hard points”) in a straight channel, which indicated that flow downstream of each structure expanded at about a 20-degree angle from the main flow, a finding compatible with FHWA guidance. This suggests that a spacing of about 3 times dike length for that type of dikes in a straight reach would be adequate.

A conservative recommendation for dikes in bends would be a spacing equal to dike length. California Department of Highways (1960) also states that spacing should equal dike length unless “scalloping” of the bankline due to erosion between the structures can be accepted. That guidance is then qualified by a recommendation that impermeable dikes not be used in bends. However, that pessimistic viewpoint may have been influenced by unsuccessful use in sharp radius bends, or by failures due to inadequate bankhead design.

Even if one of the approaches discussed above is used to quantify spacing, the location of individual dikes may need to be modified according to site conditions. For example, the project site may have localized "plunge pools" or "shelves" because of variations in bed or bank material, or other local anomalies. If so, dike locations can perhaps be adjusted so that no one dike requires a large volume of material or unusually long piling, or conversely, so that no one dike is built with insufficient volume of material or pile penetration to be stable against future local scour.

If dike spacing is determined by using an approach based on projections of tangents to streamlines or to the thalweg, the engineer should be aware that if the channel upstream of the project is migrating, the alignment of incoming flow and the thalweg may change with time. A conservative approach would be advisable in such cases if the predicted future condition will result in a more direct impingement of flow on the bank which is to be protected.

(d) The optimum angle that dikes should have with respect to the direction of flow is a subject upon which there is much disagreement. The controversy may be due to the influence of less obvious, and perhaps overlooked, factors overriding the effect of angle at a specific site. In the absence of compelling evidence to the contrary, dikes which are constructed on the shortest path from the bankline to the desired new channel alignment will be the shortest, thus the cheapest. Usually, this path will be approximately perpendicular to flow, or the bankline, or a compromise between the two. FHWA (1985) suggests that angle is not critical to permeable dikes, but that better performance may be obtained with impermeable dikes if the upstream dike in a system is constructed at an angle of about 150 degrees, with subsequent dikes having successively smaller angles, reaching a minimum of 90 degrees for the downstream dike. Whether results are better to the extent of outweighing the additional cost for longer structures is a matter for debate.

Permeable dikes are sometimes angled downstream to shed debris and ice, although if debris and ice loads are consistently heavy, permeable dikes may not be the appropriate protection method to begin with. In any event, the "shedding" effect should be considered to be only an additional safety factor, and should not lead to disregarding debris and ice loads in structural design.

Contrary to intuition, dikes angled downstream may form downstream scour holes nearer to the bank than if they were perpendicular to the bank or angled upstream to the flow, because overtopping flows will tend to form an erosive "roller," or plunging flow, immediately adjacent and parallel to the structure, to the detriment of bank stability.

As in determining dike spacing, any future change in alignment of flow due to channel migration upstream should be considered when designing the angle of dikes with respect to flow direction.

(e) General factors affecting the optimum height of erosion protection works were discussed in Section 6.2.5, "Top elevation of protection." Although the term "elevation" is more precise than "height," the term "height" will be used in the discussion below because it is more commonly used in dike design practice.

Height of dikes in a system is often related to bank height which, in turn, can be related to some recurrence frequency of river stage. In humid areas, bank height is often a one or two year return interval for streams that are neither aggrading or degrading. Unfortunately, any design relationship of dike height to bank height is more conceptual than quantitative, and no generally accepted precise guidance can be stated.

In spite of the uncertainties involved, some general guidance can be stated regarding the determination of appropriate dike height. FHWA (1985) states that dikes need be only high enough to protect the bank zone of active erosion, but follows that general axiom with the following three specific guidelines:

> Dike height should be no higher than top bank, but no lower than 3 feet below "design flow."
>
> Impermeable dikes should be submerged 3 feet at the most severe expected flow condition, because the local scour associated with submerged dikes seems to be smaller and located farther from the bank than that associated with unsubmerged dikes.
>
> Permeable dikes should be lower than flow stages that carry significant debris loads.

Application of these guidelines will often result in a fairly conservative design, which is understandable, since the guidelines were developed for application to the protection of highway facilities from channel migration. However, the latitude which exists in the determination of the design flow and the most severe expected flow condition still leaves considerable latitude for the engineer to be more or less conservative as appropriate for a specific project, even if dike height is based on these guidelines.

In practice, the uncertainties of the physical effects of height often become moot, because the economics of dike construction often dictate that dikes be considerably lower than top bank elevation. For permeable dikes, the rapid increase in cost as the height increases is due to structural factors, as discussed below under "permeable dikes." For impermeable dikes, the rapid increase in cost is due to the exponential increase in structure volume as height increases. For a specific project, there will usually be a height beyond which dikes are not economically feasible. Fortunately, that limiting height is often greater than that required for successful performance, since stabilization of the toe and lower bank slopes are the key to success in most applications. Also, the incremental construction approach discussed in 5.3.3 can sometimes be used to reduce the additional cost of increased height.

As a very broad generalization based on past experience, an acceptable range of dike heights in many situations is between 1/3 and 2/3 of bank height, or in the case of incised streams, 1/3 to 2/3 of the distance between low water elevation and the elevation of a flow with a return interval of one to two years. The lower figure will certainly not be a conservative design, or even as conservative as designing a retard to the same elevation, but dikes are not as suitable as retards for a situation requiring conservative design in any case.

As a design refinement, the height of a dike can vary from the bankhead to the riverward end, i.e., be sloped downward. This provides two advantages:

> It creates less constriction of flow as flow increases, because the riverward portion is submerged at higher flows. This is particularly important for impermeable dikes.
>
> It results in maximum economy, because the structure can then more closely follow the contour of the bank and channel bottom, reducing the required size of structural components of permeable dikes, and reducing the volume of impervious dikes. This in fact is the only feasible approach when prefabricated components of a single size, such as jacks, are used.

A combination of sloped and level profiles is often used when the channel is to be shifted away from the bank significantly.

A dike profile can be "notched" for environmental purposes, allowing some flow to enter the dike system to enhance habitat diversity and water quality, while still diverting sufficient flow to provide erosion protection to the bank.

Physical model studies reported by Franco (1982) indicated that a system of dikes having successively lower elevations in the downstream direction tended to accumulate more deposition than other designs. However, that finding is not usually pertinent to bank protection dikes. The model studies were for long structures in a wide channel, designed to deepen the crossing between two bends. Following that scheme for dikes in a typical eroding channel would require either that the upstream structures be relatively high, or the downstream structures relatively low, choices which would respectively either increase the cost of the upstream dikes substantially, or reduce the effectiveness of the downstream dikes. In a bend, the hydraulics of flow would likely overcome whatever beneficial effect a stepped-down system might have, resulting in the strongest attack on the bank being where the dikes would be the lowest.

(f) Dike bankheads must be designed so that erosion does not flank the structure; that is, disconnect it from the bank. Some local erosion is acceptable, but it must be limited. There are two basic approaches:

(1) Excavate a trench into the bank and extend the dike back into the trench (called the dike "root").

(2) Pave the downstream bank with an armor, and if conservative design is called for, also pave a lesser distance upstream. This usually involves grading the bank and placing riprap.

Specific guidance here is at least as difficult as for other dike design parameters. The best guide, unfortunately, is previous experience in similar circumstances, which is no comfort if similar experience is lacking. The difficulty lies in predicting velocity fields and the depth and precise location of the scour hole which will develop at an unprotected bankhead. For very expensive hydraulic structures, this difficulty is often resolved by large-scale physical models, which is usually impractical for bank protection projects.

The following are "rules of thumb" based on experience, but they cannot be considered formal guidance:

> For dikes in straight reaches, approach (1) above involves extending the dike root into the bank a minimum distance equal to the bank height. If the depth at high flow of local scour holes in the adjacent area, such as around erosion-resistant bank material or other obstructions to flow, can be observed or estimated, a more conservative approach is to extend the root into the bank a distance of the bank height plus that scour depth. If eroded "eddy pockets" downstream of existing protrusions into the channel are observed, the root should be at least as long as the maximum landward extent of those pockets. For areas of severe erosion, such as in bends, the root should be longer. Examples of extremes from practice: A root length of 300 feet is commonly used on Mississippi River dikes, but as little as 10 feet has been successful on very small tributary streams.
>
> USACE Mead Laboratory model tests described in USACE (1981) suggest that lateral erosion between dikes, thus required dike root length for approach (1), is related to stream depth (or bank height), velocity of flow, and dike length.
>
> When using approach (1), backfilling over the dike root, routing surface drainage away from the backfilled area, and vegetating the disturbed area will help prevent post-construction erosion and will improve the aesthetics of the project. Design of the backfill can be simple or sophisticated, depending upon specific site conditions. The simplest approach is simply to replace the excavated material in the most expeditious way (with due

allowance for subsequent settlement of the backfill) then letting nature take its course afterwards. The most sophisticated approach is to fill all voids in a stone root to the extent possible by flushing sand into the voids, then placing engineering fabric over the top of the stone and sand, then completing the backfill with compacted lifts of silt or clay, then vegetating the backfill and adjacent disturbed areas.

Bank height or bank height plus a scour allowance can also be used as a starting point for designing approach (2) above. The length of armor downstream of a dike should be a multiple, perhaps three for average conditions, of that dimension. Upstream paving is optional, but the distance need not exceed bank height. Normally, the bank toe just upstream of a dike is a depositional area. For designing stone paving, the guidance in Appendix A can be referred to, but because that guidance is not intended for application to highly turbulent situations, stone size and thickness should be greater than that which would be designed for a riprap blanket not adjacent to a dike, perhaps a multiple of 1.5 or 2.

Stone is an excellent choice for a root dike material, even if the dike itself is of other materials, because in other than mild erosion situations, the ability of the dike root to adjust to scour is critical.

In severe conditions, dike roots or armoring of bankheads can become large cost items, which is part of the reason why dikes can be more expensive than conventional bank armoring in those cases.

(g) Structural scour protection prevents undermining and failure of rigid dikes, and fortifies dikes of an adjustable material such as stone against unacceptable loss of elevation or length.

Alternative approaches to structural scour protection are to:

Place a blanket (sometimes called an "apron") of adjustable armor or a flexible mattress on the bed under and adjacent to the dike. As with bankhead armor, this blanket or mattress should be of a stronger design than if it was being used at the same site not adjacent to a structure. USACE (1981) found that an apron of stone or gabion mattress did not reduce the depth of scour at the tip of a dike, but did enhance the stability of the structure by moving the scour away from it.

Place extra material at the end and on the side slopes of the dike. The extra material will launch into a scour hole and limit its extent, thus leaving the dike length and elevation intact. This approach is simpler to construct than an apron, but allows the scour to approach close to the dike. For a stone dike, it would consist of a crown wide enough for stone to launch into the scour along the face of the dike without breaching the crest elevation, and a slope at the end of the dike which is significantly flatter than the slope of natural repose of stone.

Specify extra penetration for pile structures so that scour will not fail the piling. However, unless the facing of the dike can adjust downward with the scour, or the dike is constructed entirely of driven piling this approach detracts from performance, since the total permeability of the structure is increased as the bed beneath the structure erodes. More flow is allowed to pass through the structure, and the scour may endanger bank stability. This is likely to be an expensive approach as well. For example, required pile penetration for one dike design on the Sacramento River was computed to be 13 feet if protected from scour, 34 feet if unprotected. Even if the dikes are constructed with adjustable facing which displaces downward with bed scour, maintenance of the design elevation by adding more facing will be required, unless the original design provided for lowering of the effective height of the structure.

Add structural features such as an L-head, "hockey stick," or T-head (sometimes called "hammerhead"), in order to move the scour away from the dike proper. This approach actually coalesces into a retard design if carried to the extreme of affecting overall flow rather than just local scour. Also, the scour around the added feature itself must still be addressed by one of the other approaches. A similar approach was used on some Mississippi River stone dikes in the 1960's, in the form of stone "rib spurs" built intermittently along the upstream face of dikes which were experiencing loss of stone due to launching into the scour hole caused by lateral flow along the upstream face. There was no conclusive evidence that this attempt to move the scour away from the dike was more cost-effective than simply adding additional stone to the dike cross-section to compensate for the launching, and the practice was soon discontinued.

Use a dike design that will maintain contact with the bed as scour occurs. Examples of this approach are jacks, "Palisades," tire-post dikes, and anchored trees.

Use a hydraulically smooth design for the end of impermeable dikes, and round structural members for permeable dikes (FHWA, 1985). However, this alone is not likely to be sufficient if the dike intercepts much flow.

A safety factor is sometimes added by using two or more of the above approaches in combination. Examples are a dike structure designed to maintain bed contact, along with armor or mat to limit scour; or extra pile penetration at the end of the dike, along with armor or mat for the full length of the dike as well as beyond the tip.

## 8.1.2 PERMEABLE DIKES

### 8.1.2.1 Advantages

The advantages of permeable dikes as compared to impermeable dikes are that they are equally, if not more effective when used on streams with relatively high concentrations of suspended sediment, and are often less costly.

### 8.1.2.2 Disadvantages

The disadvantages are that they are less durable than stone dikes and some other impermeable dike materials, and are usually considered less aesthetic, although the visual impact may ultimately be lessened by the growth of vegetation.

### 8.1.2.3 Design Considerations

Design considerations beyond those general considerations discussed previously for dikes involve materials, structural design, and miscellaneous items.

(a) Posts and piles for permeable dikes, and the main members of jacks, may be wood, steel, or concrete. The economic feasibility of using treated wood for decay prevention is a project-specific decision, as discussed by Petersen (1986). However, water quality considerations may preclude the use of some preservatives. Some early jack designs were patented, and although their use has become practically generic, the present legal status of these patents is unclear. Other shapes, such as tetrahedrons, are sometimes used. The function of tetrahedrons is identical to jacks, but they are stronger and more expensive than jacks made of the same components.

    At least one proprietary design of permeable dikes exists, called "Palisades." They are constructed of panels of synthetic netting attached to pipes driven into the stream bank and bed. The panels can slide down the pipes to adjust to changing contours of the bank and bed.

    Anchored trees or brush provide an "all-in-one material." The primary shortcomings are durability, and in some regions, availability.

    The most common facings are boards and wire fencing of various types. For pile dikes in deep streams, the piles are closely spaced without a separate facing material (Peterson, 1986). This design retains the original permeability ratio even if the bed beneath a dike scours, as long as the dike does not fail from loss of pile penetration, and it also makes construction of a permeable dike practical even in fairly deep water.

Hardware and fasteners, such as nails, bolts and cable, will be largely dictated by the choice of other materials. Corrosion resistant hardware must be used unless the work is temporary.

(b) Structural design is an iterative process. The goal is to achieve the required height and permeability in the most economical way, considering the cost of materials and the construction techniques that will be used. The variables for fence-type permeable dikes are:

Lateral loads (drag force of current, impact of debris);
Spacing, size, and penetration of piles;
Size of sub-components (boards, fencing, cables, anchors); and
Supplementary bracing.

The vulnerability to failure from lateral loads increases with dike height, since the moment arm of the force is greater, and the amount of debris carried by the stream, as well as the speed of impact, is likely to increase as river stage increases. As the height of the dike increases, this combination dictates an increase in the size of the structural members, as well as an increase in pile penetration for those designs using driven piles. These factors cause the cost to increase dramatically as the dike height increases.

Typical practice for penetration of piles or posts is that at least 1/2 to 2/3 of the total length should be in the ground. Factors that influence required penetration are the nature of the bed and sub-bed material, the potential for scour, and anticipated lateral loads from hydraulic loading and floating debris or ice. The nature of the material through which the piles or posts are to be driven must be known in order to determine if driving will be feasible. Encountering unanticipated difficulties during the driving operation may cause contractual difficulties as well as perhaps necessitating redesign of the work.

If previous experience has developed a design that has been successful in applications similar to the project at hand, it is more prudent to apply that experience rather than over-extending the safe bounds of theory with numerical structural analysis using imprecise assumptions. Figure 8.1 shows some typical designs of permeable dikes.

(c) Some miscellaneous design considerations are as follows:

The facing material should be attached to the upstream side of dikes.

Large trees which may be undermined and fall onto the dikes should be removed. Otherwise, existing vegetation should be preserved to the greatest extent possible. If clearing of the bank is necessary to provide construction access, stumps should be left in the ground, since regrowth of some species will occur.

Cuts made in treated wood members should be recoated with a preservative.

### 8.1.3 IMPERMEABLE DIKES

#### 8.1.3.1 Description

The relative merits and faults of impermeable dikes as compared to permeable dikes were discussed in 8.1.2. Impermeable dikes can be built of the following materials:

Stone;
Gabions;
Earth, sand, or other material faced with armor;
Bags or tubes filled with sand or grout;
Walls of steel, wood, or concrete piling;
Wooden cribs filled with earth or stone;
Asphalt; and
Masonry.

Stone and gabions are the most commonly used of these materials. Although some of these materials are not truly impermeable, dikes constructed of them have permeabilities low enough that the amount of flow which passes through the structure is negligible. Discussion of the general characteristics of most of these materials is provided in Chapter 7. Typical impermeable dikes are shown in Figure 8.2.

(a) Palisades

(b) Board Fence Dikes

Figure 8.1 Typical Permeable Dikes

### 8.1.3.2 Design Considerations

Design considerations for impermeable dikes beyond the general factors discussed previously for dikes are as follows:

(a) Stone gradation for stone dikes is less critical than for riprap armor, which is fortunate, because there is no widely accepted method for designing stone gradation for dikes. Stone displacement due to scour will tend to be self-healing if the maximum stone size is adequate, and enough stone is present.

A larger maximum stone size required for dikes than would be used for riprap armor on the same stream, because turbulence and local acceleration of flow adjacent to the dike creates large hydraulic forces. Also, if stone is being placed in large depths and/or high velocities, larger sized stones will suffer less displacement as they fall through the water column, thus control of placement is easier and the amount of stone which falls outside the design cross-section will be reduced. The range of maximum stone sizes commonly used in practice is from 200 pounds to 5,000 pounds, depending on the depth of water, velocity of flow, and the amount of flow being intercepted by the structure, all of which influence the displacement forces on the stone and the amount of scour which will occur during and after construction.

The gradation of stone below the maximum size is dependent to a large degree on the economics of quarrying and handling the stone. Ideally, stone will be well graded, with a low percentage of spalls and waste particles. However, too restrictive a gradation will increase the cost of quarrying beyond the benefits gained. In general, the higher the cost of transporting the stone to the project site, and the more severe the hydraulic conditions, the more justified a strictly controlled gradation, since transportation costs for “waste” material in the stone is the same as for the high-quality stone.

(b) The crown width of stone dikes depends primarily on the amount of anticipated scour adjacent to the structure (see “Structural scour protection” in 8.1.1 above) and the height of the structure. As a practical matter, a crown width of about 2 feet is the smallest that can feasibly be constructed, while still providing a minimal amount of stone to launch into any scour that may occur. Crown width should be increased beyond that if the maximum stone size is larger than 2 feet, or if significant scour adjacent to the structure is expected and the height of the dike is so small that the amount of stone available to launch off the downstream side slope will be insufficient to retain an effective dike height.

Figure 8.2 Typical Impermeable Dikes

The method to be used to construct the dikes may also influence the choice of crown width. If land-based equipment is to be used, but the area where a dike is to be constructed is underwater or otherwise impassable, specifying a crown wide enough for the operation of hauling and handling equipment should be considered, since the additional crown width will strengthen the dike as well as expediting construction. Whether this is cost effective for a given structure will depend on the capabilities of the work force, the cost of stone, and the height of the structure, since the additional volume of stone required for a wider crown will increase exponentially with the height of the structure.

(c) The slope of natural repose can be specified for side slopes of stone dikes. Providing extra stone to launch into any scour hole that may occur adjacent to the structure can be accomplished more efficiently by increasing the crown width, as discussed in "Structural scour protection" in 8.1.1 above, than by attempting to construct a flatter side slope to accomplish the same purpose. Specifying the slope of natural repose simplifies construction, because then only the elevation and crown width of a dike require control in the latter stages of construction, which is especially advantageous if the side slopes of a dike are underwater. For pre-construction estimates of stone quantities, the slope of natural repose is commonly assumed to be 1 vertical on 1.5 horizontal, although some variation can be expected depending on stone gradation, construction procedures, and site conditions.

(d) The slope of the riverward end of a stone dike is often designed flatter than the slope of natural repose, as discussed in "Structural scour protection" in 8.1.1.

(e) Dikes with a core of earth or other material, with an armor on the surface, are not commonly used because they provide a smaller factor of safety against unanticipated scour and other severe hydraulic conditions than do sturdier structures. Baird and Klumpp (1992) report scour problems with such dikes on the Rio Grande River. A filter of some type between the core material and the armor is likely to be required, which increases the cost. Also, construction of this type of dike underwater is not usually practicable. In spite of these shortcomings, the potential for cost savings may be considerable if the cost of stone or other conventional dike materials is very high.

### 8.1.4 RETARDS

The relative advantages and disadvantages of retards were compared to dikes in Section 8.1.1.

### 8.1.4.1 Typical Application

The typical application of retards is where the channel is to be realigned, but the bend curvature, bank erodibility, debris load, or hydraulic conditions are too severe for dikes to be effective or economical. In some cases where channel realignment is not a factor, retards may be the preferred method if less expensive than bank armoring.

### 8.1.4.2 Design Considerations

Design considerations for retards beyond those discussed in Section 8.1 involve location, height, and tiebacks.

(a) If a change in channel alignment is not required, the preferred location for the retard from the standpoint of economy and efficiency is at a point slightly riverward from the toe of the bank slope. The location of the retard in plan view is determined by identifying that point on surveyed bank cross-sections, then plotting on a plan view that point's location at each cross-section. A smooth alignment can then be drawn through those points which “control” the overall alignment. Those points will be the ones which are farthest out in the channel. If the existing bank alignment is fairly smooth, then the retard alignment will pass through or near all the “preferred” points. However, if the existing alignment is irregular, then the retard alignment must necessarily lie riverward of many of the preferred points. If a pronounced single irregularity causes the retard to be located unacceptably far out in the channel upstream and downstream of the irregularity, then the alternative is to smooth the bankline irregularity by excavation.

(b) The height, or elevation, of the retard is determined by considering the factors discussed previously for dikes. The elevation of the retard can be varied around a bend as the attack against the bank and/or as the erodibility of material varies. This complicates design and construction somewhat, and is seldom done, but does have the potential to increase the efficiency of the design. The United Nations (1953) described some European work as having the retard highest at the apex of the bend, sloping downward to a minimum elevation at the upstream and downstream ends. A concern about that approach, however, would be that the downstream limb of a bend is often where the attack against the bank is greatest at higher flows, and the risk of a low elevation there is greater than for a low elevation at the upstream end. This is especially true after the work has been in place long enough for the normal downstream movement of scour pools and bars to have increased the hydraulic forces along the downstream portion of a retard in a bend.

(c) Tiebacks (sometimes called “baffles”) are mandatory where the retard is located well in front of the bank and in short radius bends, and are recommended in all cases. For simplicity of design and construction, they are often of the same structural design as the retard, but can be of a less costly design if site conditions permit a less conservative approach. The length of tiebacks is determined by the distance from the bank to the retard. The top elevation of tiebacks is commonly made the same as the retard, although a lower elevation can be used for a less costly, but less conservative, design.

The spacing of tiebacks can be designed according to the concepts discussed previously for spacing of dikes. However, such a design would often be overly conservative, since the tiebacks are simply used to reinforce the main protection device, the retard itself. The permissible increase in spacing can be determined for a specific site only by applying judgement, experience, and the factors discussed in 6.6, "Safety factor."

General practice is to place tiebacks on the shortest line from the retard to the bank. This is the least costly approach, and provides a compromise between them being perpendicular to the realigned flow and perpendicular to the existing bankline. The lack of agreement regarding the optimum angle that transverse structures should have with respect to direction of flow is less troubling for tiebacks than for dikes, since the tiebacks are not the primary component of the work.

Tieback bankhead design should follow the same principles as for dike bankheads, but can be less conservative in many cases since the retard itself will usually decrease erosive forces at the tieback bankhead.

### 8.1.5 PERMEABLE RETARDS

The advantages of permeable retards as compared to impermeable retards are that they are equally, if not more, effective when used on streams with relatively high concentrations of suspended sediment, and are often less costly to construct, since materials are usually available locally. Typical permeable retards are shown in Figure 8.3.

The disadvantages are that they are less durable than stone retards and some of the other impermeable retard materials, and are usually considered less aesthetic. They also interfere to a greater degree with access to the stream channel.

Most aspects of materials and structural design are the same as for permeable dikes (see 8.1.2). Other design considerations beyond those discussed previously for retards are as follows:

(a) Double-row retards are sometimes used to increase structural stability and to further reduce flow behind the retard. A double-row design also gives the impression of better toe protection, but that may be illusory for rigid retards, since if the first row fails from toe scour, the second row is likely to fail eventually also. However, the outer row of flexible double-row retards, such as jacks, can displace downward into a scour hole and still provide protection to the inner row.

(a) Board Fence Retard

(b) Jack Field

Figure 8.3 Typical Permeable Retards

Rigid double-row retards are sometimes used as "cribs," filled with various materials to further reduce velocities behind the retard. This is a site-specific decision, dependent on the economics of filling versus using a less permeable facing design, and on the durability required of the filling material. Using local material such as hay or brush reduces permeability at low cost, but at the expense of durability, and relies on future deposition and vegetation for permanent velocity reduction. A stone filling provides permanent toe protection as well as permeability reduction, but requires a substantial facing to retain the stone, and will add substantially to the cost. Used tires (perforated to reduce buoyancy) provide an inexpensive and durable filling, if regulations permit such use. However, undermining or deterioration of the crib may result in an unsightly redistribution of the tires along downstream river banks, adding environmental insult to the injury of a failed structure.

(b) Some designs, such as fence-type retards, require that the bottom member be approximately horizontal. Therefore, some leveling of the streambed along the line of the structure may be required during construction, which limits the use of these designs to ephemeral streams and minor scour situations, unless a material such as stone is used to build up the base. In that case, the stone will also serve as toe protection. Otherwise, any leveling of the streambed to expedite construction must be considered as being temporary, lasting only until the first flow event.

(c) Carlson and Dodge (1962) present a method for determining the suitability of jack retards for a given situation, and for estimating the amount of deposition likely to be induced by them.

### 8.1.6 IMPERMEABLE RETARDS

The relative advantages and disadvantages of impermeable retards as compared to permeable retards were discussed in 8.1.5. Most aspects of materials and structural design are the same as for impermeable dikes (see 8.1.3). An impermeable retard of stone can be considered to be a form of longitudinal stone toe, discussed in 7.1.4 and most aspects of design discussed there are applicable to stone retards.

## 8.2 OTHER FLOW DEFLECTING METHODS

Structures other than dikes and retards may provide a means of altering hydraulic conditions in order to resist bank erosion in bends. One of the most intractable problems of river engineering is posed by the coupled processes of deposition of sediment on point bar faces and scour in the thalweg of bends. Several approaches have successfully addressed these coupled processes in some cases. These approaches alter secondary currents so that sediment transport away from the toe of the bank is reduced. This results in a more uniform cross-section shape, with shallower thalweg depths and a wider channel at low flow. These approaches include Iowa vanes, bendway weirs, and sills.

### 8.2.1 IOWA VANES

The technique called “Iowa vanes” originated from physical model tests performed by the Iowa Institute of Hydraulic Research for the U.S. Army Corps of Engineers (Odgaard and Kennedy, 1982). The purpose of the model study was to define a bank stabilization technique for the Sacramento River which would be both effective and environmentally sound although the proposed solution was not actually implemented. The first field application was sponsored by the Iowa Department of Transportation in 1985 on the East Nishnabotna River near Red Oak, Iowa. Subsequent development of the technique has led to it being patented. At present, the primary use of Iowa vanes is on bank stability problems on small rivers and on local sedimentation problems, such as at water intakes, on larger rivers. Results from these works may in time identify broader applications.

Iowa vanes are fully submerged during high flows, but are above the water level at low flows. The location and orientation of the vanes with respect to flow is critical to success. Also, because success depends upon the structures having a precise effect on the velocity vectors in the bend, stabilization of the upstream bend is recommended if upstream channel migration is likely to change the flow patterns entering the vane system.

Initial evaluation of the East Nishnabotna installation indicated that flowlines through the project reach were not affected by the structures (Odgaard and Mosconi, 1987).

### 8.2.2 BENDWAY WEIRS

Bendway weirs were developed by the U.S. Army Corps of Engineers as a method to increase channel width in bends on the Mississippi River in order to improve navigation conditions and reduce maintenance dredging requirements (Derrick et al., 1994). They also induce deposition in the thalweg of the bend, which should enhance bank stability and reduce the tendency for scouring velocities in the overbank area during floods. The success of bendway weirs is based on the premise that the flow over the weir is redirected at an angle perpendicular to the weir. When the weirs are angled upstream, the water is directed away from the outer bank and towards the inner bank, or point bar.

The weirs on the Mississippi River are level-crested stone structures angled upstream, with a crest elevation about 15 feet (4.5 meters) below low water. The design is based on physical model studies at the Waterways Experiment Station, which indicated that a pronounced upstream angle was required for the structures to function properly. The first system was installed in 1990 on the Mississippi River upstream of the mouth of the Ohio River, and is performing well. That installation and several subsequent ones are being monitored, and other installations are planned.

Environmental aspects of bendway weirs appear to be favorable. Since they are submerged well below low water level, the detrimental impacts on esthetics and safety which are associated with most other indirect protection techniques are eliminated. Also, by providing a rocky substrate for benthic organisms and cover for fish, and by altering the velocity distribution across the cross-section and in the vertical, they improve habitat conditions for some species of aquatic life. Whether detrimental effects would accompany these beneficial effects in other applications would depend upon the environmental context of a specific application.

In recent years, bendway weir theory has been applied to small stream applications as a streambank protection measure (Figure 8.4). The first small stream application was in 1993 on Harland Creek near Tchula Mississippi where fifty-four bendway weirs were constructed (Derrick, 1997a). Since that time, bendway weirs have been built on numerous small streams throughout the country. Some of theses projects have used low-cost, hand placed stone weirs, and weirs constructed of tree trunks and geobags to protect farmland (Derrick, 1997b). Because this is a recently developed technique, the long term success of these structures as a bank stabilization scheme is not known. Further research and monitoring of existing structures is needed to document the long-term performance and to develop more definitive design criteria.

(a) Bendway Weirs on Harland Creek

(b) Bendway Weirs in Combination with Longitudinal Peaked Stone Toe Protection

Figure 8.4 Bendway Weirs on Small Streams

# CHAPTER 9

# VEGETATIVE METHODS FOR EROSION PROTECTION

The two previous chapters addressed structural approaches to erosion protection, in the form of surface armor and indirect techniques. Vegetation's great potential for use in erosion protection, and the requirement that it be carefully planned and designed using skills not usually included in traditional engineering knowledge, merits separate discussion. This chapter is not an exhaustive treatment, but does present a rational overview of the subject. The latest U.S. Army Corps of Engineers guidance for bioengineering for streambank erosion control is discussed in Appendix B.

## 9.1 OVERVIEW

Vegetation is the basic component of what is known as “bioengineering” (Schiechtl 1980) or biotechnical engineering (Gray and Leiser, 1982; Gray and Sotir, 1996). Schiechtl (1980) states that bioengineering requires “the skills of the engineer, the learning of the biologist, and the artistry of the landscape architect.” The concept of bioengineering is ancient, but there has been much recent research and documentation of the topic. The publications just cited, as well as Coppin and Richards (1990), provide comprehensive coverage, and many other works provide discussion of specialized aspects of the subject.

### 9.1.1 FUNDAMENTAL CONCEPTS

Vegetation can function as either armor or indirect protection, and in some applications, can function as both simultaneously. Grassy vegetation and the roots of brushy and woody vegetation function as armor, while brushy and woody vegetation function as indirect protection. The roots of vegetation may also add a degree of geotechnical stability to a bank slope through reinforcing the soil.

Some factors which affect the success of a bioengineering approach, such as weather and the timing and magnitude of streamflows, are beyond the designer's control. Therefore, expert advice, careful planning, and attention to detail are critical to maximizing the probability of success.

Many streambank protection projects include vegetation without conscious thought by the designer, since native vegetation often establishes itself once the processes of bank failure are stopped by structural means. However, if the potential for utilizing vegetation is considered from the beginning, then the effectiveness, environmental aspects, and economy of a project can often be significantly improved.

The general principles of erosion protection discussed in Chapter 6 are fully applicable to vegetative work. In fact, because vegetative works are generally more vulnerable than structural works, particular care must be taken to insure that the overall approach is sound. Beyond those general principles, the details of successful use of vegetation are even more site-specific than for structural bank protection. The terminology of the details can sometimes be confusing, because the technology developed somewhat independently from region to region over a long time period, whereas widespread interdisciplinary interest in the subject, and broad dissemination of the technology, is fairly recent. Also, the many variations on the basic techniques add some confusion to the terminology. However, the basic concepts are straight-forward, and have international and timeless application.

### 9.1.2 ADVANTAGES

The two obvious advantages of vegetation as erosion protection are its environmental attractions and its relatively low cost. A third and less obvious attraction is that it can increase the safety factor of structural protection by enhancing the level of performance. Because many types of vegetative treatment are labor intensive, the cost advantage will be especially prominent in regions where labor is inexpensive, skilled in agriculture, and conscientious.

### 9.1.3 DISADVANTAGES

Some characteristics which make vegetation effective and desirable in most situations may be disadvantages in other situations. However, many of the following concerns will either not be applicable for a specific project, or will be acceptable as compromises in light of vegetation's merits.

The most serious shortcoming is that even well executed vegetative protection cannot be planned and installed with the same degree of confidence, or with as high a safety factor, as structural protection. This is not to say that vegetation will not be adequate, or will not be more cost effective than structural protection in a specific situation, but is rather an acknowledgement that structural protection can be designed to function under more severe conditions of hydraulic and geotechnical instability than can vegetation. Vegetation is especially vulnerable to extremes of weather and inundation before it becomes well established.

Quantitative guidance for the use of vegetation in streambank protection is limited, although there has been progress through recent research.

Most vegetative measures have constraints on the season of the year that installation can be performed. This shortcoming can be mitigated to some degree by advance planning or by developing more than one option for vegetative treatment.

Vegetative treatments often require significant maintenance and management in order to prevent the following problems:

> Growth of vegetation causing a reduction in flood conveyance or causing erosive increases in velocity in adjacent unvegetated areas.
>
> Deterioration of the environmental function of the vegetation due to mismanagement by adjacent landowners, vandalism, or natural causes.
>
> Trunks of woody vegetation or clumps of brushy vegetation on armor revetment causing local flow anomalies which may damage the armor.
>
> Large trees threatening the integrity of structural protection by root invasion or by toppling and damaging the protection works, or by toppling and directing flow into an adjacent unprotected bank.
>
> Roots infiltrating and interfering with internal bank drainage systems, or causing excess infiltration of water into the bank.
>
> In arid regions, vegetation's ability to reduce soil moisture may be a concern. However, this is not likely to be a serious concern if the native plant ecosystem was considered in the initial selection of vegetative species. In any event, a riparian strip of vegetation is not likely to harm the groundwater resource enough to outweigh the positive values of the vegetation.
>
> Many of these problems may be avoided through selection of the appropriate type, and species or clone of vegetation for the purpose. However, designers rarely have the practical experience or formal training in biotechnology to make such selections and expert advice must be obtained from qualified individuals in plant biology and bioengineering.

## 9.1.4 TYPICAL APPLICATIONS

Vegetation is most often used in conjunction with structural protection. Exceptions may be made for very small waterways, for areas of low erosion activity, or for situations where the consequences of failure are low and there is provision for rehabilitation in case of failure. Vegetation can have a particularly important role in the stabilization of upper bank slopes.

Vegetation is especially appropriate for environmentally sensitive projects, whether benefits to recreation, esthetics, or wildlife is the object.

Vegetation is well-suited for incremental construction, either to wait for more favorable planting conditions for specific types of vegetation or to wait for deposition of sediments in the area to be planted. Vegetation is also suitable for inexpensive reinforcement or repair of existing erosion protection works in some situations.

Woody vegetation is useful in preventing or repairing scour at or behind top of bank, especially if the scour resulted from an infrequent flood event which is not likely to recur before the vegetation becomes effective.

Woody vegetation is sometimes used to prevent floating debris from exiting the channel during floods and becoming a nuisance in the floodplain.

# CHAPTER 10

## CONSTRUCTION OF STABILIZATION WORKS

There are two fundamental differences between the construction of bank stabilization works and that of more conventional structures. The first difference occurs simply because part of the work is often out of sight (underwater). The second difference concerns site conditions. These affect the design, performance, and even the appropriateness of the technique being used. Often conditions change dramatically between design and construction. These changes may be caused by unusually high flows, or may be due to normal stream dynamics. Environmental sensitivity to construction operations may change with the season of the year, since nesting, spawning, and other wildlife activities are all seasonal. Also, the timing of construction affects the success of establishing vegetation. To complicate matters further, the construction operation itself may initiate changes in site conditions. Such changes not only require the attention of the designer, they often pose problems for construction personnel and contract administrators.

Specialized aspects of the construction of bank stabilization works are sparsely documented, perhaps because construction personnel tend to focus more on performing work than on academic reporting. This is in contrast to the practice of engineers and scientists, who are usually encouraged, or even required, to document their research findings and practical experience.

It would be inappropriate for this text to attempt to deal with construction in terms of the details of plant, labor, materials, administration, and management. These aspects of construction are to a large degree dependent upon organizational policy, local custom and workforce capability, and the stabilization technique being employed. Therefore, this discussion will concentrate on concepts and ideas which are peculiar to river stabilization work, and which are not widely documented. These aspects are easily overlooked by the designer when other matters are clamoring for attention, but they should be integrated into the planning and execution of construction in order to obtain the most effective, environmentally sound, and economical project.

Some detailed factors which relate to construction of specific types of bank stabilization, but which are more pertinent to the selection and design process, have been discussed in Chapters 5 and 6.

Concepts of construction will be discussed under these headings:

Coordination of design and construction;
Specialized contract provisions;
Environmental protection;
Specialized construction procedures; and
Site preparation and restoration.

## 10.1 COORDINATION OF DESIGN AND CONSTRUCTION

The designer should encourage communication with, and develop a rapport with, construction personnel by being supportive and cooperative when problems or disagreements occur. Communication should begin as early as possible in the planning and design stages. Ideally, the concept of the proposed work would be discussed with construction and operations personnel, followed by a review of plans and specifications prior to finalization. The formality of these actions can vary, but it is good practice to document the results in order to prevent future misunderstandings.

The designer should visit the job site occasionally when the work is going well, and frequently when it is not, not to infer a lack of trust in construction personnel, but simply to demonstrate an understanding of the importance of competent construction. These visits also familiarize the designer with construction practices, as well as informing construction personnel of the factors which went into the design of the work. They can then better distinguish between those changes in site conditions which are significant and those which are not. At the same time, construction personnel should be encouraged to notify the designer of significant site changes.

## 10.2 SPECIALIZED CONTRACT PROVISIONS

The following areas offer opportunities for insuring that contract provisions provide a framework for quality workmanship, fair and efficient contract administration, and a maintainable project:

Specifications and bid items;
Restrictions imposed by river conditions;
Preconstruction verification of design ;
Stone gradation and quality;
Specifications for commercial products; and
Documentation of as-built condition.

Specifications should also clearly address environmental aspects of the work, as discussed in 10.3.

### 10.2.1 SPECIFICATIONS AND BID ITEMS

The format and content of specifications for work to be contracted are usually dictated by organizational policy, with only the technical details left to the engineer's discretion. If that is not the case, then specifications can be developed by obtaining guide specifications, or specifications for a particular project similar to the one being designed, from an agency or firm accustomed to doing similar work in the region.

The following approach to bid items will expedite contract administration:

> Use lump sum bid items for elements of the work for which (1) reliable estimates of quantities can be prepared based on the advertisement for the work, and (2) quantities are not likely to be affected by changed site conditions, and (3) measurement and payment by unit price would be tedious. Mobilization and demobilization are commonly done this way, but other items such as site clearing, surface drainage work, and small items of excavation (such as for dike bankheads) can also be effectively treated this way.
>
> Use unit price bid items for work which may be significantly affected by changed site conditions, and for which the actual quantities can be measured easily. The unit of measurement should be the simplest one which is a reliable indicator of work performed, such as ton of stone, area of vegetative treatment, and linear measure of retard.

If more than one stabilization method or material would satisfactorily meet project requirements, and the comparative cost of the alternatives cannot be reliably estimated beforehand, then alternative bid items can be presented in the advertisement for bids, and the contract can be awarded for the alternative with the lowest bid price. For example, if either of two types of flexible mattress would be satisfactory, then the advertisement for bids could provide for either or both to be bid on, and the contract would be awarded for the least expensive one. This approach increases the design effort, since the specifications must address all alternatives to be bid on. It is most likely to be feasible when other design factors, such as geotechnical stabilization measures, beginning and ending points, and channel alignment, would be identical with either alternative. A similar approach is commonly used for vegetative treatment, with the date that the treatment is accomplished being the variable which determines the type of treatment.

If the rate of supply of materials to the site is the controlling factor in the time required to complete the work, then provision in the specifications to allow partial payment to the contractor for stockpiled materials can be helpful in completing the work without delay. This encourages the contractor to stockpile materials if that is required for timely completion, although the capital costs involved will presumably be reflected by an increase in the bid price. An alternative for speedy completion is to specify a compressed time allowance for completion, with the contractor being penalized financially for exceeding that time, unless unusual circumstances beyond the contractor's control were the cause. The risk in this approach is that legal and administrative constraints on determining and assessing the penalty for exceeding the time allowed for completion may prevent the penalty from being high enough to result in any significant acceleration of the contractor's operations.

### 10.2.2 RESTRICTIONS IMPOSED BY RIVER CONDITIONS

A generally accepted axiom of construction contracting is that only the end product should be specified, leaving operational procedures to the contractor. However, some river stabilization projects involve an exception, where it is advisable to impose operational restrictions on a contractor if river stages are temporarily too high for effective work. This may be necessary to insure that the contractor does not attempt work such as bank grading, placement of filter material, and placement of subaqueous stone when depth of water or flow velocity are too great for satisfactory results to be ensured. Since it is often impossible to precisely verify the in-place condition of subaqueous work, specifying the conditions under which the work must be done reduces the risk of an inadequate job.

The simplest way to impose such restrictions is to include information on river stages in the advertisement, and to specify any restrictions that will be imposed on the contractor's operation when river stage exceeds a given value. Information on river stages is usually provided to prospective bidders in the form of historical records at the nearest gaging station, accompanied by a caveat that historical stages do not necessarily represent the extremes which may be experienced during the course of the contract.

An alternative to that approach is to define restrictions on operations in terms of depth of water or current velocity at the jobsite. The risk in this approach is that it involves some conjecture even by the designer, and that bidders unfamiliar with site conditions will certainly not be able to precisely define the impact of such restrictions on their operation. Therefore, bidders may not have a common basis for preparing bids.

A third alternative presents the most flexible approach from an engineering standpoint, but is potentially the most troublesome from an administrative perspective. That is to state that operations will be temporarily suspended when, in the judgement of the contract administrator, river conditions preclude satisfactory execution of the work. If it becomes necessary to actually impose these undefined restrictions, claims by the contractor for additional payment may ensue, but that will likely be preferable to the alternative of continuing work under unsatisfactory conditions.

A related but separate issue concerns damage to partially completed work as a result of the river's flow. Specifications for river stabilization work sometimes include a definition of the responsibilities of both parties if “unusual” flows or “floods” damage the work in progress, but interpretation of these clauses often involves legal disagreements when such an event occurs, or is alleged by the contractor to have occurred. Contract clauses defining “changes in site conditions” versus “conditions that would have reasonably been anticipated,” and clauses concerning “acts of God” may enter the fray, with final resolution often resting on legal rather than engineering determinations.

### 10.2.3 PRECONSTRUCTION VERIFICATION OF DESIGN

If site conditions are likely to change between design and construction to such a degree that details of the work will require modification, provision should be made for this in the contract. An effective procedure is to advertise for bids based on general site conditions, showing sufficient design detail to allow confident bidding, and to state in the specifications that details of the work will be provided prior to notice to proceed with the work. The site can then be surveyed and inspected at the last moment that allows detailed plans for the work to be finalized and furnished to construction personnel on schedule. The possibility of a claim for changed site conditions still exists, but this procedure reduces that risk. It also provides for the most effective and efficient final design, and is especially useful in emergency situations or to meet a compressed schedule for project completion.

### 10.2.4 STONE GRADATION AND QUALITY

Appendix A contains detailed guidance for designing the gradation for stone armor for a specific site. However, it is not usually essential that precisely that gradation be specified. Stone of a similar and equally effective gradation may be commonly used in the area; if so, it can usually be obtained at a lower cost, and with a high probability of meeting the specification without intensive inspection. Even if the commonly used gradation requires a slightly greater blanket thickness total cost may still be less than if a slightly thinner blanket of a “new” gradation is specified, and the safety factor will be greater.

If standard specifications for stone gradation and quality do not exist within one's own organization, it is advisable to obtain guidance from large construction organizations in the region. Such guidance should include specification of stone quality, testing and inspection procedures, and a list of quarries known to be capable of producing acceptable stone, as well as standard gradations.

Verifying that the gradation and quality of stone produced for the project meets specifications can be difficult, because determining the precise gradation of quarried stone requires tedious and expensive handling of large quantities. Determining stone quality also requires sophisticated and expensive testing. These difficulties can be reduced by:

Specifying a standard gradation, as discussed above;

Minimizing the number of different gradations used in a contract;

Listing in the contract those quarries known to be capable of producing stone of acceptable quality, and requiring that the contractor produce certification from the quarry that the stone does meet specifications; and

Using personnel experienced in stone work for contract inspection.

### 10.2.5 SPECIFICATIONS FOR COMMERCIAL PRODUCTS

Section 6.5 discussed the use of manufacturers' recommendations in the design phase of a project. The specification phase often poses a dilemma - should the designer specify a particular trade name product and include the manufacturer's specifications for installation, or should the designer allow the use of any one of a group of similar products, all of which have similar installation specifications? The decision depends upon two factors:

Whether the work is to be advertised for bids, with the contractor furnishing the materials, or whether materials are instead to be purchased by one's own organization or the project sponsor. The significance of this factor is that if the contractor is to furnish the materials, then the specifications must not be subject to misinterpretation which might result in the use of unsuitable materials.

Organizational policy regarding the use of trade names in specifications as opposed to “generic” specifications. If policy allows, careful use of trade names eliminates ambiguity without requiring lengthy generic specifications. However, the potential exists for overlooking other suitable, and perhaps less costly, products. This difficulty can be addressed by using the phrase “or equal” to trade names. However, differences of opinion among engineering personnel, contract administration personnel, and the contractor as to the definition of “equal,” and the procedure for judging equality, sometimes results in significant misunderstandings.

In summary, the specification writer's goal is to ensure that essential aspects of the materials and the installation are satisfied, without unduly restricting choice of materials and equipment. Such restriction may result in protests by unsuccessful bidders and/or unnecessarily high project costs. The policy of the responsible organization will determine the most feasible specification structure to accomplish this.

### 10.2.6 DOCUMENTATION OF AS-BUILT CONDITION

Documentation of the as-built condition of a bank stabilization project is essential not only for quality control of the construction, but also for future inspection and maintenance of the work. The most thorough way to satisfy this need is to make a comprehensive survey of the entire project area immediately after construction, noting all features of the work on the survey. The survey would include the entire channel in the area of the work, and would extend far enough upstream and downstream to allow detection of channel migration or bed scour which might become a threat to the work. Documenting channel bed elevations in the vicinity of the work is especially critical, since toe scour is often the greatest threat to the work.

The survey which was used for detailed design will often satisfy this requirement if no significant channel changes occurred during construction. It can simply be amended to include details of the work and changes in site conditions during construction.

If a comprehensive survey is beyond the means of the project sponsor, and the stream is small enough to visually observe future changes that would threaten the work, it may be sufficient to simply retain the contract plans and specifications, supplemented by photographic documentation. Ground-level photographs from documented points as the construction proceeds are useful for contract inspection as well as for future monitoring of the work. Aerial views are particularly useful for future monitoring of major upstream and downstream changes. Making the photographs can be the duty of either the contractor, the inspection personnel, or the designer.

It may be expedient to specify in the contract that the contractor will furnish the capability to make the as-built survey, and any other surveys required during the course of the work. Surveys during the course of the work may be required for measurement of quantities, or for design adjustments if site conditions are unstable. If surveys by the contractor are to be used for measurement of pay quantities, it is good practice to insure that they are certified by a professional surveyor. Organizational policy may impose other restrictions as well.

## 10.3 ENVIRONMENTAL CONSIDERATIONS

The impact of construction operations on environmental quality is often temporary if the specifications include environmental features and the operations are well-managed. Since economics will often govern construction operations if left totally to the contractor, allowable methods of operation, or alternately, methods which are not permissible, should be clearly specified if environmental aspects are critical. Critical habitat areas, vegetation, or other sensitive areas not to be disturbed should be identified as being outside the construction right-of way. Environmental features of the work should be addressed in the specifications and during construction as thoroughly as engineering features of the work.

Control of surface water and erosion and sedimentation during construction is always good practice. The design of permanent measures was discussed in 6.4, but other, more temporary, measures may be a legal requirement for the political jurisdiction in which the work is being done. Minimizing site disturbance and revegetating areas disturbed by construction are always desirable measures, and are examples of measures that may be required by ordinance. Disposal of clearing debris and excavated material should be done in an environmentally sensitive manner.

Ecologically sensitive periods such as spawning and nesting should be considered in the timing of construction. This is particularly compelling if threatened or endangered species are involved. A potential difficulty is that nesting sites may not be known at the time a contract is issued, in which case the specifications should provide for a conflict resolution procedure that will be environmentally sound, but that is also fair to the contractor. A contract modification and extra payment may be necessary if construction operations are significantly disrupted by unforeseen circumstances. An example of a species-specific approach used by the U.S. Army Corps of Engineers on the Sacramento River, California, for sites at which bank stabilization work could not feasibly be performed before or after bank swallow nesting season, was to cover the bank with plastic sheeting prior to the nesting season. This forced the swallows to nest at other suitable sites not scheduled for bank protection.

It is advisable to consult with appropriate agencies, organizations, and environmental experts to define the construction practices that are required or recommended for a particular region.

## 10.4 SPECIALIZED CONSTRUCTION PROCEDURES

The impact of construction procedures on project economics and the selection of a bank stabilization method was discussed in 5.3. This section presents some specific construction approaches that may result in a more efficient and effective job. These approaches are discussed under the following headings:

Access for construction equipment;
Sequence of construction;
Subaqueous placement of stone or similar materials; and
Procedures for proprietary products.

### 10.4.1 ACCESS FOR CONSTRUCTION EQUIPMENT

On small streams, the choice of access is a matter of weighing the trafficability, environmental impacts, and real estate aspects of alternate routes. Fortunately, the most trafficable route often has the least environmental impact. In some cases, the streambed, especially sand and gravel bars, may be suitable for maneuvering of construction equipment. The environmental impact may be acceptable, especially if the need for clearing of vegetation along the streambank is reduced. On streams with high bed material transport, the next flow event will likely obliterate any traces of construction activity in the streambed. However, it is advisable to be prepared to provide alternative access, because rapid increases in streamflow or undetected areas of "quicksand" can be hazardous to streambed operations.

On navigable streams, the use of floating plant expedites construction and reduces costs and environmental impacts, and on larger streams is in fact the only feasible way to construct most works. If the stream is not navigable, but sufficient water for floatation exists at the worksite, portable barges can be used as access ways or working platforms for equipment and materials. This may allow the use of smaller machines than if all work is constructed with shore-based equipment, and will provide environmental benefits by reducing site disturbance from on-shore working and staging areas.

The procurement of rights of way should also consider the need for permanent access for monitoring and maintaining the completed work.

### 10.4.2 SEQUENCE OF CONSTRUCTION

(a) For projects which are not likely to have significant channel changes during construction, sequence of operations can be left to the discretion of construction personnel so that the most efficient procedure can be used.

However, a common practice is to specify that construction begin at the upstream end of each worksite. This may seem to be inconsistent with the principle stated in 6.1.1 that the downstream end of the work is usually the most vulnerable to damage. However, the apparent discrepancy is resolved by noting that vulnerability of the downstream end is a relatively long-term process, related to the opposite bar moving downvalley and encroaching into the channel. Conversely, a construction operation will be of relatively short duration, during which the attack on the downstream end is unlikely to increase. Thus, at least for armor protection, the tradition of beginning work at the upstream end probably stems from a fear that bank erosion and channel migration of the upstream portion of a bend while construction is underway might upset channel alignment more severely than similar erosion downstream during the work. For indirect protection, a more tangible case can be made for beginning upstream. Completed or partially completed indirect protection structures upstream often reduce streamflow attack on the structures downstream as they are being built, sometimes to the point of inducing deposition, thus reducing the quantity of materials required for the downstream structures. The result is a de facto “incremental construction” approach. However, significant amounts of induced deposition at the site of uncompleted structures downstream may dictate changes in design or construction procedures, which may in turn require extra contract administrative effort, but with the reward that total cost of the work is often reduced.

Theoretically, a case can be made for beginning construction at the point on the channel that maximum erosion is occurring, then completing construction as rapidly as possible in both directions. This is rarely done in practice because it requires two concurrent operations, which may limit the number of contractors capable of the work and increase the cost of the work. For emergency jobs, though, or situations where rapid erosion is occurring, such an approach may be advisable.

(b) In all cases, it is advisable to construct the toe protection component as early in the operation as is practicable.

(c) For projects involving large quantities of armor protection and a long construction period, it may be advisable to place the armor in strips proceeding up the bank slope, with the lowest strip following immediately behind the slope preparation. This reduces the probability of a rapid rise in water level overtopping and eroding the unarmored portion of the slope. A similar precaution is to limit the distance that slope preparation operations can precede completion of slope armoring. That limit may also be stated in terms of maximum allowable time interval between the two operations.

(d) The most economical procedure for construction of stone dikes in flowing water is to construct them in vertical increments, or "lifts," if a significant amount of flow will be intercepted by the structure during construction. If the dike is constructed to full grade in one lift, a significant scour hole is likely to develop ahead of stone placement, which will increase the amount of stone substantially over the original estimate.

Even if the lift procedure is used, some overrun due to scour and stone displacement will occur, and should be provided for in the estimate of required stone quantity. The amount of overrun will vary widely due to the following factors:

Amount of flow intercepted by the structure;
Erodibility of the bed material;
Rate of stone placement (rapid placement reduces overrun);
Gradation of the stone (larger stone reduces overrun); and
Height of the dike (higher structures increase overrun).

No quantitative guidance exists for estimating this overrun for a given situation; therefore, it is usually based on experience. As an example, for dikes on the lower Mississippi River, which are normally constructed in lifts of about 10 feet in height, common practice for estimating the overrun is to assume an average bed scour of 5 feet in front of stone placement in the initial lift only.

Placing a "blanket" of stone ahead of the placement of the initial lift will also reduce overrun due to scour. However, placement of stone in a blanket takes longer than placing an equivalent amount of stone in a "peaked" section within the ultimate dike cross-section. Therefore, the reduction in scour provided by the blanket is offset to some degree by the longer period of time required to complete the initial lift.

(e) For multi-stage construction of dikes, stone should be placed in the downstream part of the dike cross-section first, because a scour hole will occur downstream of the stone almost immediately as a "plunge pool" forms from flow over the stone. Placing subsequent lifts of stone in the upstream part of the dike cross-section will require less stone than if subsequent lifts were placed in the downstream scour hole.

(f) Alternatives available to the construction forces if specifications call for a dike crown width too narrow for the operation of construction equipment are to:

> Construct "turn-arounds," or maneuvering areas, by widening the crown at intervals along the dike. The expense of doing this will of course be reflected, at least indirectly, in the bid price for the work if the work is contracted out.
>
> Construct the top portion of the dike (that portion which is higher than the elevation at which there is a crown width sufficient for hauling and handling stone) as a separate operation, in which stone that was previously "stockpiled" alongside the dike, but outside the specified dike cross-section, is pulled up and into the design cross-section. The piece of equipment which performs this operation literally backs toward the end of the dike, finishing the dike as it proceeds. Some inefficiency is involved in this method, and some stone is inevitably left outside the design cross-section, especially if some of the stockpiled stone is underwater.

(g) Stone dikes can be successfully constructed even if the entire structure riverward of the bankline is underwater during construction, although careful control of the operation is necessary. Precise control of the profile and crown width of the finished structure is not feasible, but the structure will be functional nevertheless. An overrun in quantity is likely, because of less precise placement and increased stone displacement by the flow over the structure. This overrun will be offset in some cases by the relief provided by flow over the structure that would otherwise have been diverted around the end of the dike during construction, causing scour ahead of it, had it been above water during construction.

### 10.4.3 SUBAQUEOUS PLACEMENT OF STONE OR SIMILAR MATERIALS

Conventional draglines and bucket machines of all descriptions can be used successfully to place stone fill underwater. Bottom-dumping hopper barges can be used to increase rate of placement. This practice is more common in European practice than in the United States. Lacking such specialized equipment, ingenious use of available equipment, such as coal hoppers mounted between pontoon barges, can expedite the work.

Subaqueous stone paving or granular filters can be placed more efficiently by specifying the quantity in terms of weight or volume per unit of area covered rather than in terms of blanket thickness. In-place thickness is difficult to measure underwater, and adequate coverage can be obtained by careful placement of the material using a grid system and pre-placement allocation of the amount of material to be placed per grid unit (for example, 7 tons per 100 square feet, in lieu of a paving thickness of 15 inches). Regardless of the placement and measurement procedure, the specified quantity should be greater than that required for the design thickness in order to compensate for the uncertainties of subaqueous placement.

Likewise, a subaqueous longitudinal toe dike to provide launching stone for toe protection can be placed at a rate per linear distance along the bank rather than to a specified height or cross-section shape (for example, 2.1 tons per linear foot of bank in lieu of a peaked dike 5 feet high).

There are many ways to control the horizontal position of subaqueous placement operations once shoreline survey points are established. Buoys, driven piles, and moored or anchored barges are common practices. Sophisticated electronic survey techniques expedite these controls, and may eliminate the need for stationary markers completely in some cases.

### 10.4.4 PROCEDURES FOR PROPRIETARY PRODUCTS

Some proprietary products have specialized procedures and equipment associated with them. In some cases, those procedures and equipment may be necessary for satisfactory work; for example, restrictions on permissible height from which to drop stone on engineering fabric, or the use of frames to lift and place prefabricated mattresses. In other cases, they may simply expedite the work; for example, metal “templates” for filling gabions. Those which are necessary should be directed in the specifications. Regarding those procedures and equipment which merely expedite the work, construction personnel or prospective contractors should be made aware of them, with choice of use being their prerogative. This may be done by providing points of contact for manufacturers' representatives in the specifications. Some manufacturers will provide on-site assistance during construction.

## 10.5 SITE PREPARATION AND RESTORATION

Most aspects of site preparation and restoration are inherent in the topics discussed previously in this chapter. However, the following points are worthy of separate mention here.

Subgrade preparation is especially important for armor protection. Debris which interferes with placement of the armor or filter material must be removed. Freshly graded slopes are especially susceptible to erosion from surface drainage prior to placement of the armor, therefore preparing the site to control surface drainage may be critical.

Debris which would hinder proper construction of dikes or retards must be removed. It can sometimes be placed in a manner to enhance the work by providing additional indirect protection to the bank.

Restoring access routes to the original condition, but perhaps with additional erosion control measures, may be appropriate for some projects. For other projects, leaving them with improved trafficability may be preferable. The choice depends upon the potential future need for inspection and maintenance of the project, the landowners' desires, and environmental considerations. A factor for projects involving trenchfill or other protection methods which depend upon post-project erosion to a predetermined bankline is that hard-surfaced ramps or roads riverward of the structure sometimes resist erosion more than the adjacent natural bank material, perhaps to the detriment of obtaining the desired bankline in a timely manner. In any case, disposition of access roads or ramps should be clearly stated in the easements and in the specifications.

If the project involves stockpiling of stone or similar material prior to placement in the work, restoration of stockpile areas must be addressed. Depending upon the landuse, residual material may be a nuisance after construction, but completely removing it from the natural ground surface without disturbing topsoil and drainage is difficult. To prevent these problems, it may be appropriate to specify that such material be placed on a pad of natural material or on skids of metal, wood, or engineering fabric to prevent it from becoming buried in the existing topsoil. The stockpile area site can then easily be restored to natural conditions after construction.

# CHAPTER 11

# MONITORING AND MAINTENANCE OF STABILIZATION WORKS

This chapter presents concepts for effectively monitoring riverbank stabilization works, for determining the need for maintenance and repair, and for designing repairs if necessary. Previous chapters have discussed the characteristics of river behavior and the characteristics of the different types of stabilization work, and the application of those topics to the design of stabilization work. Those topics are equally applicable to monitoring and maintaining the work. This chapter does not present comprehensive details of maintenance and repair techniques for the many types of stabilization work, because to do so would be both tedious and redundant. Neither does this chapter address maintenance aspects of flood control projects, except where those aspects are directly related to bank stabilization works.

## 11.1 IMPORTANCE OF MONITORING AND MAINTENANCE

Monitoring and maintenance of bank stabilization works are essential in order to ensure successful performance over the lifespan of the project. Even properly designed works often require some maintenance eventually. Because of the dynamic nature of streams, a lack of maintenance often results in major failures, which become progressively more difficult and expensive to repair. Therefore, monitoring of stabilization works is more important than for structures in a static environment. Because critical components of the work are often underwater, thus not visible to simple observation, monitoring often involves significant but cost-effective effort and expense. Foresight is essential, because it is too late to begin an effective monitoring program once unforeseen damage requires major repair.

For works to be maintained by a governmental agency, there may be formal requirements, or at least guidance, for monitoring and maintenance. In the case of works to be maintained by a local sponsor, it may be necessary for the constructing agency to provide to the sponsor a manual describing monitoring and maintenance requirements. The applicability of, and the necessity for rigidly adhering to, these official requirements varies from project to project. However, the project manager should be aware that official requirements may exist, and should satisfy them as appropriate, keeping in mind that satisfying official requirements is a means to an end - maintaining functional work - and not an end in itself.

## 11.2 MONITORING

### 11.2.1 PURPOSES

An effective monitoring program accomplishes three things:

Detects the need for maintenance or repair in a timely fashion.

Provides a basis for designing repairs, if required.

Provides valuable insight into stream behavior and performance of stabilization work, which can be applied to future projects.

### 11.2.2 CONCEPTS

Ideally, the critical elements of river characteristics and their impact on the design of the protection work will have been considered and documented when the work was planned and designed. The most vulnerable aspects of the work will also have been identified as part of the planning and design process. Also, the as-built condition of the work will have been documented as part of the construction operation. If some of this information is missing or was never considered, monitoring activities will be compromised, but will still be worthwhile.

It is essential that monitoring detect any events or changes in stream characteristics or the condition of the work that exceed design assumptions. Although the stream itself is now the judge of whether the work is adequate, the fallacy in total reliance upon short-term observations of the performance of the work is that, in the absence of a rare event, design conditions will not be experienced until some future time. Therefore, all conclusions based on monitoring should be tempered by consideration of the actual conditions that the protection work has experienced.

Monitoring should include upstream and downstream conditions that may have future impacts upon the project. Examples are:

Changes in upstream channel alignment may threaten bank stabilization works downstream.

Channelization work may induce degradation upstream and may change hydraulic and geomorphic conditions downstream.

Significant changes in operating procedures of reservoirs upstream of the project site, or significant land use changes may change hydraulic and geotechnical parameters at the site.

Those topics have been addressed in previous chapters, but their relevancy does not diminish once a bank stabilization project has been completed.

The key word for monitoring is "comparability." For visual inspection, consider the season of the year and precedent flows. Observations made in late winter following high flows, with dormant vegetation, are not totally comparable to observations made in late summer. The perspective of sequential photographs should be comparable. Cross-sections and hydraulic data should be taken at comparable points and by comparable methods.

### 11.2.3 PRIMARY ELEMENTS

Available resources usually don't permit a monitoring program which obtains all possible or even all desirable data. The following discussion will re-emphasize some important concepts and identify those elements most critical to monitoring protection work. The primary elements are:

Site inspection;
Site surveys;
Geomorphic observations;
Hydrologic and hydraulic data;
Geotechnical data; and
Environmental aspects.

Within the scope of these primary elements, Mellema (1987) provides sound advice by stating that periodic fixed cross sections, gages to monitor river stages, and photographs are the basic monitoring tools. More sophisticated measures are often worthwhile, but must be weighed in the context of project requirements and long-term commitment of resources. Pickett and Brown (1977) provide an extensive listing of potential data collection activities, from which those appropriate for a specific project can be selected.

#### 11.2.3.1 Site Inspection

On-site inspection of the work itself is certainly the primary and most cost-effective monitoring element. Photographs made during these inspections are an invaluable supplement to written notes because they document conditions more reliably than memory or written observations, they can be referred to in the office while designing repairs if required, and they provide useful evidence of maintenance needs if competition with other projects for maintenance funds is a factor. Inspection should extend as far upstream and downstream of the project reach as is necessary to ensure that events elsewhere are not threatening the project.

If stream depths permit, wading and probing may reveal scour holes and displacement of armor materials at the toe of a revetment.

Interviews with local persons to obtain additional information, and encouraging reliable local observers to report significant observations to those responsible for monitoring, is often worthwhile.

#### 11.2.3.2 Site Surveys

Surveys are particularly important where the water level does not permit visual observation of the toe and lower bank slope, or where a primary concern is scour during high flows. Cross-sections of the bank, extending well past the toe, are the most reliable and useful type of survey, since toe deepening and undermining are the major factors in many failures. Handling of large amounts of data can be expedited by the use of computers.

Thalweg profiles require less effort than bank cross-sections, but are less reliable as to repeatability and accurate documentation of significant changes, therefore should be supplemented by cross-sections at representative locations.

Surveys taken during low flow periods may not adequately document conditions, particularly at the toe of the bank in bends, where scour and deposition may occur during high and low flows, respectively. Surveying during high flow periods is more difficult and expensive than during low flows, but may be advisable where adequacy of protection against toe scour is questionable.

The level of survey detail required for routine monitoring will usually be less than that which will be required if the need for repair work is detected. Monitoring surveys must be comparable to previous surveys, but surveys for design of repairs should be tailored to the specific one-time need. Routine monitoring surveys will, however, provide a basis for defining the additional level of detail required for surveys to be used in designing repairs.

#### 11.2.3.3 Geomorphic Observations

Changes in overall planform or local channel alignment can threaten the integrity of the work. Application of the principles discussed in 6.1 will determine when such threats exist. Also, the work itself may affect downstream conditions. The length of stream requiring geomorphic observation will depend upon the overall stream stability and the type and extent of protection work.

Aerial inspection is often a cost-effective technique for monitoring geomorphic conditions. Oblique and/or vertical aerial photography is particularly useful in documenting progressive changes. Vertical photography should provide sufficient overlap of adjacent photographs to allow at least a qualitative stereoscopic analysis of changes in bar heights and bank slope steepness. If detailed mapping is justified, vertical aerial photography can be geodetically controlled to permit mapping of the channel topography above water.

Significant changes in bed material composition in the project reach can affect the performance of the work, either favorably or unfavorably. For example, an increase in the amount or size of gravel in the bed may limit bed scour, but may also cause changes in planform which could adversely affect the work. Changes in bed material can be monitored either informally by detailed visual observation and photographic documentation, or through a formally designed sampling and analysis program. A formal program is not likely to be cost-effective solely for monitoring the protection work itself, but may sometimes be justified as part of a broader study.

### 11.2.3.4 Hydraulic Data

Initial collection and analysis of essential hydraulic data will presumably have been done in planning and design of the protection work. Hydraulic monitoring should concentrate on those parameters which were identified during planning and design as being most critical to the adequacy of the work. Over the short-term, routine monitoring of hydraulic parameters will usually not provide a basis for changing design criteria that are based on longterm probabilities, but it may provide an indication of whether or not design assumptions were reasonable.

Velocity is usually the most critical element for riprap protection. Also, if the alignment of flow into the protection work is changing significantly, the effect on the curvature correction factor for riprap design velocity should be evaluated.

Hydraulic data in the form of long-term stage-discharge relationships can indicate whether streambed degradation is cause for concern. These observations are best made at stations downstream of the project, so that degradation can be detected in time to reinforce the toe of the work or take other remedial action if necessary.

### 11.2.3.5 Geotechnical Data

The degree of geotechnical monitoring required is highly site-specific. A simple rule is that if critical geotechnical parameters cannot be monitored visually, then sophisticated devices such as piezometers and slope indicators may be required, especially if there was some degree of uncertainty in the original design, or if the consequences of failure would be high. In other cases, a visual inspection for proper function of internal drains, surface indications of slope settlement, and similar items may be sufficient.

#### 11.2.3.6 Environmental Aspects

The listing of this element last does not detract from its importance. It simply acknowledges that the project must perform its bank stabilization function adequately, else environmental function will likely be compromised as well. Also, some of the engineering elements just discussed, particularly geomorphology and hydraulics, have environmental implications as well.

The environmental parameters which should be monitored can be defined by reviewing the environmental factors which were pertinent to the selection of the protection method. Environmental specialists should be involved in the monitoring as appropriate.

### 11.2.4 FREQUENCY OF MONITORING

The first few years after construction and the first major flood flow are the two critical periods in the life of bank protection works. Monitoring should be intensified during these periods. Although events such as toe scour and scour at termination points and bank heads may not fully develop during this period, major problem areas will usually become evident, and will help define the required monitoring intensity for the long-term. As a minimum, inspect the work immediately following the high flow season. If a need for immediate reconstruction or repairs is discovered, then the process of design and construction can be completed before the next high flow season.

During these critical periods, an additional inspection during low water levels in late summer will allow an evaluation of the portion of the work that may have been underwater during earlier inspections, as well as an evaluation of vegetative growth during the summer.

Problems with gradual deterioration of structural components does not carry the urgency of major changes in channel geometry or alignment, or major flaws in the initial design of the work. Therefore, any reasonable interval of monitoring will suffice for that element.

Beyond these general guidelines, frequency of monitoring is determined either by regulation) or by engineering judgement, whichever results in the most frequent monitoring. Engineering judgement should take into account the safety factor of the original design and the consequences of failure (see 6.6), the severity of hydraulic conditions, and the degree of geomorphic instability of the stream. Monitoring of specific conditions such as channel velocity, waves, or ice forces should be timed to coincide with the occurrence of critical events if possible.

### 11.2.5 PERSONNEL

Ideally, each project will have a single person who is responsible for the technical aspects of its monitoring, and for providing evaluation and recommendations to decision-makers, although other persons will be involved as appropriate for specialized technical input, and for training and continuity in the event of personnel changes.

Agency regulations may specify the personnel to be involved in monitoring, particularly if the bank protection work is part of a broader project purpose, such as flood control. However, even if such guidance exists for a particular project, it may be advisable to supplement those required personnel with others who were involved in the design of the work, such as environmental specialists, vegetative experts, or others. The cost of involving such specialists will be small compared to the potential benefits.

If circumstances prevent the close involvement of those who planned and designed the work, it is even more important that the non-technical personnel who are responsible for monitoring and maintenance be involved early in the planning and design process. In addition to perhaps having provided useful input at that point, they will also be better prepared to later monitor the work.

### 11.2.6 POINTS REQUIRING SPECIAL ATTENTION

Particular attention must be paid to two areas at which scour or failures can quickly lead to the work becoming ineffective or requiring extensive rehabilitation:

> The toe of the protection.
>
> Bank connections at the upstream and downstream ends of the protection, or in the case of indirect protection, all bank connections.

Other important items to be monitored will vary with the type of protection and stream characteristics. Examples are:

> Interior drainage components of impermeable revetments. Evidence of voids forming underneath rigid armor should be especially noted.
>
> Upper bank slope where structural components transition to the natural bank material or vegetation, including areas where overbank drainage returns to the stream. Flood flows may have scoured or otherwise disturbed this zone. Although this may not result in major failure of the work, it can generate considerable concern for the landowner or sponsor.

Displacement or separation of armor material which exposes erodible bank material. Such displacement can be due to hydraulic forces, to geotechnical factors such as slope settlement or piping, or to vegetative disruption.

Gradual deterioration of mattress elements or retard and dike components.

Condition of planted and volunteer vegetation.

### 11.2.7 LEVELS OF MONITORING EFFORTS

#### 11.2.7.1 Level 1 Monitoring

Level 1 monitoring effort consists of a field reconnaissance and visual observation of the site, and a written report detailing the present conditions found at the site. This written report can also include a comparison between the present existing conditions and previous findings recorded during other site visits or supplemental data from aerial photographs or other sources and analysis of future stream movement and behavior (down valley migration, degradation, etc.). If an existing streambank protection project is within the study reach, an analysis of the performance of this project can also be included in this report.

#### 11.2.7.2 Level 2 Monitoring

Level 2 monitoring effort consists of all activities performed during a Level 1 monitoring effort plus a permanent photographic and/or videotape record of the project area. Photos should be shot from fixed and marked locations so that comparisons can be made over time. All photos should be indexed and accompanied by well written descriptions of the contents of the photographs plus the location where the photo was shot.

#### 11.2.7.3 Level 3 Monitoring

Level 3 monitoring effort consists of all activities performed during a Level 2 monitoring effort plus some physical measurements of the site (possibly using the typical low-flow water surface elevation as a datum). These measurements would typically be limited to delineation and locations of active scour areas, maximum scour depths in the stream, and scour depths near or at the toe of existing protection works.

#### 11.2.7.4 Level 4 Monitoring

A Level 4 monitoring consists of all activities associated with a Level 3 monitoring effort plus a comprehensive survey of the reach of stream in the immediate area of the protection works. This survey can also include a less intensive survey of the general planform and channel depths of the crossing and bend immediately upstream of the study reach if stream migration is a concern. If degradation is a concern, cross-section measurements at a convenient location downstream of the study site (a bridge for example) may be recorded.

#### 11.2.7.5 Level 5 Monitoring

A Level 5 monitoring effort consists of all activities performed during a Level 4 monitoring effort plus additional data on bed material size and gradation, water quality, roughness, fish habitat, and biomass analysis, etc.

#### 11.2.7.6 Pulsed Monitoring

A "pulsed" monitoring system is where a project or reach of stream is monitored on a long-term schedule with varying levels of effort. In most cases a pulsed effort will provide sufficient data and at the same time meet economic and time constraints.

An example of a pulsed monitoring effort would be to comprehensively survey and intensely monitor (Level 4 or 5 effort) a site annually for two to three years following construction, followed by a less intensive monitoring effort (Level l or 2) at the same frequency and after major flood events.

## 11.3 MAINTENANCE

### 11.3.1 DETERMINATION OF NEED

Two alternative approaches to determining the need for major maintenance (sometimes called "repairs" or "reinforcement") seem on the surface to be mutually exclusive:

Take action at the first indication of a threat to the work

Take no action until major maintenance appears inevitable.

When carried to extremes, these approaches are in fact contradictory. However, an effective compromise can be reached through the concept of "functionality," in which the engineer considers the consequences of failure, probable trends in stream behavior, type of protection (i.e., ability to function even if not intact), and availability of funds. Streambank protection works do not have to be in pristine condition to be effective. What appears to be damage may actually be minor and non-progressive "adjustment."

This contradiction is typified by toe deepening, which often requires no action, yet at the same time is a common cause of major failure. Adequacy of the original design to accommodate toe scour (see 6.3) will determine if maintenance or reinforcement of the toe is required.

Other examples where maintenance may not be required are:

Loss of stone at the riverward end and along the downstream side of stone dikes. This is to be expected as scour holes develop to an equilibrium condition, and should have been allowed for in the original design (see 8.1).

Settlement of flexible mattresses at the toe of the bank slope. Again, this is a common occurrence and is not cause for alarm unless the original design (see 7.4) did not adequately provide for it.

“Chipping” or “shelving” at the top of the bank slope where armor protection terminates below top bank, or within indirect protection structures. This condition may stabilize naturally as the slope flattens and as vegetation becomes established (see 6.2.5).

Minor, long-term deterioration of structural components, if stone or other permanent toe protection is present, and if vegetation is becoming well established on the slope or within the dike or retard field.

Maintenance requirements for vegetative components of stabilization is highly regional and site-specific. Substantial effort may be involved for some projects. The transition from the construction phase into the maintenance phase is often ill-defined, and may in fact depend more upon administrative distinctions for funding purposes than upon engineering and biological judgements. The situation is further complicated by vegetation being a basic component of the work in some cases, but perversely posing a threat to the work's integrity in other cases. The key is a judgmental determination of whether the vegetation adversely affects the “functionality” of the work.

### 11.3.2 DETERMINATION OF METHOD OF REPAIR

As with the determination of the need for maintenance, there are seemingly contradictory alternative approaches to selecting a method of maintenance if part of the work fails:

Restore the work using the original approach;

Increase the safety factor of the work by using the same type of protection with more severe design criteria; and

Select a different method of stabilization for the repairs.

Selection of the correct approach, or an effective compromise, is a matter of engineering judgement focused on the question "Why did the work fail?".

If the need for maintenance is due to *conditions not expected to recur*, in which neither the type of protection nor the basic design criteria were at fault, then the original material may be adequate for repair. For example, bare areas on the bank slope caused by differential slope settlement can be rearmored with the original material of the same size and thickness, if the subgrade has stabilized.

If the type of protection is still deemed suitable, but the *original design criteria* were apparently *not conservative enough*, then the same method can be used, but with more conservative design criteria. For example, stone armor displaced by hydraulic forces can be replaced with stone of a greater size and layer thickness.

However, if *the method itself appears at fault* because of unforeseen circumstances, consider the use of a different technique or material for repairs. For example, if toe scour adjacent to a rigid armor or flexible mattress revetment exceeds the capacity of those materials to accommodate the scour, then consideration should be given to using stone for repair of the toe protection.

The great variation in protection methods and stream characteristics makes it infeasible to further discuss details of maintenance techniques. However, there are two materials which, between them, are useful over such a wide range of conditions that they merit special mention: stone and vegetation.

> Stone is often the material of choice because it can be used in almost any maintenance situation. In particular, damaged work is often characterized by irregular contours and the possibility of further scour or settlement. Stone's self-adjustment ability is well adapted to this situation.
>
> Vegetation is well suited for marginal situations where bank erodibility is low, and no emergency exists, but where taking no remedial action would lead to eventual failure. Vegetation is often inexpensive yet effective where it is supported by adjacent structural protection. Planting can be done at the optimum season, since no emergency exists.

### 11.3.3 OTHER CONSIDERATIONS

Failures in streambank protection work is occasionally so extensive that major reconstruction is necessary. Determining *responsibility* for, and funding of, that work may become complicated. Whether repairs should be done under maintenance authority and funding, or construction, or "rehabilitation," or some other authority, will depend upon the authorities applicable to the project, and the nature of the failure and the proposed remedial work.

*Rights of way* may be a problem in monitoring and maintenance. Ideally, adequate arrangements will have been made as part of the initial construction rights of way. If not, when the need for rights of way for monitoring and maintenance does arise, consideration should be given to obtaining it for the life of the project, as least for monitoring purposes. The practicality of doing so will depend upon such factors as project authorities, land use, attitude of the affected landowners, and the nature of easement required. Rights of way at the worksite itself is not usually a problem, since maintenance is to the landowner's best interests. However, acquiring access from other landowners is more likely to be difficult.

Repair work may need to include the removal of failed portions of the protection work which might be a *hazard* if left in place . Two potential hazards are:

> Injury to persons or animals traversing the bank. Examples of situations which may cause injury are: jagged broken piling; loose wires or cables in flexible mattress; severe irregularities in the bank caused by scour, settlement, or return flows, particularly if obscured by vegetation.
>
> Timber components of failed indirect protection structures being carried downstream by the flow to lodge on bridges or other structures in the stream.

*Zoning* regulations to prevent the public from building structures too close to the protection work or the stream bank may be advisable. This management practice was adopted on the Willamette River in Oregon.

# CHAPTER 12

# GRADE STABILIZATION

## 12.1 GRADE CONTROL CONCEPTS

Implementation of bank stabilization measures without proper consideration of the stability of the bed can result in costly maintenance problems and failure of structures. For this reason, it is essential to consider the stability of the bed as part of any bank stabilization scheme. Bank stabilization measures are generally appropriate solutions to local instability problems, such as erosion in bendways. However, when system-wide channel degradation exists, a more comprehensive treatment plan, which usually involves some form of grade control, must be implemented.

In the widest sense, the term “grade control” can be applied to any alteration in the watershed which provides stability to the streambed. By far the most common method of establishing grade control is the construction of in-channel grade control structures. There are basically two types of grade control structures. One type of 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. Lane’s relation would illustrate the situation by $QS^+ \% \, Q_sD_{50}{}^+$, where the increased slope ($S^+$) of the degradational reach would be offset by an increase in the bed material size ($D_{50}{}^+$). For this discussion, this will be referred to as a “Bed Control Structure.” The other type of structure is designed to function by reducing the energy slope along the degradational zone to the point that the stream is no longer capable of scouring the bed ($QS^- \% \, Q_sD_{50}$). This will be referred to as a “Hydraulic Control Structure.” The distinction between the processes by which these structures operate is important whenever grade control structures are considered.

Because of the complex hydraulic behavior of grade control structures, it is difficult to develop an “ideal” classification scheme that will apply without exception to all situations. For many situations, the classification of a structure as either a bed control structure or hydraulic control structure is readily apparent. However, there may be circumstances where a distinct classification of a structure as strictly a bed control or hydraulic control structure may be less evident and, in many cases, the structure may actually have characteristics of both. It also must be recognized that the hydraulic performance and, therefore, the classification of the structure, can vary with time and discharge. This can occur within a single hydrograph or over a period of years as a result of upstream or downstream channel changes.

## 12.2 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 the stream, along with the elevation and dimensions of 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 siting conditions for grade control structures. In practice, the hydraulic considerations must be integrated with a host of other factors, which 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 discussed in the following sections.

### 12.2.1 HYDRAULIC CONSIDERATIONS

One of the most important steps in the siting of a grade control structure or a series of structures is the determination of the anticipated drop at the structure. This requires some knowledge of the ultimate channel morphology, both upstream and downstream of the structure which involves assessment of sediment transport and channel 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 that the optimum spacing of structures is such 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 (Figure 12.1). Figure 12.1 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:

$$H = (S_o - S_f)x \tag{12.1}$$

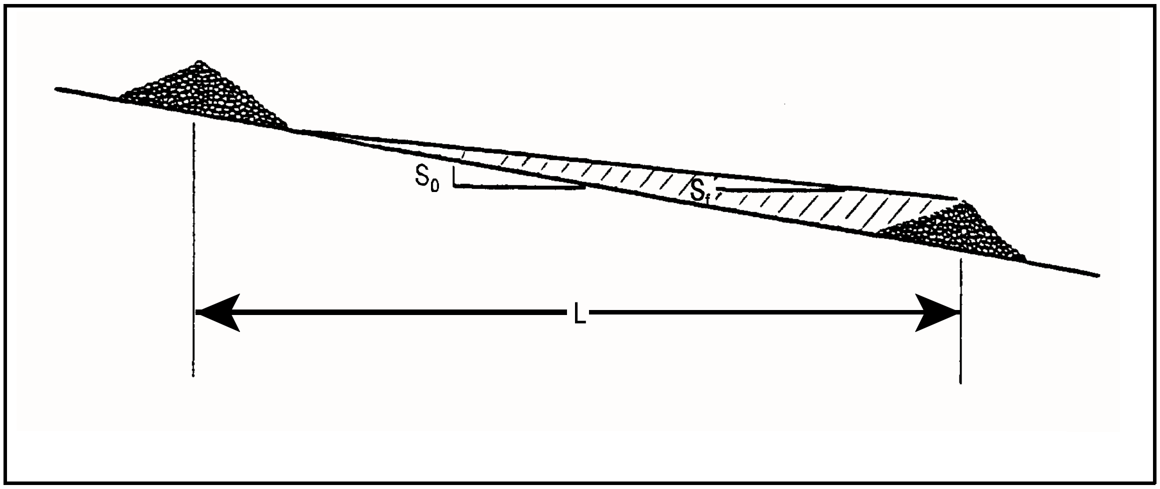



Figure 12.1 Spacing of Grade Control Structure (adapted from Mussetter, 1982)

where H is the amount of drop to be removed from the reach, $S_o$ is the original bed slope, $S_f$ is the final, or equilibrium slope, and x is the length of the reach (Goitom and Zeller, 1989). The number of structures (N) required for a given reach can then be determined by:

$$N = H/h \tag{12.2}$$

where h is the selected drop height of the structure.

It follows from Equation (12.1) that one of the most important factors 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. Failure to properly define the equilibrium slope can lead to costly, overly conservative designs, or inadequate design resulting in continued maintenance problems and possible complete failure of the structures.

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, etc.), 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.

There are many different methods for determining the equilibrium slope in a channel. These can range from detailed sediment transport modeling to less elaborate procedures involving empirical or process-based relationships such as regime analysis, tractive stress, or minimum permissible velocity (see 3.1.3). 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 upon several factors such as the level of study (reconnaissance or detail design), availability and reliability of data, project objectives, and time and cost constraints.

### 12.2.2 GEOTECHNICAL CONSIDERATIONS

The above discussion focused only on the hydraulic aspects of siting grade control structures. However, 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 channel degradation has caused, or is anticipated to cause, severe bank instability due to exceedance of the critical bank height (Thorne and Osman, 1988). When this occurs, bank instability may be widespread throughout the system rather than restricted to the concave banks in bendways. Traditional bank stabilization measures may not be feasible in situations where system-wide bank instabilities exist. In these instances, grade control may be the more appropriate solution.

Grade control structures can enhance the bank stability of a channel 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 advantages with respect to bank stability: (1) bank heights are reduced due to sediment deposition, which increases the stability of the banks with regard to mass failure; and (2) by creating a backwater situation, velocities and scouring potential are reduced, which reduces or eliminates the severity and extent of basal cleanout of the failed bank material, thereby promoting self-healing of the banks.

### 12.2.3 FLOOD CONTROL IMPACTS

Channel improvements for flood control and channel stability often appear to be mutually exclusive objectives. For this reason, it is important to ensure that any increased post-project flood potential is identified. This is particularly important when hydraulic control structures are considered. In these instances 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, then the frequency and duration of overbank flows may be impacted. When this occurs, the impacts must be quantified and appropriate provisions such as acquiring flowage easements or modifying structure plans should be implemented.

Another factor that must be considered when siting grade control structures is the safe return of overbank flows back into the channel. This is particularly a problem when the flows are out of bank upstream of the structure but still within bank downstream. The resulting head differential can cause damage to the structure as well as severe erosion of the channel banks depending upon where the flow re-enters the channel. 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 structures exerts hydraulic control throughout a wider range of flows including overbank, then 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 which 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 re-enter the channel without causing significant damage (Hite and Pickering, 1982).

### 12.2.4 ENVIRONMENTAL CONSIDERATIONS

The key phrase in water resources management today is “sustainable development” which simply means that projects must work in harmony with the natural system to meet the needs of the present without compromising the ability of future generations to meet their needs. Engineers and geomorphologist are responding to this challenge by trying to develop new and innovative methods for incorporating environmental features into channel 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 man-made pools below grade control structures in north Mississippi. They concluded that although there was greater species diversity in the natural pools, there was increased growth of game fish and a larger percentage of harvestable-size fish in the man-made pools. They also observed that the man-made pools provided greater stability of reproductive habitat. Shields 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. She observed that following construction of a series of bed sills, there was a significant increase in the density of trout. The increase in trout density was attributed to the accumulation of gravel between the sills which improved the spawning habitat for various species of trout.

Perhaps the most serious negative environmental impact of grade control structures is the 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, in situations where structures are impassable, and where the migration of fish is an important concern, openings, fish ladders, or other passageways must be incorporated into the design of the structure to address the fish movement problems (Nunnally and Shields, 1985). The various methods of accomplishing fish movement through structures are not discussed here. Interested readers are referred to Nunnally and Shields (1985), Clay (1961), and Smith (1985) for a more detailed discussion.

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, there is very little published guidance concerning the incorporation of environmental features into the design of grade control structures. One source of useful information can be found in the following technical reports published by the Environmental Laboratory of the Corps of Engineers, WES (Shields and Palermo, 1982; Henderson and Shields, 1984; and Nunnally and Shields, 1985).

### 12.2.5 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 and thereby provide 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 restraints, to ensure protection of existing structures.

It must also be recognized that grade control structures can have adverse as well as beneficial effects on existing structures. This is a concern upstream of hydraulic control structures due to the potential for increased stages and sediment deposition. In these instances, 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 engineer should take advantage of any existing structures which may already be providing some measure of grade control. This usually involves culverts or other structures that provide a non-erodible surface across the streambed. Unfortunately, these structures are usually not initially designed to accommodate any significant bed lowering and, therefore, can not 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 have several bridges that are in need of repair or replacement. In these situations 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, then it may be possible to construct the grade control structure concurrently with the road crossing (Figure 12.2).

### 12.2.6 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

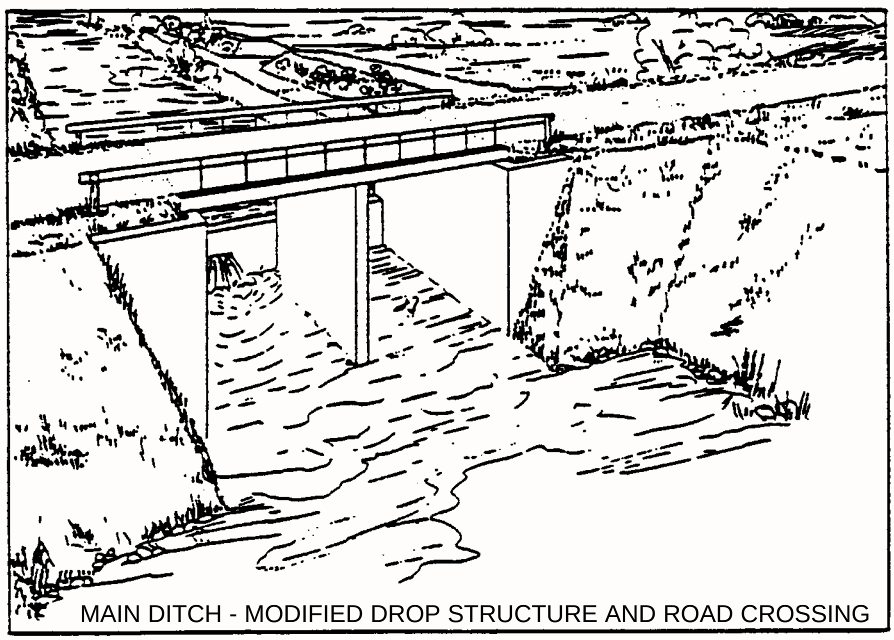



Figure 12.2 Combination Grade Control Structure and Road Crossing (adapted from U.S. Soil Conservation Service, 1976)

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 the case 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, road side ditches, or other sources often play an important part in the siting of grade control structures. Failure to provide protection from local drainage can result in severe damage to a structure (U.S. Army Corps of Engineers, 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. In these instances, the local drainage should either be re-directed away from the structure or incorporated into the structure design in such a manner that there will be no damage to the structure.

### 12.2.7 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 will vary from site to site. In some instances the effects of grade control structures on sediment loading may be so small that downstream degradational problems may not be encountered. However, in some situations such as when a series of hydraulic control structures is planned, the cumulative effects of sediment trapping may become significant. In these instances, it may be necessary to modify the plan to reduce the amount of sediment being trapped or to consider placing additional grade control structures in the downstream reach to protect against the induced degradation.

### 12.2.8 GEOLOGIC CONTROLS

Geologic controls often provide grade control in a similar manner to a bed control structure. In some cases a grade control structure can actually be eliminated from the plan if an existing geologic control can be utilized to provide a similar level of bed stability. However, caution must always be used when relying on geologic outcrops to provide long-term grade control. In situations 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 or flanked during the project life.

### 12.2.9 EFFECTS ON TRIBUTARIES

The effect of main stem structures on tributaries should be considered when siting grade control structures. As degradation on a main stem channel migrates upstream it may branch up into the tributaries. Therefore, the siting of grade control structures should consider effects on 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.

### 12.2.10 SUMMARY

The above discussion illustrates that the siting of grade control structures is not simply a hydraulic exercise. Rather, there are many other factors that must be included in the design process. For any specific situation, some or all of the factors discussed in this section may be critical elements in the final siting of grade control structures. It is recognized that this does not represent an all inclusive list since there may be other factors not discussed here that may be locally important. For example, in some cases, maintenance requirements, debris passage, ice conditions, or safety considerations may be controlling factors. Consequently, there is no definitive “cookbook” procedure for siting grade control structures that can be applied universally. Rather, each situation must be assessed on an individual basis.

## 12.3 TYPES OF GRADE CONTROL STRUCTURES

There are certain features which are common to most grade control structures. These include a control section for accomplishing the grade change, a section for energy dissipation, and protection of the upstream and downstream approaches. However, there is considerable variation in the design of these features. For example, a grade control structure may be constructed of riprap, concrete, sheet piling, treated lumber, soil cement, gabions, compacted earth fill, or other locally available material. Also, the shape (sloping or vertical drop) and dimensions of the structure can vary significantly, as can the various appurtenances (baffle plates, end sills, etc.). The applicability of a particular type of structure to any given situation depends upon a number of factors such as: hydrologic conditions, sediment size and loading, channel morphology, floodplain and valley characteristics, availability of construction materials, project objectives, and time and funding constraints. The successful use of a particular type of structure in one situation does not necessarily ensure it will be effective in another. Some of the more common types of grade control structures used in a variety of situations are discussed in the following sections. For more information on various structure designs, the reader is referred to Neilson et al. (1991), which provides a comprehensive international literature review on grade control structures with an annotated bibliography.

### 12.3.1 SIMPLE BED CONTROL STRUCTURES

Perhaps the simplest form of a grade control structure consists of dumping rock, concrete rubble, or some other locally available non-erodible material across the channel to form a hard point. These structures are often referred to as rock sills, or bed sills. These type of structures are generally most effective in small stream applications and where the drop heights are generally less than about 2 to 3 feet. A series of rock sills, each creating a head loss of about two feet was used successfully on the Gering Drain in Nebraska (Stufft, 1965). The design concept presented by Whitaker and Jaggi (1986) for stabilizing the streambed with a series of rock sills is shown in Figure 12.3. The sills in Figure 12.3 are classic bed control structures which are simply acting as hard points to resist the erosion of the streambed.

Construction of bed sills is sometimes accomplished by simply placing the rock along the streambed to act as a hard point to resist the erosive forces of 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 sufficient volume of non-erodible material to resist the general bed degradation, as well as the local scour at the structure. This is illustrated in Figures 12.4a and 12.4b which shows a riprap grade control structure designed to resist both the general bed degradation of the approaching knickpoint as well as any local scour that may be generated at the structure. In this instance, the riprap section must have sufficient mass to launch with an acceptable thickness to the anticipated scour hole depth.

### 12.3.2 STRUCTURES WITH WATER CUTOFF

One problem often encountered with the above structures is the displacement of rock (or rubble, etc.) due to the seepage flow around and beneath the structure. This is particularly a problem when the bed of the channel 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. This type of water barrier is illustrated in Figures 12.5a and 12.5b. One problem with this type of barrier is its longevity due to susceptibility to erosion. This problem can be avoided by using concrete or sheet piling for the cutoff wall. The conceptual design of a riprap grade control structure with a sheet pile cutoff wall is shown in Figures 12.6a and 12.6b. In the case of the sloping riprap drop structures used by the Denver Urban Drainage and Flood Control District, an impervious clay fill is used in conjunction with a lateral cutoff wall (McLaughlin Water Engineers, Ltd., 1986). This design is illustrated in Figure 12.7.

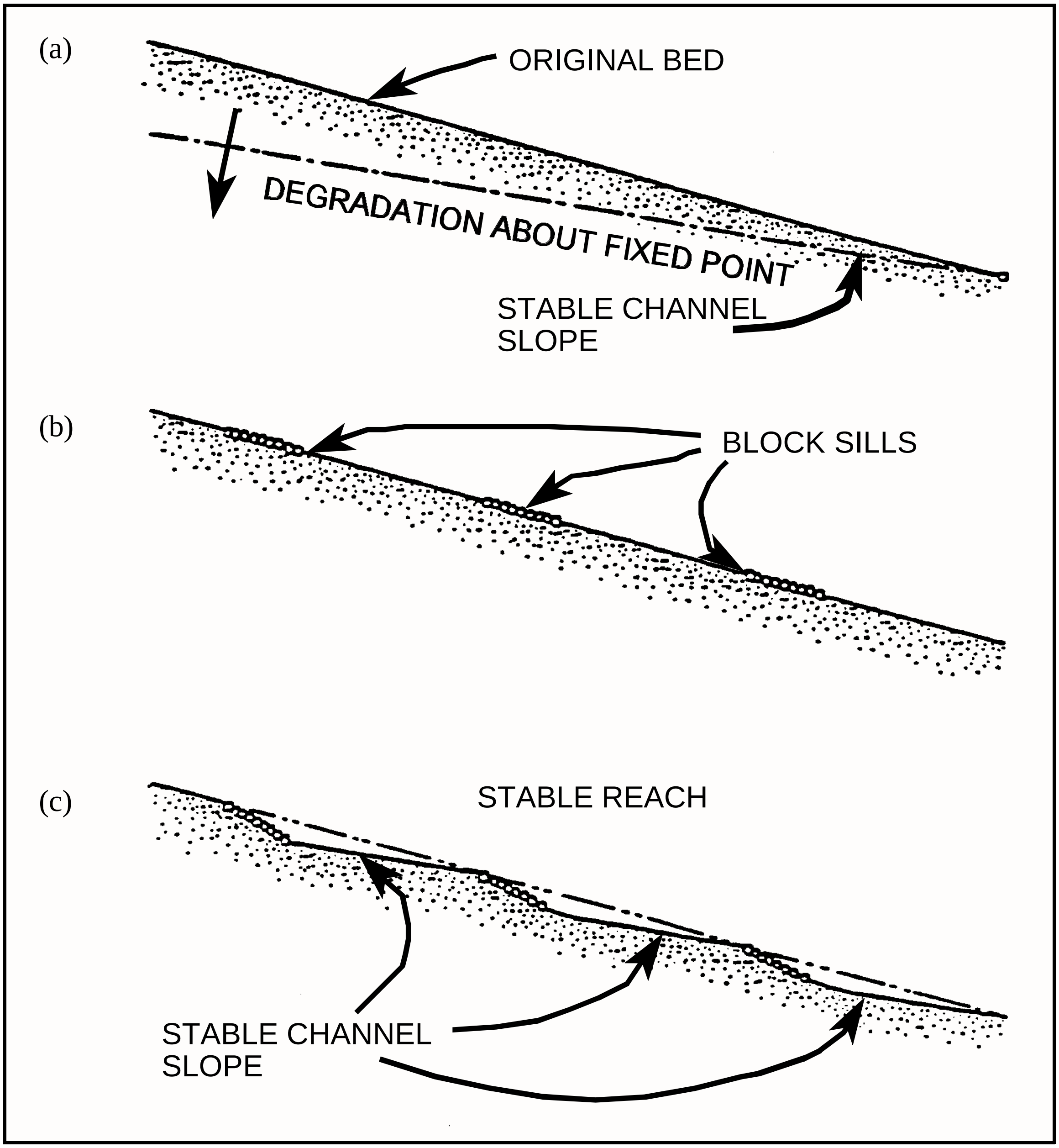
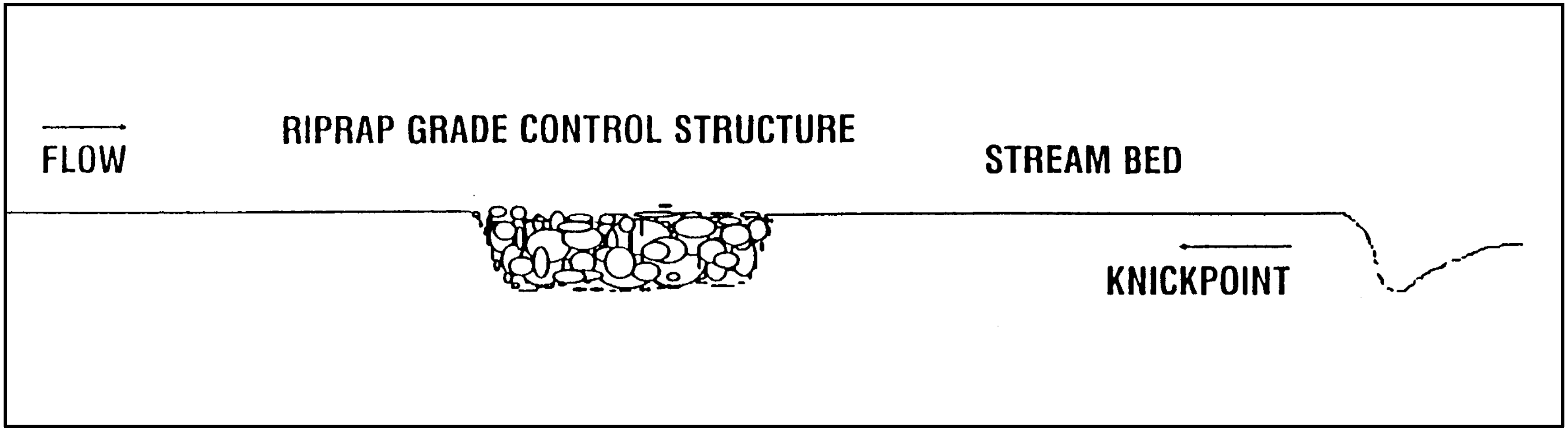
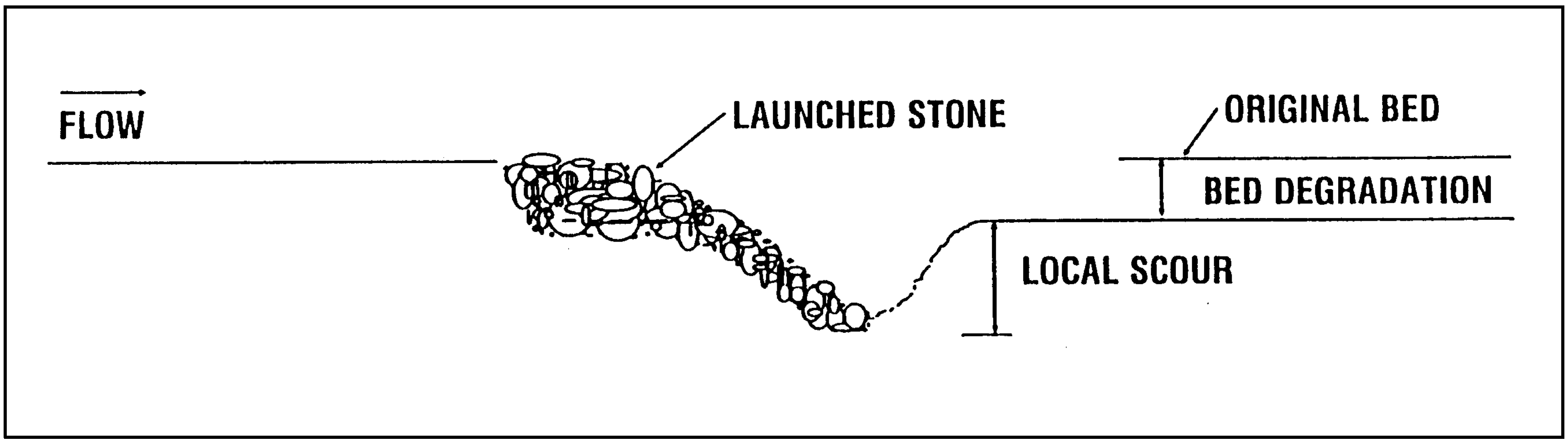


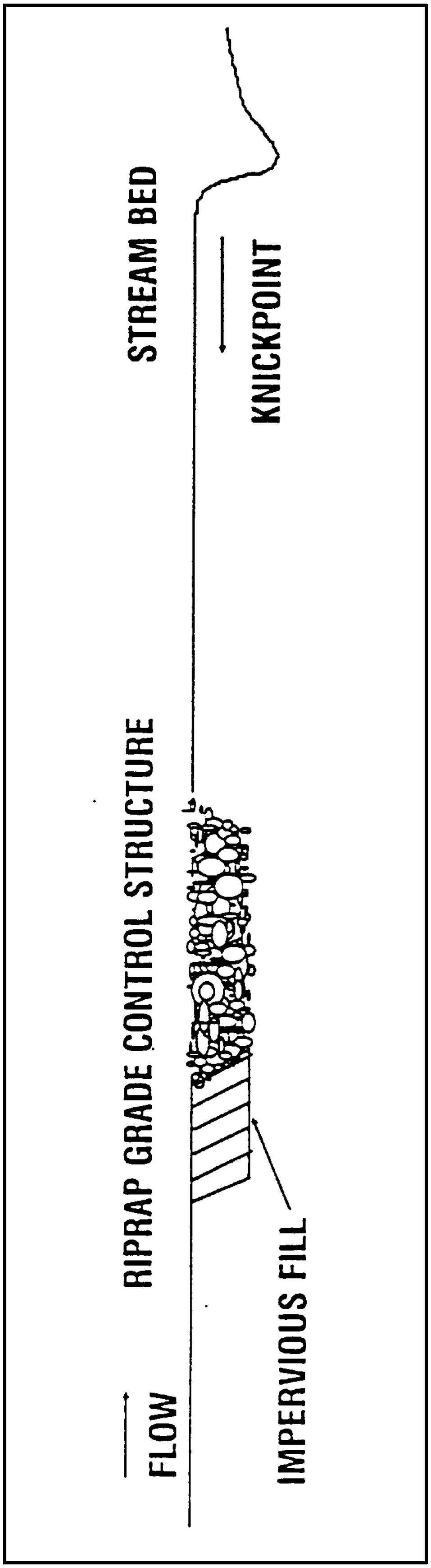

Figure 12.3 Channel Stabilization with Rock Sills (adapted from Whitaker and Jaggi, 1986)



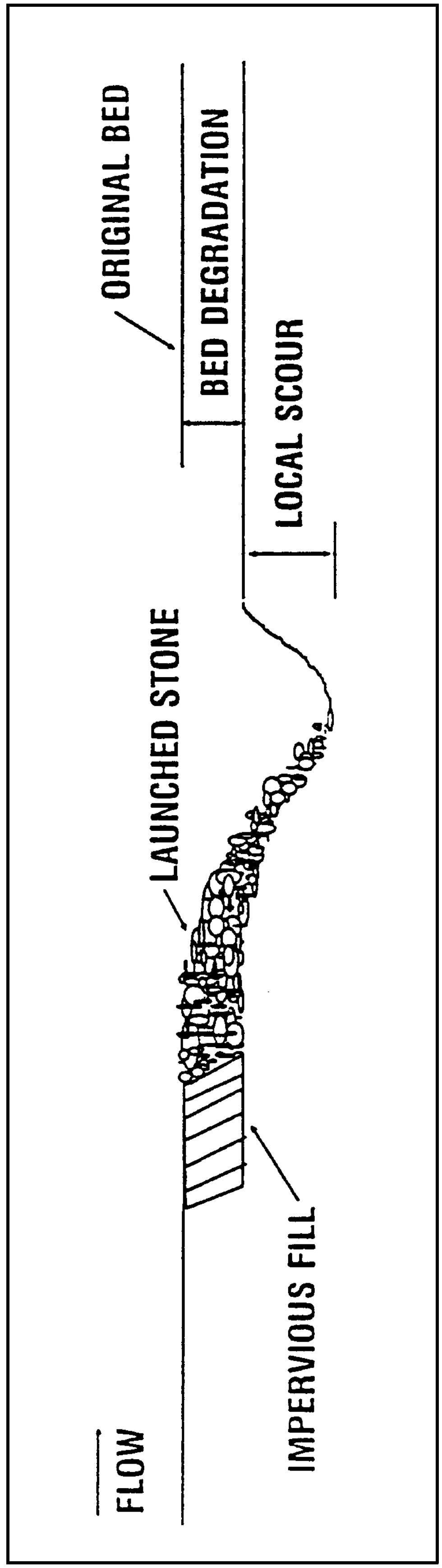

Figures 12.4a  As Built Riprap Grade Control Structure with Sufficient Launch Stone to Handle Anticipated Scour



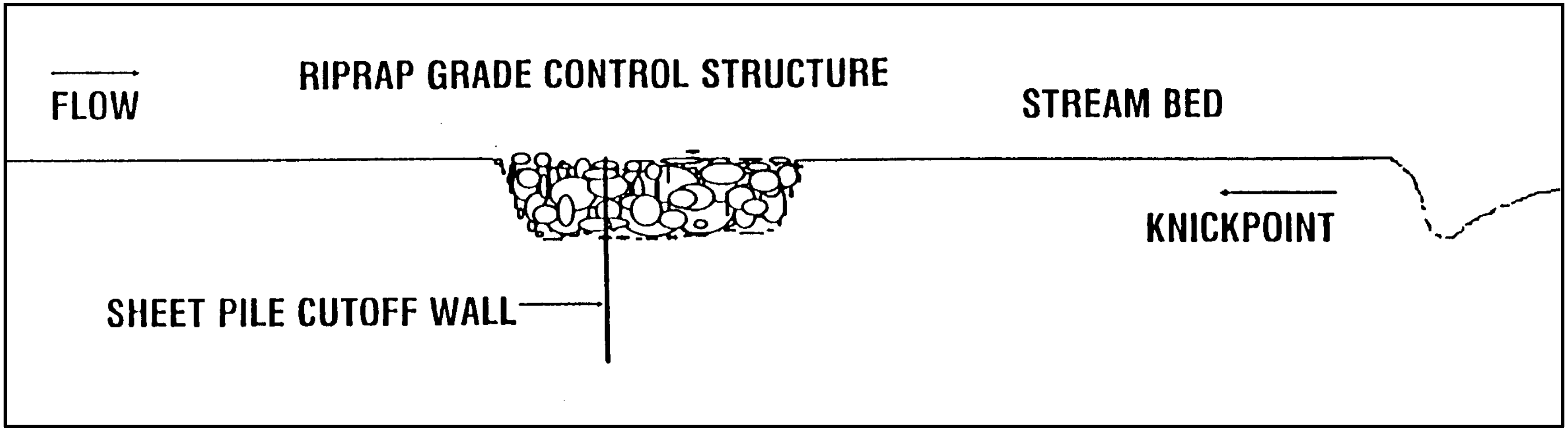

Figures 12.4b Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour



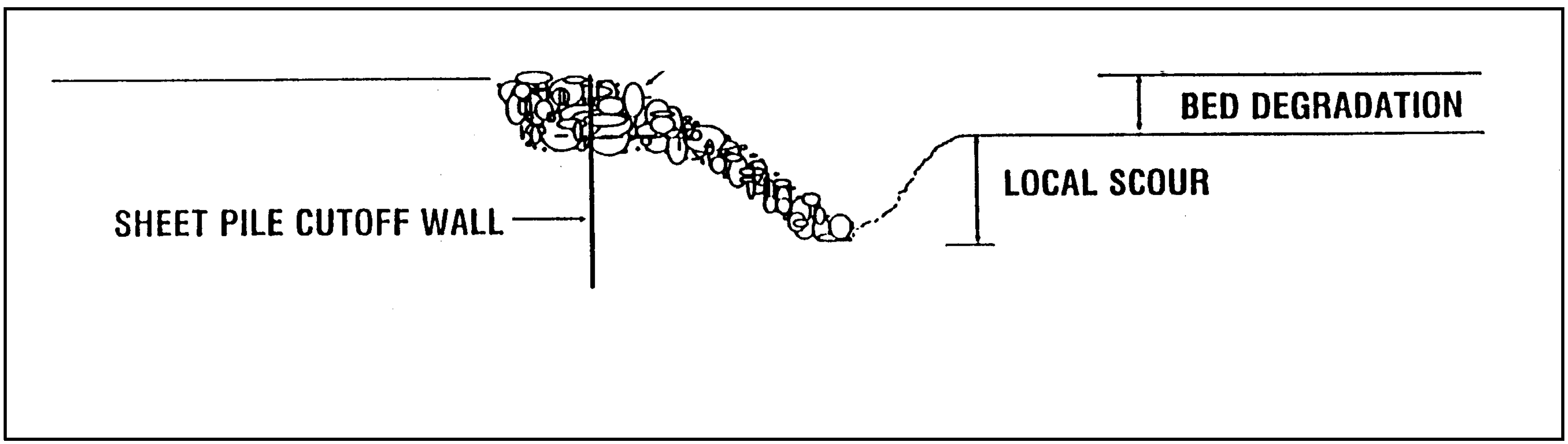

Figures 12.5a  As Built Riprap Grade Control Structure with Impervious Fill Cutoff Wall



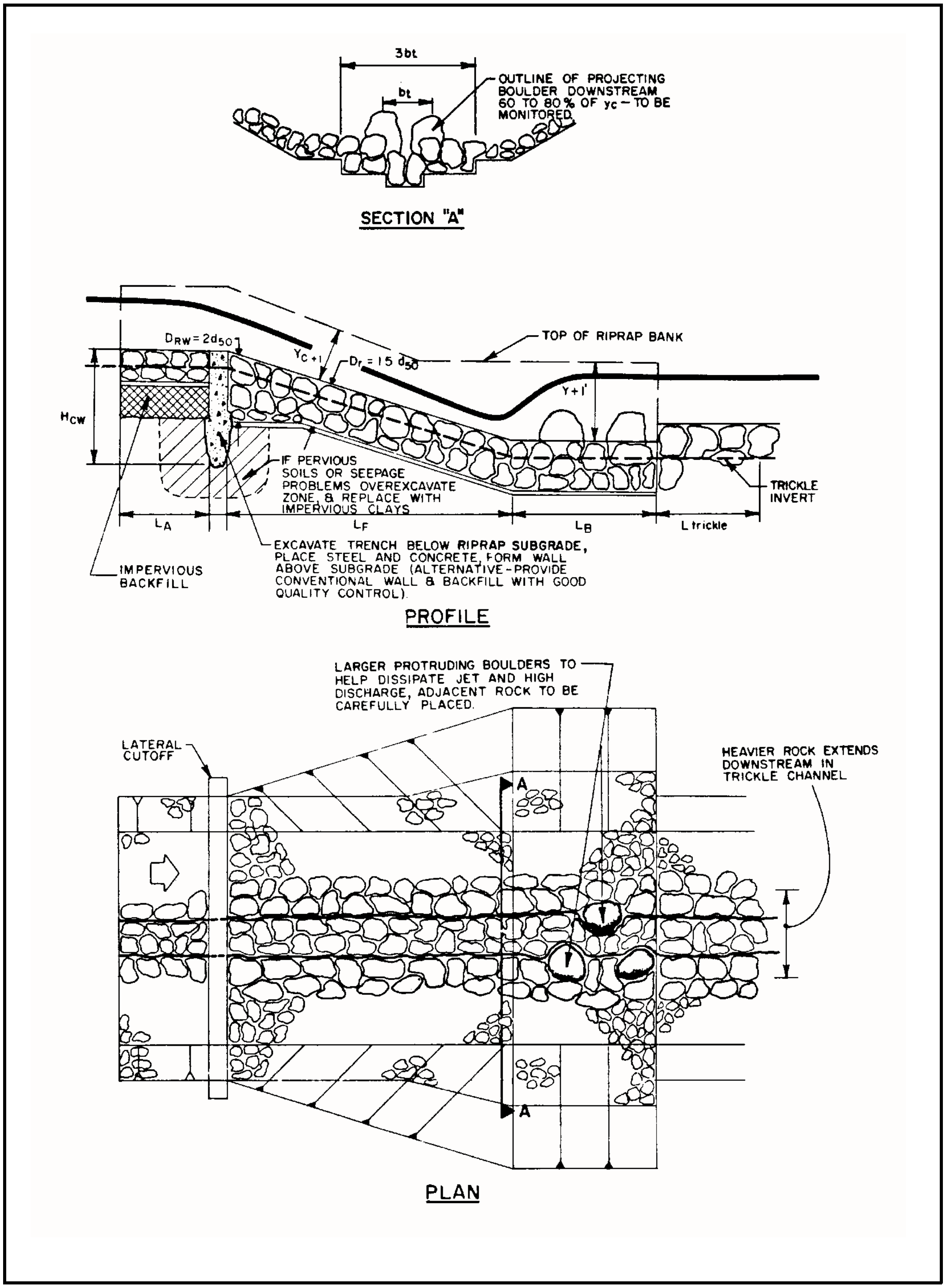

Figures 12.5b Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour




Figures 12.6a  As Built Riprap Grade Control Structure with Sheet Pile Cutoff Wall




Figures 12.6b Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour




Figures 12.7 Sloping Drop Grade Control Structure with Pre-formed Riprap Lined Scour Hole (McLaughlin Water Engineers, 1986)

### 12.3.3. STRUCTURES WITH PRE-FORMED SCOUR HOLES

A significant feature that distinguishes the sloping riprap structure of Figure 12.7 from the other structures discussed in Sections 5.1 and 5.2 is the preformed, rock protected scour hole. A scour hole is a natural occurrence downstream of any drop whether it is a natural overfall or a man-made structure. As mentioned in Section 5.1 a rock grade control structure must have sufficient launching rock to protect against the vertical scour immediately below the weir section. However, the lateral extent of the scour hole must also be considered to ensure that it does not become so large that the structure is subject to being flanked. With many simple grade control structures in small stream applications, very little, if any attention is given to the design of a stilling basin or pre-formed scour hole, but rather, the erosion is allowed to form the scour hole. However, at higher flow and drop situations, a pre-formed scour hole protected with concrete, riprap, or some other erosion resistant materials is usually warranted. This scour hole serves as a stilling basin for dissipating the energy of the plunging flow. 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 due to the stability of the rock, which is often the limiting factor in determining the maximum drop height of the structure. One way to ensure the stability of the rock is to design the structure to operate in a submerged condition. This is the basis for design of the bed stabilizer shown in Figure 12.8 (U.S. Army Corps of Engineers, 1970). These structures generally perform satisfactorily as long as they are designed to operate at submerged conditions where the tailwater (T') does not fall below 0.8 of the critical depth ($D_c$) at the crest section (Linder, 1963). Subsequent monitoring of the in place structures confirmed their successful performance in the field (U.S. Army Corps of Engineers, 1981).

In many instances, the energy dissipation in a grade control structure is accomplished by the plunging action of the flow into the riprap protected stilling basin. This is generally satisfactory where the degree of submergence is relatively high due to small drop heights and/or high tailwater conditions. However, at lower submergence conditions where drop heights are large or tailwater is low, some additional means of dissipating the energy must be provided. Little and Murphey (1982) 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 (Figure 12.9). This structure which is referred to as the ARS type low-drop structure has been used successfully in North Mississippi for drop heights up to about six feet by both the U.S. Army Corps of Engineers and the Soil Conservation Service (U.S. Army Corps of Engineers, 1981). A recent modification to the ARS structure was developed following model studies at Colorado State University (Johns et al., 1993; and Abt et al., 1994). The modified ARS structure, presented Figure 12.10 retains the baffle plate but adopts a vertical drop at the sheet pile rather than a sloping rock-fill section.

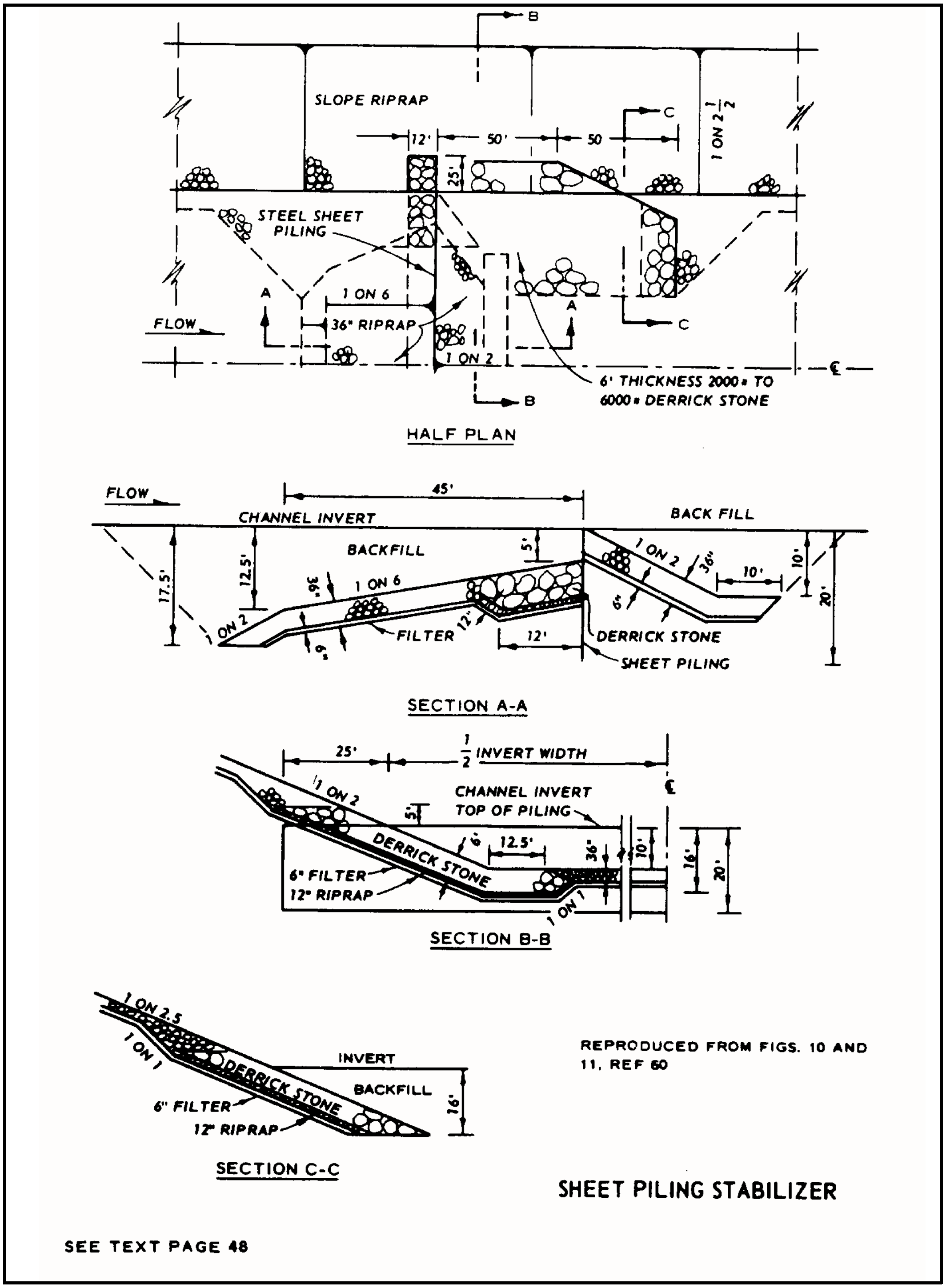



Figure 12.8 Bed Stabilizer Design with Sheet Pile Cutoff (U.S. Army Corps of Engineers, 1970)

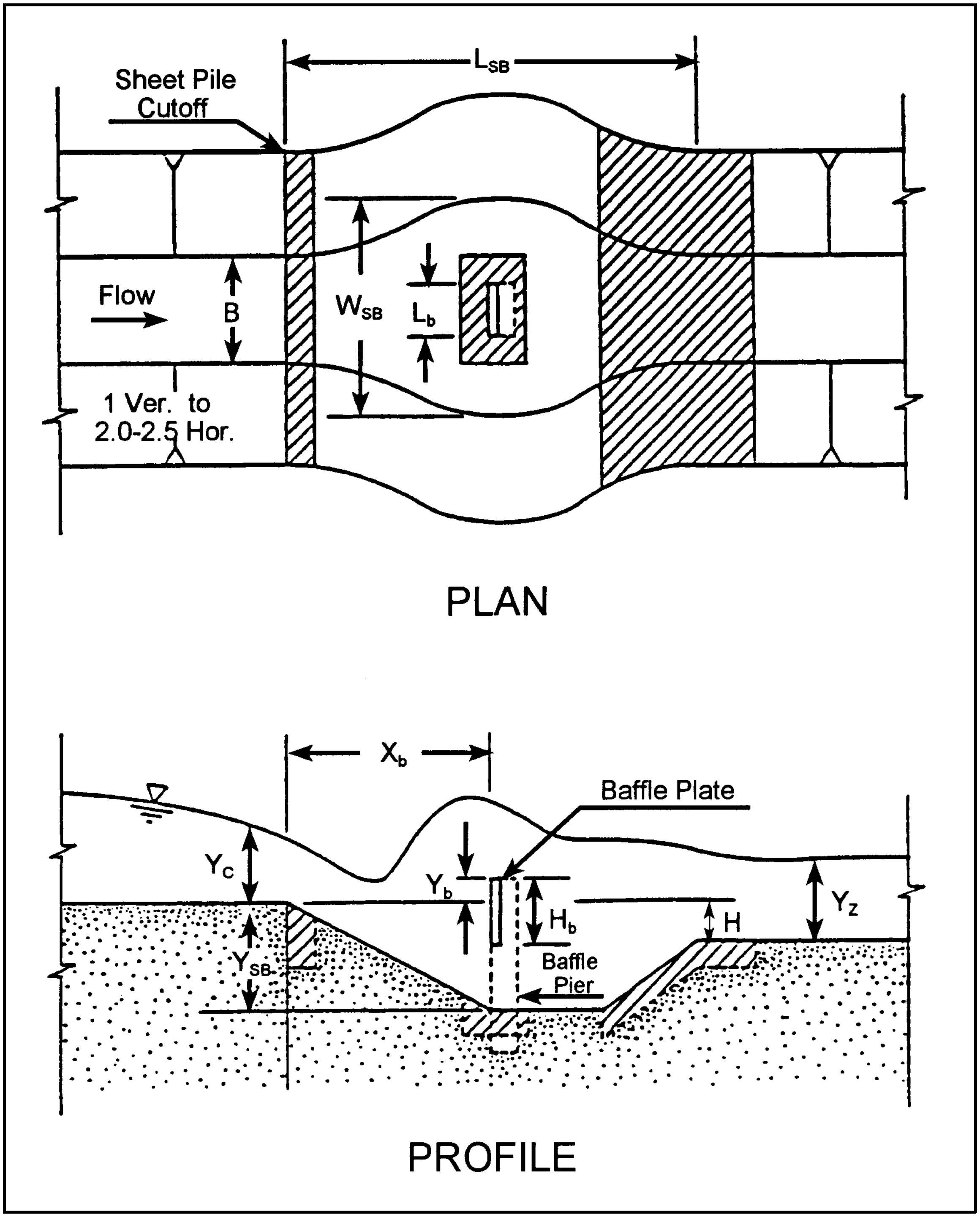



Figure 12.9 ARS-Type Grade Control Structure with Pre-formed Riprap Lined Stilling Basin and Baffle Plate (adapted from Little and Murphey, 1982)

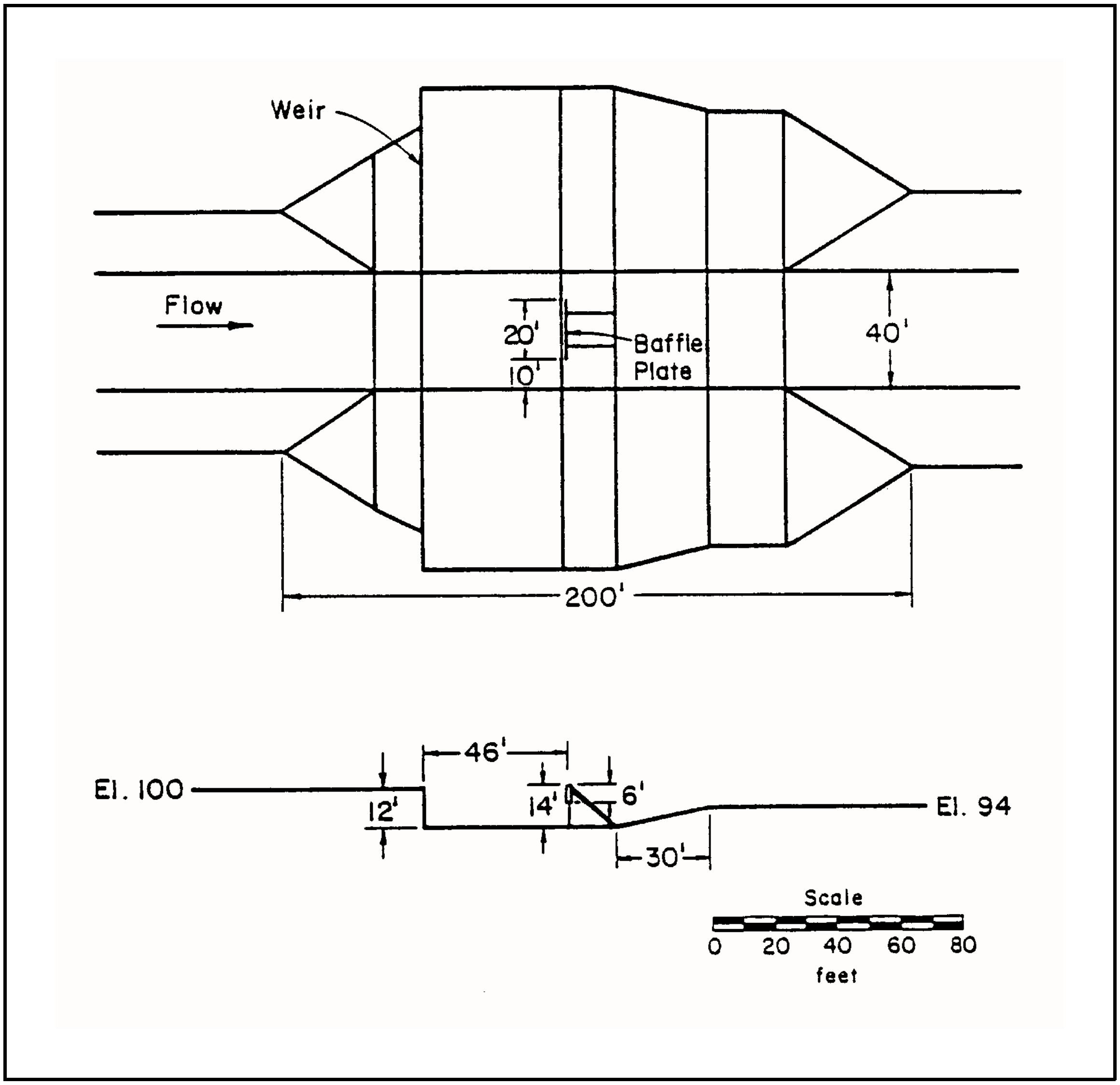



Figure 12.10 Schematic of Modified ARS-Type Grade Control Structure (Abt et al., 1994)

### 12.3.4 CONCRETE DROP STRUCTURES

In many situations where the discharges and/or drop heights are large, grade control structures are normally constructed of concrete. There are many different designs for concrete grade control structures. The two discussed herein are the CIT and the St. Anthony Falls (SAF) structures. Both of these structures were utilized on the Gering Drain project in Nebraska, where the decision to use one or the other was based on the flow and channel conditions (Stufft, 1965). Where the discharges were large and the channel depth was relatively shallow, the CIT type of drop structure was utilized. The CIT structure is generally applicable to low-drop situations where the ratio of the drop height to critical depth is less than one; 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 WES in Vicksburg, Mississippi, and is shown in Figure 12.11, (Murphy, 1967). Where the channel 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 U.S. Soil Conservation Service (Blaisdell, 1948). This structure is shown in Figure 12.12. The SAF structure is capable of functioning in flow situations where the drop height to critical depth ratio is greater than one 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 over 25 years.

### 12.3.5 CHANNEL LININGS

Grade control can also be accomplished by lining the channel bed with a non-erodible material. These structures are designed to ensure that the drop is accomplished over a specified reach of the channel which has been lined with riprap or some other non-erodible material. Rock riprap gradient control structures have been used by the U.S. Soil Conservation Service for several years (U.S. Soil Conservation Service, 1976). 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, there have been some associated local scour problems. This precipitated a series of model studies at the WES to correct these problems and to develop a design methodology for these structures (Tate, 1988; and Tate, 1991). A plan and profile drawing of the improved structure is shown in Figure 12.13.

### 12.3.6 ALTERNATIVE CONSTRUCTION MATERIALS

While riprap and concrete may be the most commonly used construction materials for grade control structures, many situations where cost or availability of materials may prompt the engineer to consider other alternatives. Gabion grade control structures are often

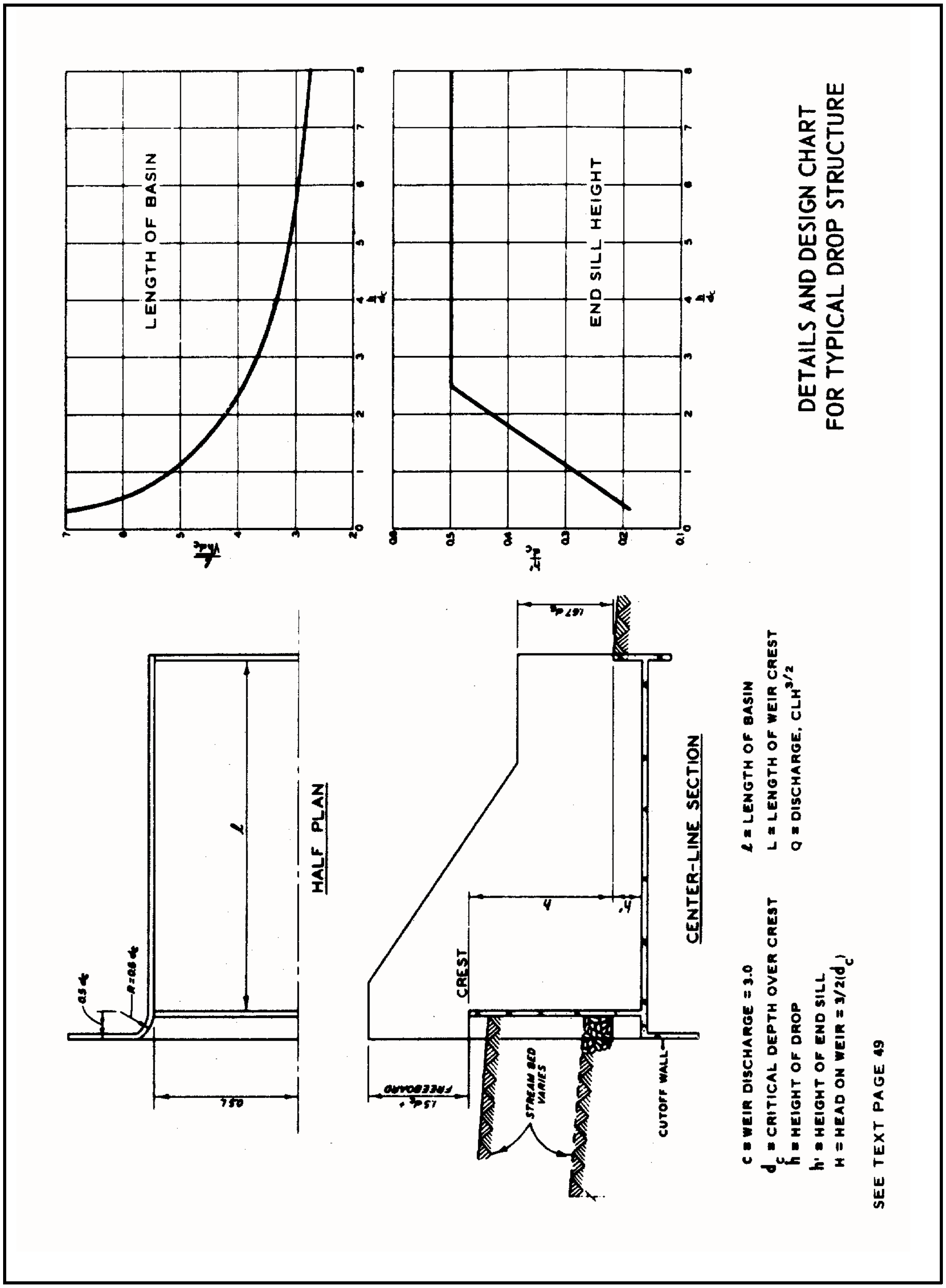



Figure 12.11 CIT-Type Drop Structure (Murphy, 1967)

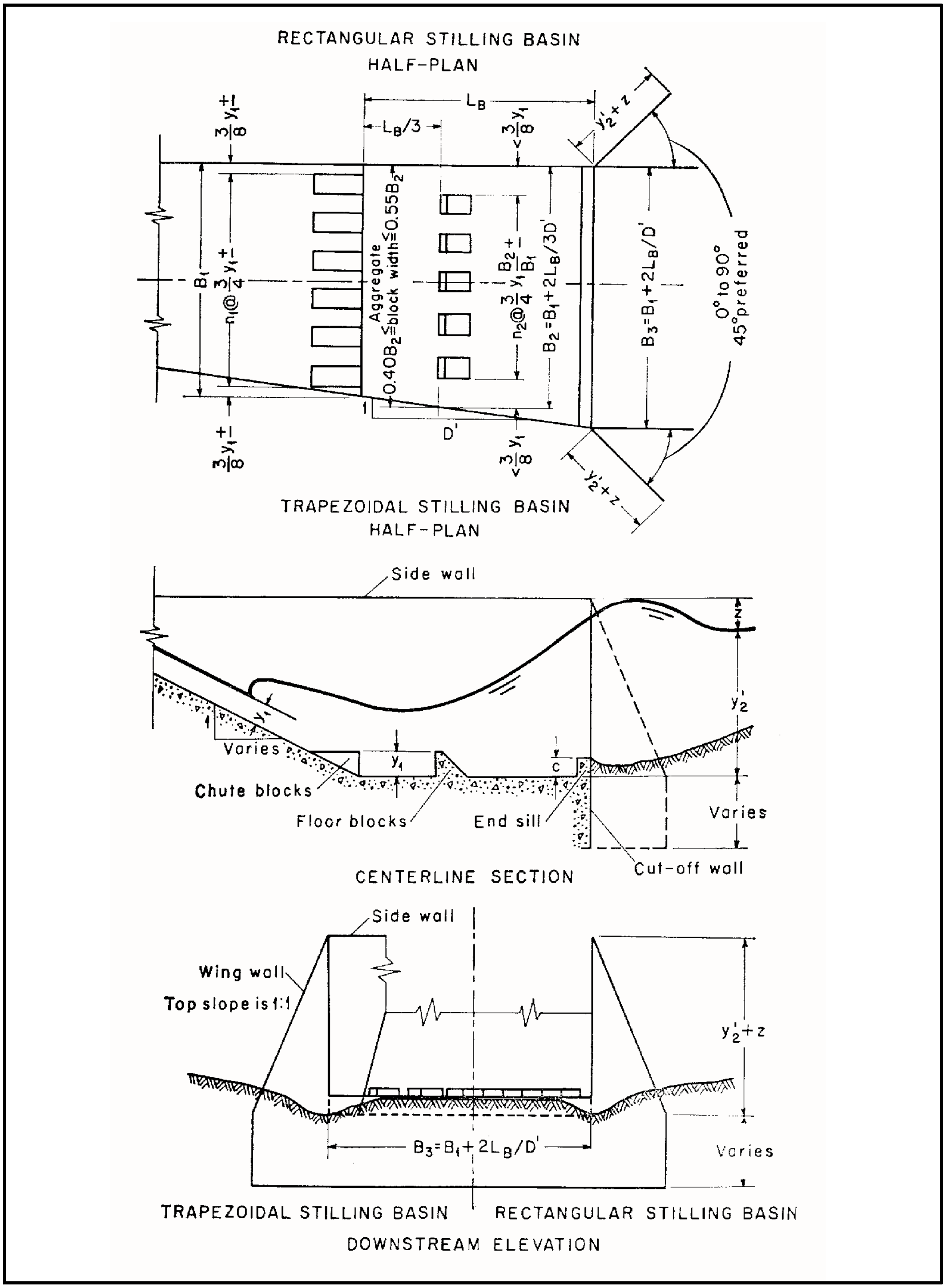



Figure 12.12  St. Anthony Falls (SAF) Type Drop Structure (Blaisdell, 1948)

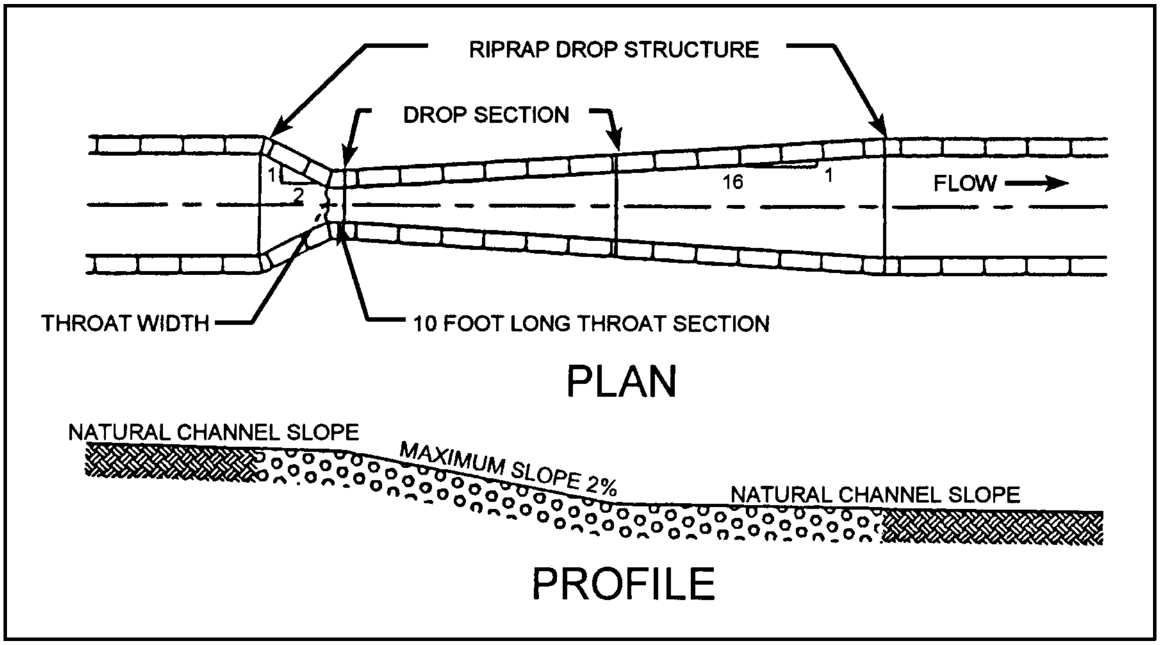



Figure 12.13 Riprap Lined Drop Structures (adapted from Tate, 1991)

an effective alternative to the standard riprap or concrete structures (Hanson et al., 1986). Agostini et al. (1988) provides design criteria for vertical, stepped, and sloped type gabion grade control structures, as well as examples of completed works. Guidance for the construction of gabion weirs is also provided by the Corps of Engineers' ETL 1110-2-194.

Another alternative to the conventional riprap or concrete structure which has gained popularity in the southwestern U.S. is the use of soil cement grade control structures. These structures are constructed of on site 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 has indicated that a limiting drop height for these structures is on the order of three feet. Design criteria for these structures is presented by Simons, Li, & Associates, Inc. (1982).

# CHAPTER 13

# CLOSING

Streambank erosion causes great economic loss, loss of cultural resources, degradation of water quality, aquatic and riparian habitat, and numerous downstream problems. For example, wetlands are frequently ephemeral features in the natural landscape, which through geomorphic process will eventually fill and become drier. Accelerated streambank erosion in the basin upstream of a wetland decreases the biological productivity of the wetland and reduces the life of the wetland. Another example that is frequently experienced is the loss of flood control capacity due to sedimentation, which is caused by accelerated streambank erosion upstream.

Streambank erosion is recognized by the Environmental Protection Agency (EPA) as a major source of nonpoint pollution. The Water Quality Act of 1987, section 101, includes the following policy statement: **It is the national policy that programs for the control of nonpoint sources of pollution be developed and implemented in an expeditious manner.** Unfortunately, at this point, no nationally recognized set of design and performance criteria exist to meet this mandate, nor is there a comprehensive manual that provides guidance for the design and construction of the many different types of streambank protection measures.

This handbook has been developed as a reference to be used in stabilization training courses to be taught by the Waterways Experiment Station (WES) in cooperation with the EPA. The topics presented in this handbook are:

- Fundamentals of fluvial geomorphology and channel process;
- Geomorphic assessment and analysis of the proposed project site and watershed system;
- General approach and principles of bank stabilization;
- Selection of site specific stabilization techniques to include surface armor, indirect methods, and bioengineered methods;
- Design and techniques for implementing grade control for system stabilization;

- Aspects of management and contracting for construction of stabilization methods; and

- Monitoring and maintenance of stabilization.

Stream bank stabilization is not a simple matter of sizing armor material large enough to resist fluvial entrainment, placing the armor at locations of bank erosion, and moving to the next eroding site. As we have attempted to emphasize throughout this document, a cookbook approach to bank stabilization is the embodiment of the phrase, **complex problems often have quick and simple wrong answers.** Although this first attempt in developing a bank stabilization handbook is in excess of 300 pages and requires a significant effort by the user, we firmly believe that the effort to consider the full range of topics in a bank stabilization design is worth the effort. **Proper streambank stabilization is a complex problem without quick and simple answers.**

This handbook will be used by persons of a wide diversity of experience and education, we would appreciate your comments and suggestions for enhancement of the document. Please contact:

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# APPENDIX A

# DESIGN PROCEDURE FOR RIPRAP ARMOR

# APPENDIX A

# DESIGN PROCEDURE FOR RIPRAP ARMOR

## A.1 INTRODUCTION

### A.1.1 GENERAL

The guidance presented herein is being used by the U.S. Army Corps of Engineers (EM 1110-2-1601) and applies to riprap design for the following conditions:

> Open channels are immediately downstream of stilling basins or other highly turbulent areas.
>
> Channel slopes less than 2 percent.

The ability of riprap revetment to resist the erosive forces of channel flow depends on the interrelation of the following stone and channel factors:

> stone shape, size, weight, durability, gradation; riprap layer thickness; and channel side slopes, roughness, shape, alignment, and invert slope.

The bed material and local scour characteristics determine the design of toe protection, which is essential for riprap revetment stability. The bank material and groundwater conditions affect the need for filters between the riprap and underlying material. Construction quality control of both stone production and riprap placement is essential for successful bank protection. Riprap protection for flood-control channels and appurtenant structures should be designed so that any flood that could reasonably be expected to occur during the service life of the channel or structure would not cause damage exceeding nominal maintenance. While the procedures presented herein yield definite stone sizes, results should be used for guidance purposes and revised if appropriate, based on experience with specific project conditions.

### A.1.2 CURRENT RESEARCH

Some aspects of riprap design are not precisely defined. In order to provide more specific guidance, the Corps of Engineers has conducted studies to determine stability of various riprap gradations and thickness, velocity estimation in bends, impinged flow in braided streams, and design of toe and end protection. Many of these studies were conducted in a large outdoor test channel having a capacity of 200 cfs. For information about these studies, contact the Hydraulics Laboratory at the U.S. Army Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199, USA.

### A.1.3 OTHER APPROACHES

The references contain descriptions of some other approaches to riprap design. The designer may want to examine these other approaches to determine if they would be useful in some applications to supplement the guidance presented here.

## A.2 RIPRAP CHARACTERISTICS

The following provides guidance on stone shape, size/weight relationship, unit weight, gradation, and layer thickness. Reference EM 1110-2-2302 for additional guidance on riprap material characteristics and construction.

### A.2.1 STONE SHAPE

Riprap should be blocky in shape rather than elongated, as more nearly cubical stones "nest" together best and are more resistant to movement. The stone should have sharp, clean edges at the intersections of relatively flat faces. Cobbles with rounded edges are less resistant to movement, although the drag force on a rounded stone is less than on sharp-edged cubical stones. As graded cobble interlock is less than that of equal-sized angular stones, the cobble mass is more likely to be eroded by channel flow. If used, the cobbles should be placed on flatter side slopes than angular stone and should be about 25 percent larger in diameter. The following shape limitations should be specified for riprap obtained from quarry operations:

> The stone shall be predominately angular and subangular in shape.
>
> Not more than 30 percent of the stones reasonably well distributed throughout the gradation shall have a/c greater than 2.5.
>
> Not more than 15 percent of the stones reasonably well distributed throughout the gradation shall have a/c greater than 3.0.

No stone shall have a/c exceeding 3.5.

To determine stone dimensions "a" and "c," consider that the stone has a long axis, an intermediate axis and a short axis each being perpendicular to the other. Dimension "a" is the maximum length of the stone which defines the long axis of the stone. The intermediate axis is defined by the maximum width of the stone. The remaining axis is the short axis. Dimension "c" is the maximum dimension parallel to the short axis. These limitations apply only to the stone within the required riprap gradation and not to quarry spalls and waste that may be allowed.

## A.2.2 RELATION BETWEEN STONE SIZE AND WEIGHT

The ability of riprap revetment to resist erosion is related to the size and weight of stones. Design guidance is often expressed in terms of the stone size $D_{\%}$, where % denotes the percentage of the total weight of the graded material, including quarry wastes and spalls, that contains stones of less weight. The relation between size and weight of stone is described herein using a spherical shape by the following equation:

$$D_{\%} = \left( \frac{6W_{\%}}{\pi \gamma_s} \right)^{1/3} \qquad \text{(A.1)}$$

where

$D_{\%}$ = equivalent-volume spherical stone diameter (ft),
$W_{\%}$ = weight of individual stone having diameter of $D_{\%}$, (lbs), and
$\gamma$ = saturated surface dry (SSD) specific weight of stone (lbs per cu. ft).

Design procedures for determining the stone size required to resist the erosive forces of channel flow are presented in Section A.4.

## A.2.3 UNIT WEIGHT

Unit weight of stone $\gamma_s$ generally varies from 150 to 170 pounds per cubic foot. Riprap sizing relations are relatively sensitive to unit weight of stone, and $\gamma_s$ should be determined as accurately as possible. In many cases, the unit weight of stone is not accurately known because the quarry is selected from a list of approved riprap sources after the construction contract is awarded. Under these circumstances, a unit weight of stone close to the minimum of the available riprap sources should be used in design. Contract options covering specific weight ranges of 5 or 10 pounds per cubic foot should be offered when sufficient savings warrant.

## A.2.4 GRADATION

(1) The gradation of stones in riprap revetment affects the riprap's resistance to erosion. The stone should be reasonably well graded throughout the in-place layer thickness. Specifications should provide for two limiting gradation curves, and any stone gradation as determined from quarry process, stockpile, and in-place field test samples that lies within these limits should be acceptable. Riprap sizes and weights are frequently used as $D_{30}$(min), $D_{100}$(max), $W_{50}$(min), etc. The D or W refers to size or weight, respectively. The number is the percent finer by weight. The (max) or (min) refers to the upper or lower gradation curves, respectively. A standard form for plotting riprap gradation curves is provided as Figure A.1. The gradation limits should not be so restrictive that production costs would be excessive. The choice of limits also depends on the underlying bank soils and filter requirements if a graded stone filter is used.

(2) Standardized gradations having a relatively narrow range in sizes ($D_{85}/D_{15} = 1.4$ - 2.2) are shown in Table A.1. Other gradations can be used and often have a wider range of allowable sizes than those given in Table A.1. One example is the Lower Mississippi River standardized gradations (see EM 1110-2-1601) that are identical to the Table A.1 gradations except that the $W_{50}$(max) and $W_{15}$(max) weights are larger, making them easier to produce. Most graded ripraps have ratios of $D_{85}/D_{15}$ less than 3. Uniform riprap ($D_{85}/D_{15} < 1.4$) has been used at sites on the Missouri River basin for reasons of economy and quality control of sizes and placement.

(3) Rather than using a relatively expensive graded riprap, a greater thickness (1-1/2 to 2 times that of graded stone as indicated in paragraph e) of a quarry-run stone may be considered. Some designers consider the quarry stone to have another advantage: the gravel and sand-size rock present in the quarry stone provide a rudimentary filter. This concept has resulted in considerable cost savings on large projects such as the Arkansas and Red River navigation projects in the United States. Not all quarry-run stone can be used as riprap; stone that is gap-graded (some sizes missing from gradation) or has a large range in maximum to minimum size is probably unsuitable. Quarry-run stone for riprap should be limited to $D_{85}/D_{15}$ # 7.

(4) Determining optimum gradations is also an economics problem that includes the following factors:

Rock quality (durability under service conditions);

Cost per ton at the quarry (including capability of quarry to produce a particular size);

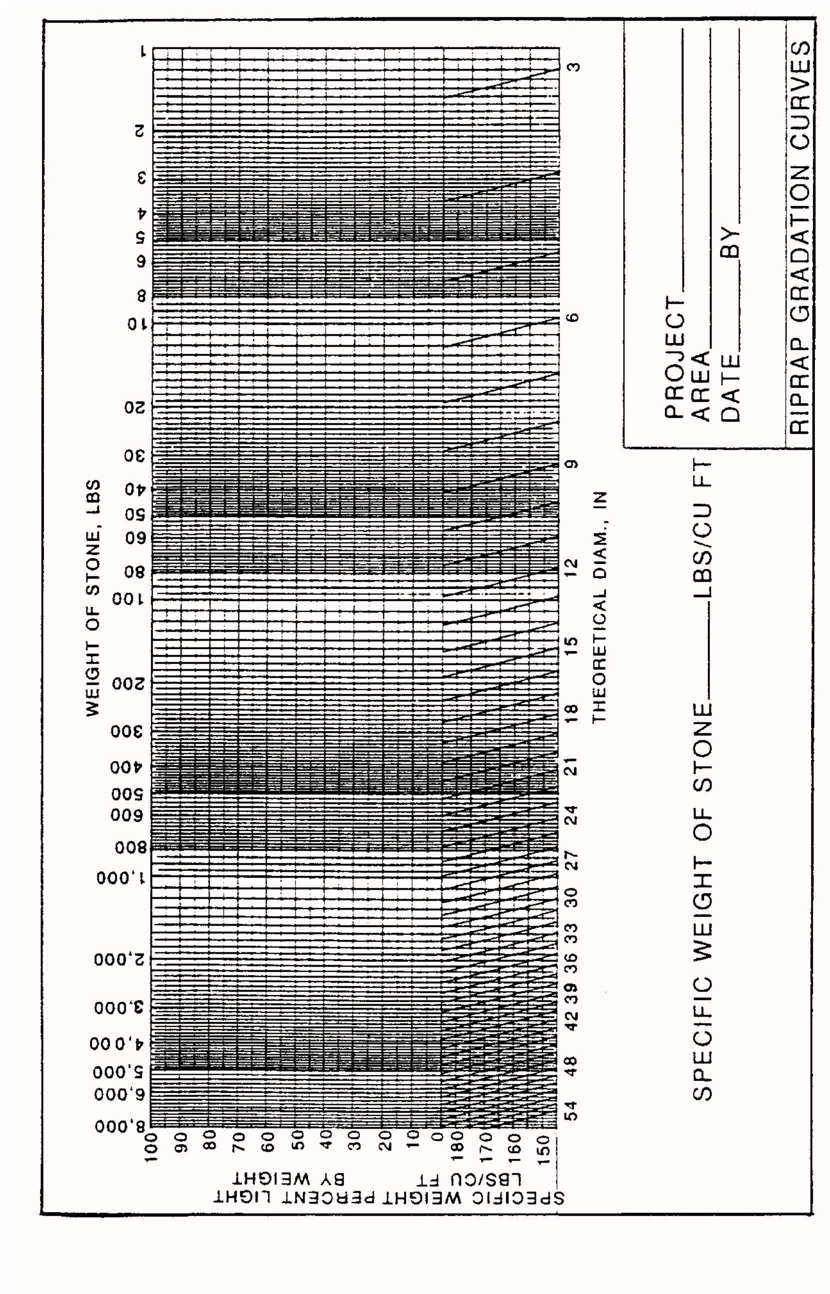


Figure A.1 Riprap Gradation Curves

Table A.1. Gradations for Riprap Placement in the Dry, Low-Turbulence Zones

Limits of Stone Weight, lb[1], for Percent Lighter by Weight

| $D_{100}$(max) (in.) | 100 Max | 100 Min | 50 Max[2] | 50 Min | 15 Max[2] | 15 Min | $D_{30}$(min) (ft) | $D_{90}$(min) (ft) |
|---|---|---|---|---|---|---|---|---|
| Specific Weight = 155 pcf | | | | | | | | |
| 12 | 81 | 32 | 24 | 16 | 12 | 5 | 0.48 | 0.70 |
| 15 | 159 | 63 | 47 | 32 | 23 | 10 | 0.61 | 0.88 |
| 18 | 274 | 110 | 81 | 55 | 41 | 17 | 0.73 | 1.06 |
| 21 | 435 | 174 | 129 | 87 | 64 | 27 | 0.85 | 1.23 |
| 24 | 649 | 260 | 192 | 130 | 96 | 41 | 0.97 | 1.40 |
| 27 | 924 | 370 | 274 | 185 | 137 | 58 | 1.10 | 1.59 |
| 30 | 1,268 | 507 | 376 | 254 | 188 | 79 | 1.22 | 1.77 |
| 33 | 1,688 | 675 | 500 | 338 | 250 | 105 | 1.34 | 1.94 |
| 36 | 2,191 | 877 | 649 | 438 | 325 | 137 | 1.46 | 2.11 |
| 42 | 3,480 | 1,392 | 1,031 | 696 | 516 | 217 | 1.70 | 2.47 |
| 48 | 5,194 | 2,078 | 1,539 | 1,039 | 769 | 325 | 1.95 | 2.82 |
| 54 | 7,396 | 2,958 | 2,191 | 1,479 | 1,096 | 462 | 2.19 | 3.17 |
| Specific Weight = 165 pcf | | | | | | | | |
| 12 | 86 | 35 | 26 | 17 | 13 | 5 | 0.48 | 0.70 |
| 15 | 169 | 67 | 50 | 34 | 25 | 11 | 0.61 | 0.88 |
| 18 | 292 | 117 | 86 | 58 | 43 | 18 | 0.73 | 1.06 |
| 21 | 463 | 185 | 137 | 93 | 69 | 29 | 0.85 | 1.23 |
| 24 | 691 | 276 | 205 | 138 | 102 | 43 | 0.97 | 1.40 |
| 27 | 984 | 394 | 292 | 197 | 146 | 62 | 1.10 | 1.59 |
| 30 | 1,350 | 540 | 400 | 270 | 200 | 84 | 1.22 | 1.77 |
| 33 | 1,797 | 719 | 532 | 359 | 266 | 112 | 1.34 | 1.96 |
| 36 | 2,331 | 933 | 691 | 467 | 346 | 146 | 1.46 | 2.11 |
| 42 | 3,704 | 1,482 | 1,098 | 741 | 549 | 232 | 1.70 | 2.47 |
| 48 | 5,529 | 2,212 | 1,638 | 1,106 | 819 | 346 | 1.95 | 2.82 |
| 54 | 7,873 | 3,149 | 2,335 | 1,575 | 1,168 | 492 | 2.19 | 3.17 |
| Specific Weight = 175 pcf | | | | | | | | |
| 12 | 92 | 37 | 27 | 18 | 14 | 5 | 0.48 | 0.70 |
| 15 | 179 | 72 | 53 | 36 | 27 | 11 | 0.61 | 0.88 |
| 18 | 309 | 124 | 92 | 62 | 46 | 19 | 0.73 | 1.06 |
| 21 | 491 | 196 | 146 | 98 | 73 | 31 | 0.85 | 1.23 |
| 24 | 733 | 293 | 217 | 147 | 109 | 46 | 0.97 | 1.40 |
| 27 | 1,044 | 417 | 309 | 209 | 155 | 65 | 1.10 | 1.59 |
| 30 | 1,432 | 573 | 424 | 286 | 212 | 89 | 1.22 | 1.77 |
| 33 | 1,906 | 762 | 565 | 381 | 282 | 119 | 1.34 | 1.94 |
| 36 | 2,474 | 990 | 733 | 495 | 367 | 155 | 1.46 | 2.11 |
| 42 | 3,929 | 1,571 | 1,164 | 786 | 582 | 246 | 1.70 | 2.47 |
| 48 | 5,864 | 2,346 | 1,738 | 1,173 | 869 | 367 | 1.95 | 2.82 |
| 54 | 8,350 | 3,340 | 2,474 | 1,670 | 1,237 | 522 | 2.19 | 3.17 |

[1] Stone weight limit data from ETL 1110-2-120 (HQUSACE, 1971 (14 May) "Additional Guidance for Riprap Channel Protection, encl. 1," U.S. Government Printing Office, Washington, DC). Relationship between diameter and weight is based on the shape of a sphere.

[2] The maximum limits at the $W_{50}$ and $W_{15}$ sizes can be increased as in the Lower Mississippi Valley Division Standardized Gradations shown in EM 1110-2-1601.

Number of tons required;

Miles transported;

Cost of transportation per ton-mile;

Cost per ton for placement;

Quality control during construction (it is easier to ensure even coverage with a narrow gradation than with a wide gradation); and

Need for and cost of filter.

On large project involving many different design conditions, savings in total cost can often be realized by using a few standard gradations, selecting the standard gradation next up from the computed gradation.

## A.2.5 LAYER THICKNESS

All stones should be contained reasonably well within the riprap layer thickness to provide maximum resistance against erosive forces. Oversize stones, even in isolated spots, may result in riprap failure by precluding mutual support and interlock between individual stones, causing large voids that expose filter and bedding materials, and creating excessive local turbulence that removes smaller size stone. Small amounts of oversize stone should be removed individually and replaced with proper size stones. When a quarry produces a large amount of oversized stone, consideration should be given to changing the quarrying method, using a grizzly (sieve for large rock) to remove the oversize stone, obtaining the stone from another source, or increasing the riprap layer thickness to contain the larger stone. The following criteria apply to the riprap layer thickness:

It should not be less than the spherical diameter of the upper limit $W_{100}$ stone or less than 1.5 times the spherical diameter of the upper limit $W_{50}$ stone, whichever results in the greater thickness. The thickness thus determined should be increased by 50 percent when the riprap is placed underwater to provide for uncertainties associated with this type of placement. At one location in the Missouri River basin in the United States, divers and sonic sounders were used to reduce the underwater thickness to 1.25 times the dry placement thickness.

## A.3 CHANNEL CHARACTERISTICS

### A.3.1 SIDE SLOPES

The stability of riprap bank revetment is affected by the steepness of channel side slopes. Side slopes should not be steeper than 1V on 1.5H, except in special cases where it may be economical to use larger hand-placed stone keyed well into the bank. Side slopes from 1V on 2H to 1V on 3H are recommended for riprap stability. The size of stone required to resist the erosive forces of channel flow increases when the side slope angle approaches the angle of repose of a riprap revetment. Rapid water-level recession and piping-initiated failures are other factors in determining channel side slopes. Embankment stability analysis should properly address soils characteristics, ground-water and river conditions, and probable failure mechanisms.

### A.3.2 CHANNEL ROUGHNESS, SHAPE, ALIGNMENT, AND INVERT SLOPE

As boundary shear forces and velocities depend on channel roughness, shape, alignment, and invert slope, these factors must be considered in determining the size of stone required for riprap revetment. Comparative cost estimates should be made for several alternative channel plans to determine the most economical and practical combination of channel factors and stone size. Resistance coefficients (manning's n) for riprap surfaces should be estimated using the following form of Stricklers equation:

$$n = K \left[D_{90}(\min)^{1/6}\right] \qquad \text{(A.2)}$$

where

| | | |
|---|---|---|
| $D_{50}(\min)$ | = | size of which 90 percent of sample is finer, from minimum or lower limit curve of gradation specification (ft), |
| k | = | 0.036 average of all flume data, |
| K | = | 0.034 for velocity and stone size calculation, and |
| | = | 0.038 for capacity and freeboard calculation. |

The K values represent the upper and lower bounds of laboratory data determined for bottom riprap. Resistance data from a large laboratory channel having an irregular riprap surface similar to riprap placed underwater resulted in a 15% increase in Manning's n above the dry placement values given above. These Manning n values represent only the grain resistance of the riprap surface.

## A.4 DESIGN GUIDANCE FOR STONE SIZE

### A.4.1 GENERAL

Riprap protection for open channels is subjected to hydrodynamic drag and lift forces that tend to erode the revetment and reduce its stability. Undermining by scour beyond the limits of protection is also a common cause of failure. The drag and lift forces are created by flow velocities adjacent to the stone. Forces resisting motion are the submerged weight of the stone and any downward and lateral force components caused by contact with other stones in the revetment. Characteristics of the available stone and the designer's experience play a large part in determining size of riprap. This is particularly true on projects where hydraulic parameters are ill-defined and the total amount of riprap required is small.

### A.4.2 DESIGN CONDITIONS

Stone size computations should be conducted for flow conditions that produce the maximum velocity at the riprapped boundary. In many cases, velocities continue to increase beyond bank-full discharge; but in some cases backwater effects or loss of flow into the overbanks results in velocities that are less than those at bankfull. Channel bend riprap is conservatively designed for the point having the maximum force or velocity. For braided channels, bank-full discharges may not be the most severe condition. At lesser flows, flow is often divided into multiple channels. Flow in these channels often impinges abruptly on banks or levees at sharp angles. Precise guidance is lacking in defining design conditions for braided channels, although a correction factor for velocity is suggested.

### A.4.3 STONE SIZE

The method presented here for determining stone size uses depth averaged local velocity, since a designer will be better able to estimate local velocity than local boundary shear. Local depth-averaged velocity and local flow depth are used in this procedure to quantify the imposed forces. Riprap size and submerged unit weight quantify the resisting force of the riprap. This method is based on a large body of laboratory data and has been compared to available prototype data (Maynord, 1988). This method defines the stability of a wide range of gradations if placed to a thickness of $1D_{100}$(max). Guidance for thickness greater than $1D_{100}$(max) is presented. The method is applicable to side slopes of 1V on 1.5h or flatter.

(1) Velocity Estimation. The characteristic velocity for side slopes $V_{SS}$ is the depth-averaged local velocity over the slope at a point 20% of the slope length from the toe of slope. The 20% location was selected because it represents the point of maximum side slope shear in straight channels. Various methods exist to estimate local depth-averaged velocity for use in this design procedure. Numerical methods include two-dimensional depth averaged models. Physical

models can be used to determine depth-averaged velocities but are rarely justified due to cost. Figure A.2 presents an empirical method to estimate the ratio $V_{ss}/V_{avg}$ as a function of the channel alignment and geometry which is described by R/W and aspect ratio. The following notation is used:

| | | |
|---|---|---|
| $V_{ss}$ | = | characteristic side slope velocity (maximum at any point along bend) (length/time), |
| $V_{avg}$ | = | average channel velocity at upstream end of bend in the main channel only (length/time), |
| R | = | center-line radius of bend (length), and |
| W | = | water-surface width of the main channel, length (note that W here should not be confused with stone weight). |

Velocity downstream of bends decays at approximately the following rate: No decay in first channel width of bend exit; decay of $V_{ss}/V_{avg} = 0.1$ per channel width until $V_{ss}/V_{avg} = 1.0$. For straight channels sufficiently far (>5W-10W) from upstream bends, $V_{ss}/V_{avg}$ shown in Figure A.3 are recommended. However, few channels are straight enough to use $V_{ss}/V_{avg} < 1$. See Figure A.4 for a description of $V_{SS}$ and Figure A.5 for the location in a trapezoidal channel bend where the maximum near-bank velocity was located. Figure A.6 shows the variation in velocity over the side slope in the exit region downstream of a bend. Figures A.4, A.5, and A.6 are presented to illustrate concepts; the designer should consider the specific geometry. For equal cross-sectional areas, steep side slopes tend to move the maximum bend velocities away from the side slope; whereas, mild side slopes allow the maximum bend velocities to occur over the side slope. Analytical methods are velocity estimation, such as velocities resulting from subsections of a water-surface profile computation, should be used only in straight reaches, in which case the velocity from the subsection adjacent to the bank subsection should be used as $V_{SS}$ in design of bank riprap. Appendix G in EM 1110-2-1601 provides a velocity estimation method based on using observed field data to estimate riprap design velocities.

(2) Stone Size Relations. The basic equation for the representative stone size in straight or curved channels is

$$D_{30} = S_f C_s C_v C_T d \left[ \left( \frac{\gamma_W}{\gamma_s - \gamma_W} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5} \tag{A.3}$$

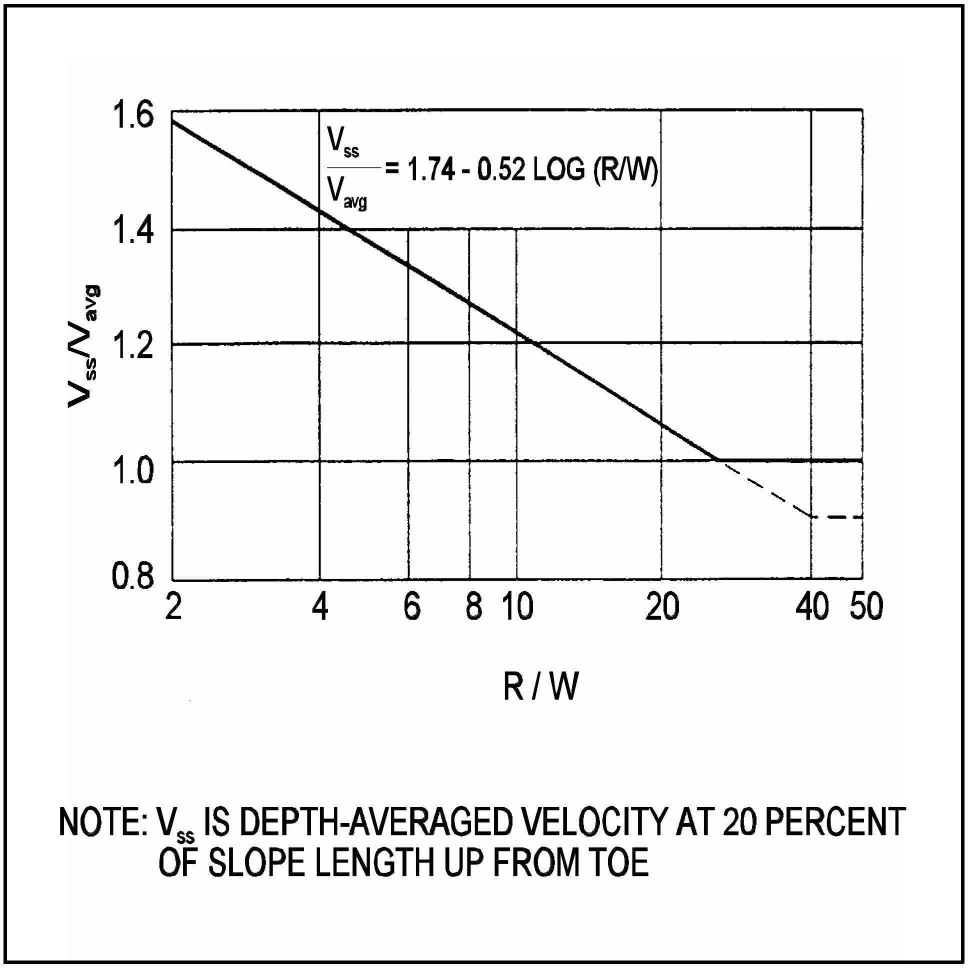


Figure A.2a Riprap Design Velocities

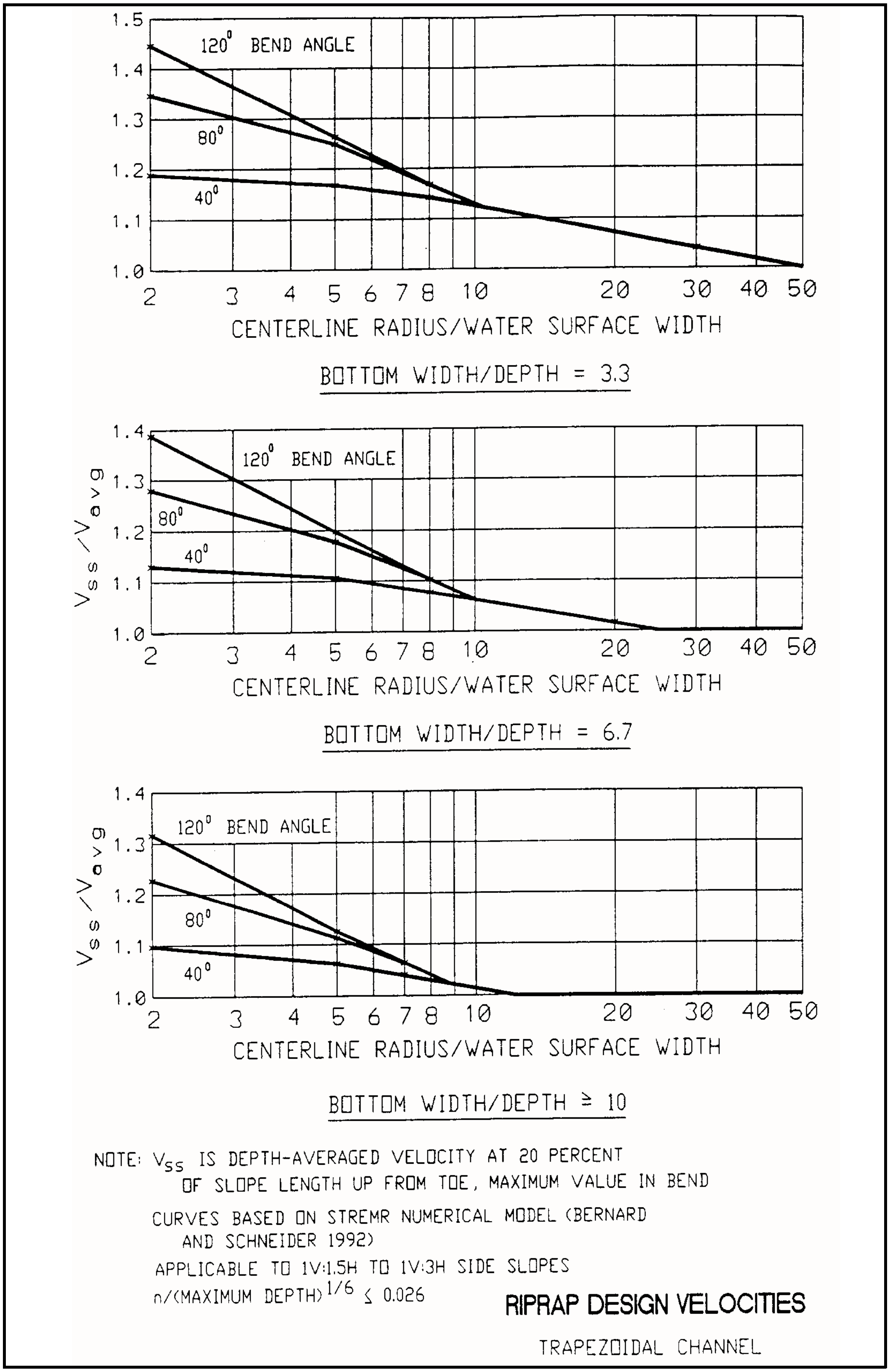


Figure A.2b Riprap Design Velocities

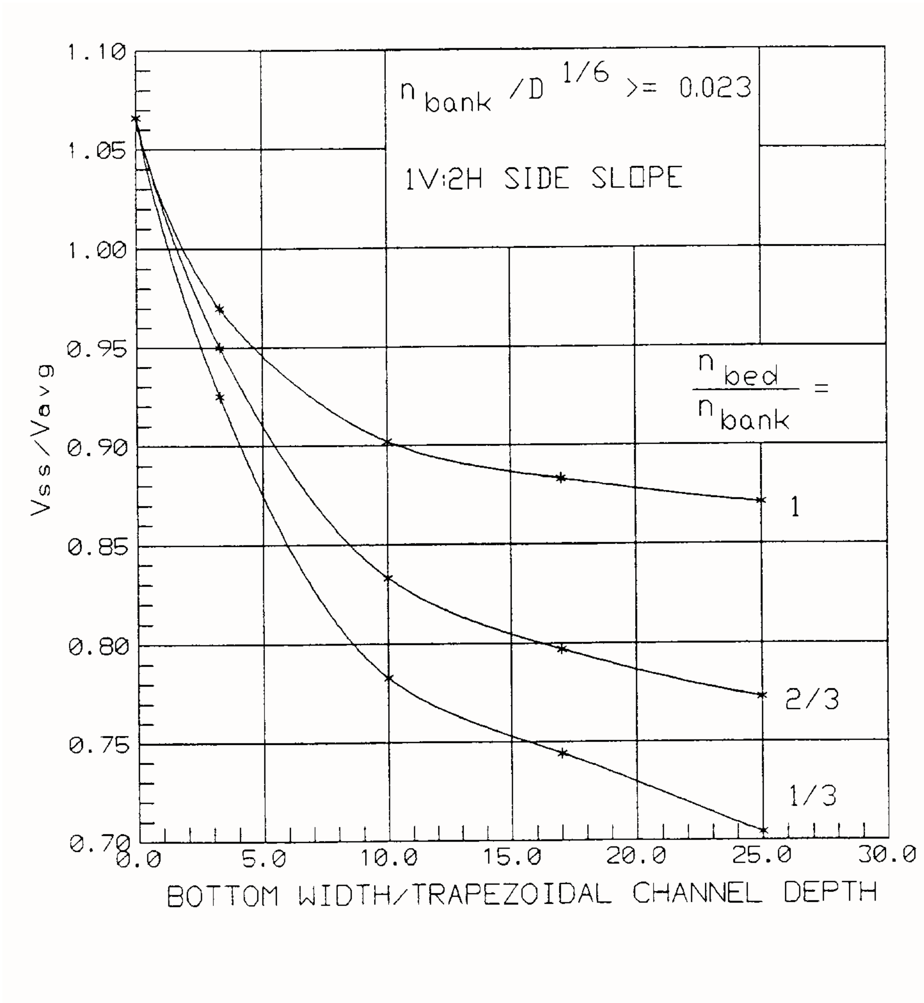


Figure A.3 $V_{ss}/V_{avg}$ for Straight Channels Sufficiently Far From (>5w-10w) Upstream Bends

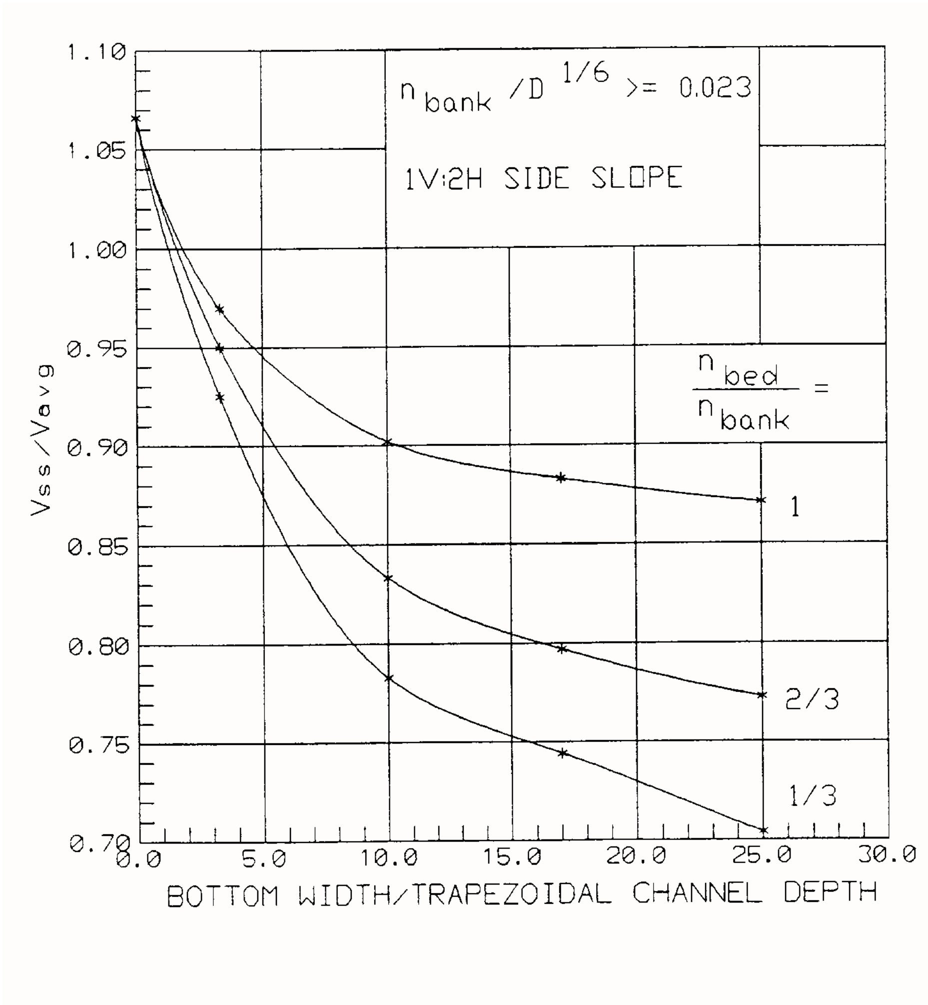


Figure A.3 $V_{ss}/V_{avg}$ for Straight Channels Sufficiently Far From (>5w-10w) Upstream Bends

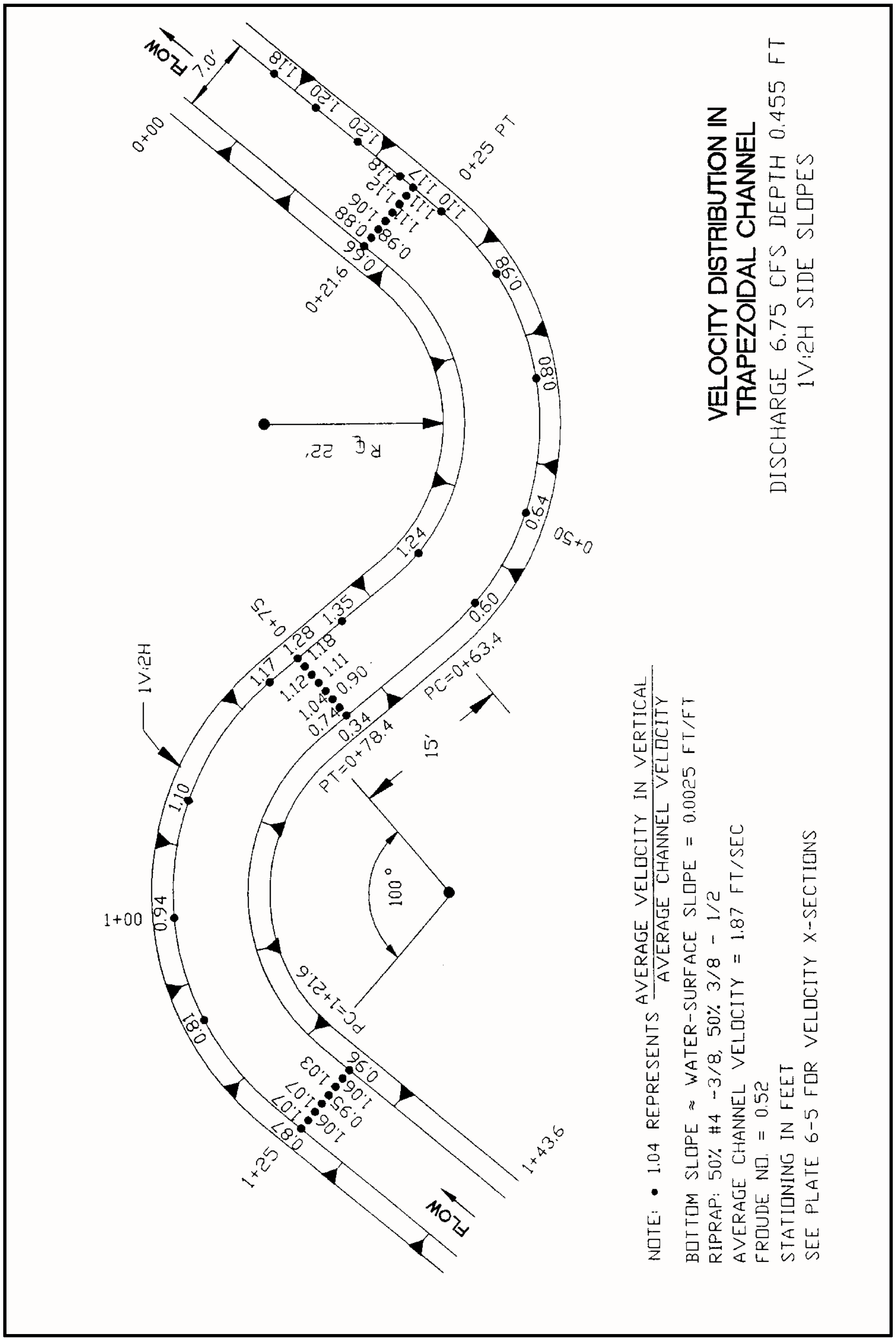


Figure A.5 Velocity Distribution in Trapezoidal Channel

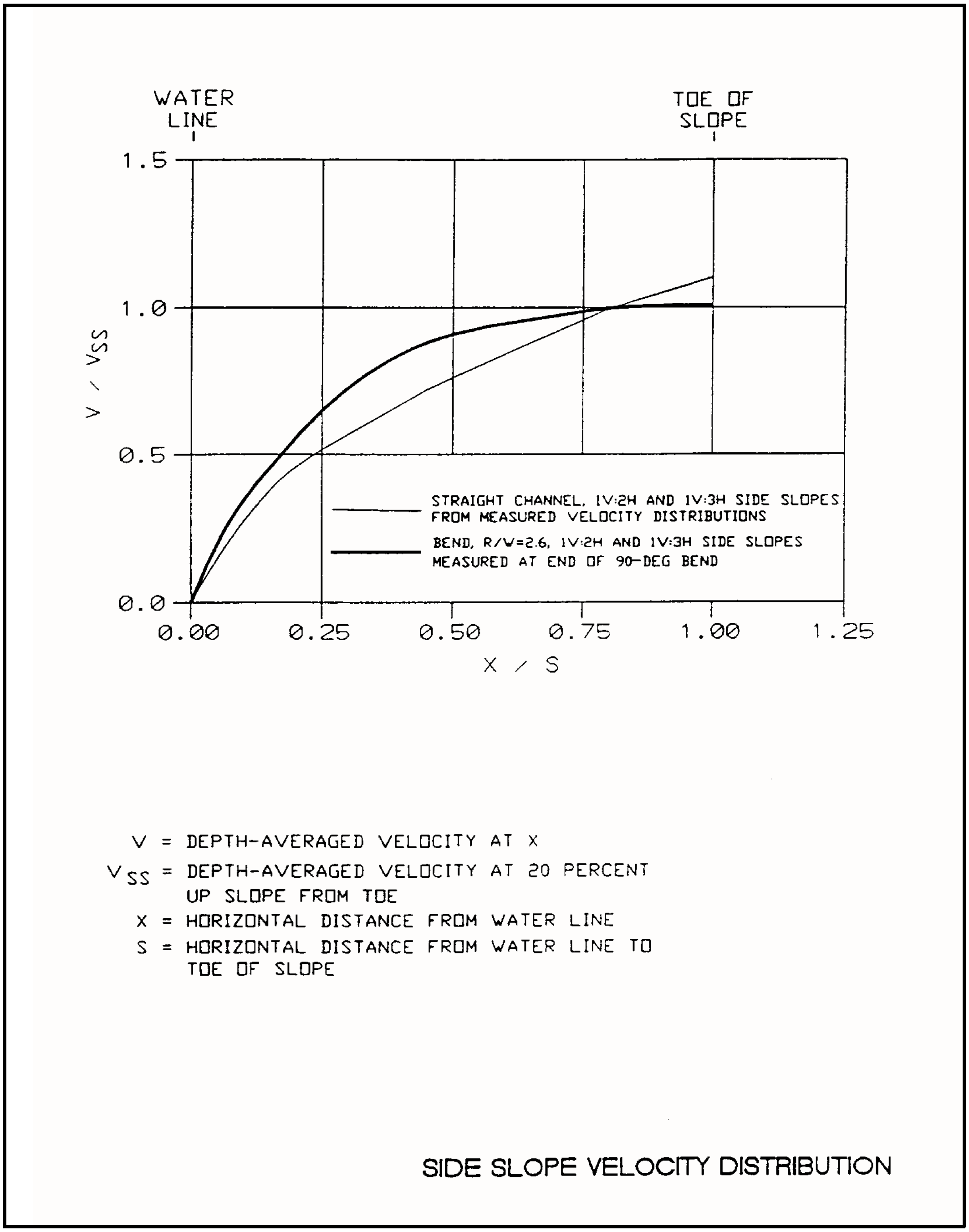


Figure A.6 Side Slope Velocity Distribution where

$D_{30}$ = riprap size of which 30 percent is finer by weight (length),
$S_f$ = safety factor (see (3) below),
$C_s$ = stability coefficient for incipient failure, thickness = $1D_{100}$(max) or $1.5D_{50}$(max), whichever is greater, $D_{85}/D_{15}$ = 1.7 to 5.2,
= 0.30 for angular rock,
= 0.375 for rounded rock,
$C_v$ = vertical velocity distribution coefficient,
= 1.0 for straight channels, inside of bends,
= 1.283 - 0.2 log (R/W), outside of bends (1 for (R/W) > 26),
= 1.25, downstream of concrete channels,
= 1.25, ends of dikes,
$C_T$ = thickness coefficient (Figure A.8),
d = local depth of flow (length),
$\tilde{a}_W$ = unit weight of water (weight/volume),
V = local area velocity, usually $V_{SS}$ (length/time),
$K_1$ = side slope correction factor (see below), and
g = gravitational constant (length/time$^2$).

This equation can be used with either SI (metric) or non-SI units.

(3) Safety Factor. The basic equation for stone size as defined by Equation (A.3) produces a rock size that is at incipient failure for $S_f$ = 1. To produce a competent riprap design, the characteristic rock size must be increased in size to resist hydrodynamic and a variety of nonhydrodynamic imposed forces and/or uncontrollable physical conditions. The size increase can best be accomplished by including the safety factor coefficient, which will be a value greater than unity. The minium safety factor is $S_f$ = 1.0. The basic safety factor may have to be increased in consideration for the following conditions:

(a) Imposed impact forces resulting from logs, uprooted trees, loose vessels, ice, and other types of large floating debris. Impact will produce more damage to a lighter weight riprap section than to a heavier section. Therefore, consideration of an added safety factor should be given to the lighter weight riprap design. For moderate debris impact, it is unlikely that an added safety factor should be used when the blanket thickness exceeds 18 inches.

(b) The basic stone sizing parameters of velocity, unit weight of rock, and depth need to be determined as accurately as possible. The ability to determine exact values is highly unlikely; therefore, a safety factor should be included to compensate for small inaccuracies in these parameters. If conservative estimates of these parameters are used in the analysis, the added safety factor should not be used. The safety factor should be based on the anticipated error in the values used. It should not be relied on to correct inaccurate or assumed stone sizing parameters. The depth-averaged velocity over the riprap is the parameter to which the rock size is the most sensitive. A 10 percent change in the velocity will result in a nearly 100 percent change in the weight limits of the riprap gradation (based on a sphere) and about a 30 percent change in the riprap thickness. The riprap size is also quite sensitive to the unit weight of the rock to be used: A 10 percent change in the unit weight will result in a 70 percent change in the weight limits of the riprap gradation (based on a sphere) and about a 20 percent change in the riprap thickness. The unit weight of the rock can be determined only from tests of samples from the quarry and is not exactly representative of all the rock that the quarry will produce. The rock size is not nearly as sensitive to the depth parameter as to the other two parameters, but depth should not be neglected. Reliable estimates for all three parameters should be made before the inclusion of a safety factor.

(c) Vandalism and/or theft of the stones is a serious problem in urban areas where small riprap has been placed. A $W_{50}$(min) of 80 pounds will reduce the effects of theft and vandalism. Sometimes grouted stone is used around vandalism-prone areas to prevent loss of riprap.

(d) The completed revetment will contain some pockets of undersized rocks, no matter how much effort is devoted to obtaining a well-mixed gradation throughout the revetment. This placement problem can be assumed to occur on any riprap job to some degree but probably more frequently on jobs that require stockpiling or additional handling, and where improper placement methods are used. Therefore a larger safety factor should be used when stockpiling, additional hauling, or poor placement methods will be used.

(e) The safety factor should be increased where severe freeze-thaw is anticipated.

The safety factor based on each of the above considerations should be considered separately and then the largest of these values should be used in Equation (A.3).

(4) Applications.

(a) The outer bank of straight channels downstream of bends should be designed using velocities computed for the bend. The $K_1$ side slope factor is normally defined by the Carter et al. (1953) relationship

$$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \tag{A.4}$$

where

$\theta$ = angle of side slope with horizontal, and
$\phi$ = angle of repose of riprap material (normally 40 deg).

Results given in Maynord (1988) show that Equation (A.4) is conservative and that the repose angle is not a constant 40 deg but varies with several factors. The recommended relationship for $K_1$ as a function of $\theta$ is given in Figure A.7 along with Equation (A.4) using $\phi$ = 40 deg. Correction for the vertical velocity distribution in bends is given in Figure A.8. Limited testing has been conducted to determine the effects of blanket thickness greater than $1D_{100}$(max) on the stability of riprap. Results are shown in Figure A.8 and require interpolation between the curves for each value of $D_{85}/D_{15}$. Gradations having $D_{85}/D_{15}$ greater than 5.2 should use the 5.2 curve. When greater blanket thickness is used, it must be realized that some rock movement will occur before the revetment becomes stable. While $D_{30}$ is considered to be the appropriate characteristic size, many designers prefer to use $D_{50}$. The required or computed $D_{30}$ can be used to determine the required $D_{50}$ according to

$$D_{50} = \left(\frac{D_{85}}{D_{15}}\right)^{1/3} D_{30} \tag{A.5}$$

(b) The basic procedure to determine riprap size using this method is as follows:

1. Determine average channel velocity for the design condition using computational methods, physical modeling or extrapolation of on-site measurement

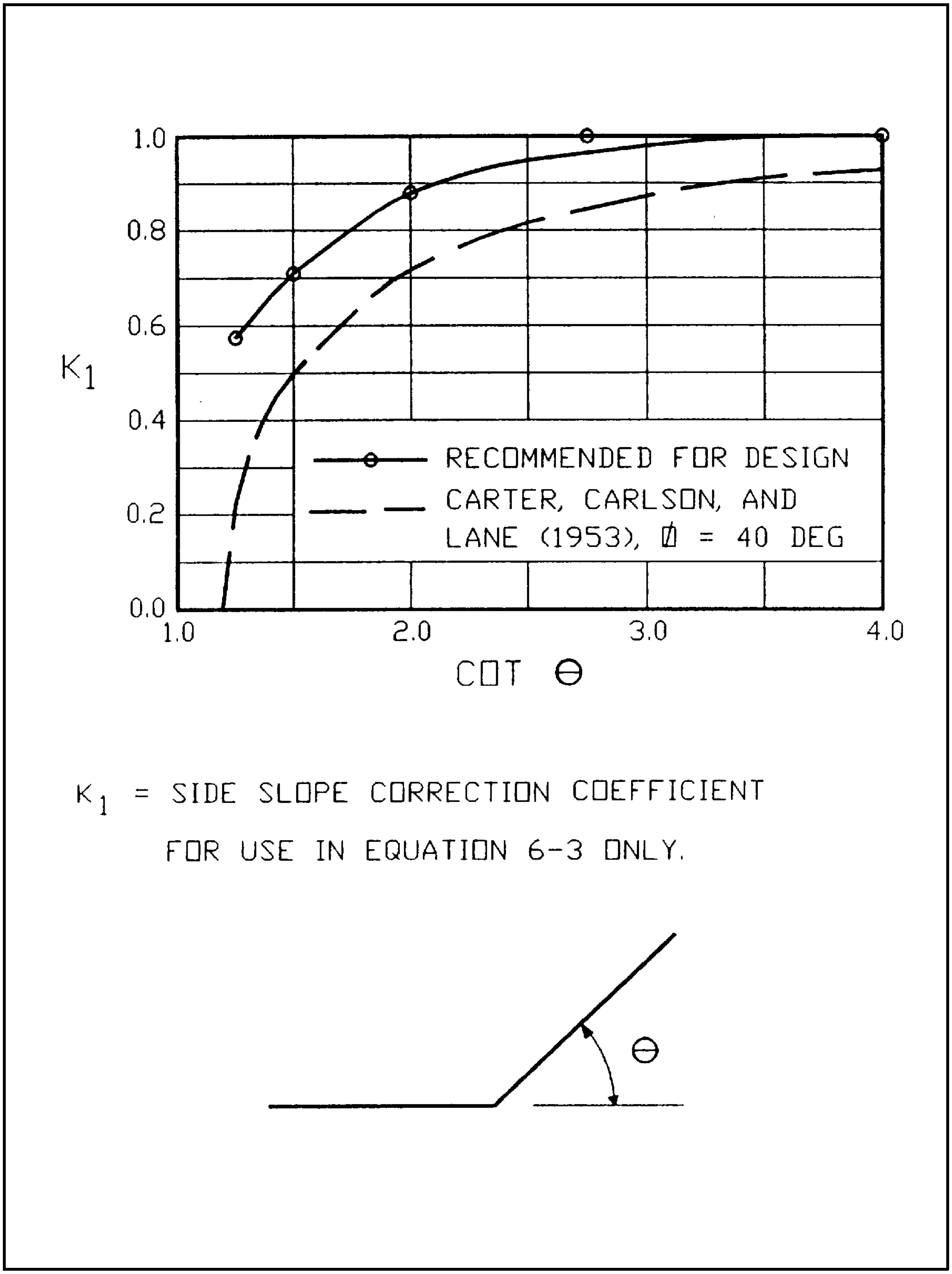


Figure A.7 Correction for Side Slope Angle

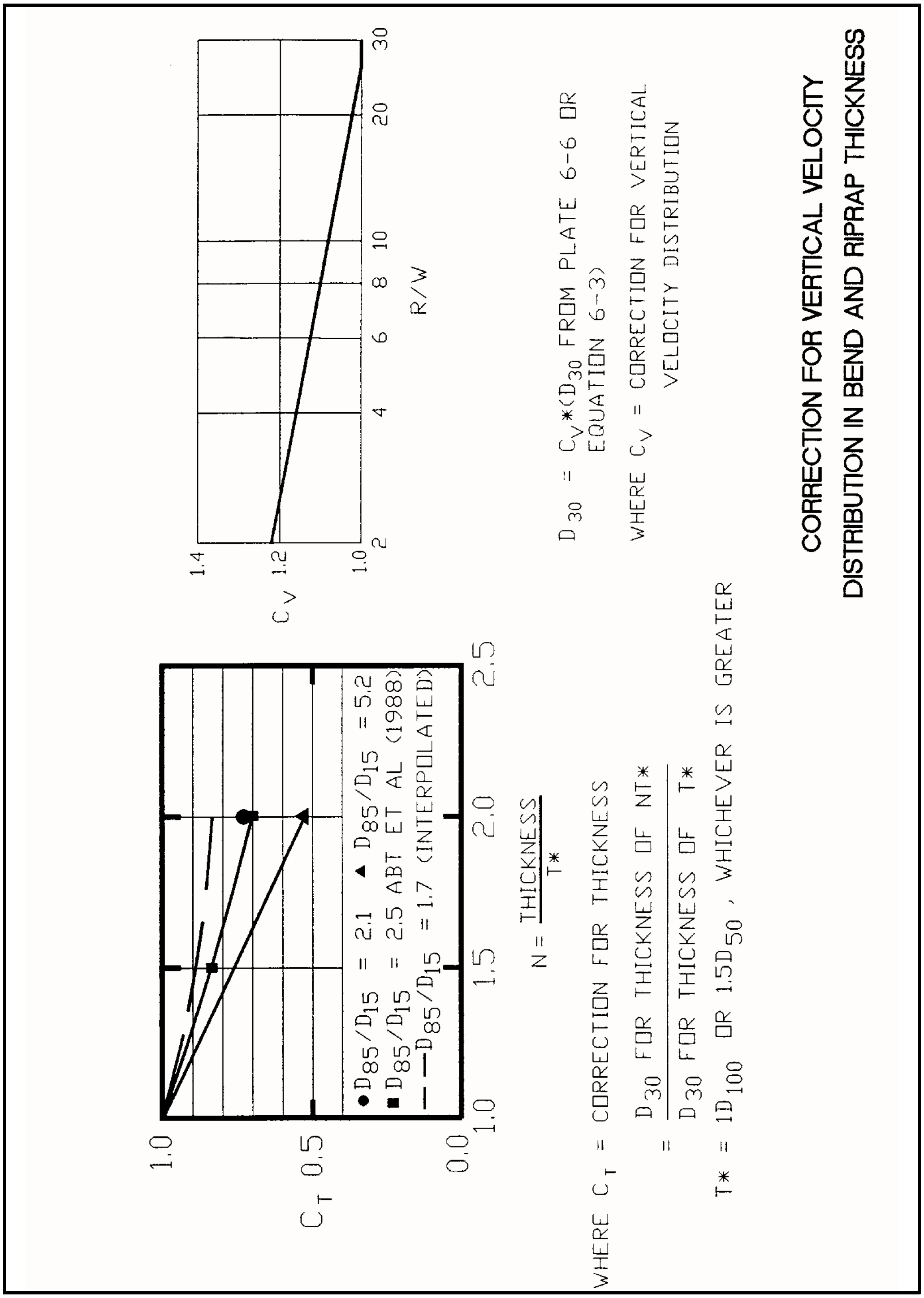


Figure A.8 Correction for Vertical Velocity Distribution in Bend and Riprap Thickness

2. Find $V_{ss}$ using Figure A.2.
3. Find $D_{30}$ using Equation (A.3).
4. Find gradation having $D_{30}$(min) \$ computed $D_{30}$.

A PC-based computer program of this procedure is available from the U.S. Army Engineer Waterways Experiment Station.

(c) This procedure can be used in both natural channels with bank protection only and prismatic channels having riprap on bed and banks. Most bank protection sections can be designed by direct solution. In these cases, the extent of the bank compared to the total perimeter of the channel means that the average channel velocity is not significantly affected by the riprap. Example 1 demonstrates this procedure.

1. <u>Example 1</u>

a. <u>Problem</u>. Determine stable riprap size for the outer bank of a natural channel bend in which maximum velocity occurs at bank-full flow. Water-surface profile computations at bank-full flow show an average channel velocity of 7.1 feet per second and a depth at the toe for the outer bank of 15 feet. The channel is sufficiently wide so that the added resistance on the outer bank will not significantly affect the computed average channel velocity (true in many natural channels). Quarries likely to be used for the project have rock weighing 165 pounds per cubic foot and routinely produce the 12-, 18-, and 24-inches $D_{100}$(max) gradations shown in Table A.1. A bank slope of 1V on 2H has been selected based on geotechnical analysis. A blanket thickness of 1$D_{100}$(max) will be used in this design. Bend radius is 620 ft and water-surface width is 200 ft.

b. <u>Solution.</u> Using Figure A.2, the maximum bend velocity ($V_{ss}$) is 1.48 (7.1) or 10.5 feet per second. The side slope depth at 20 percent up the slope from the toe is 12 ft. Using Equation (A.3), the required $D_{30}$ is 0.62 ft. From Table A.1, the 18-in. $D_{100}$(max) gradation has $D_{30}$(min) = 0.73 ft is the minimum routinely produced gradation that has $D_{30}$(min) greater than or equal to 0.62 ft. In this example, the actual safety factor of (1.1(0.73/0.62)) = 1.3 results from using standard gradations to avoid the extra production costs incurred by specifying a custom gradation for every design condition.

(d) In some cases, a large part of the channel perimeter is covered with riprap; the average channel velocity, depth, and riprap size are dependent upon one another; and the solution becomes iterative. A trial riprap gradation is first assumed and resistance coefficients are computed using Equation (A.2). Then the four steps described in paragraph (b) are conducted. If the gradation found in Step 4 is equal to the assumed trial gradation, the solution is complete. If not a new trial gradation is assumed and the procedure is repeated. Example 2 demonstrates this procedure.

2. Example 2

a. Problem. Determine stable riprap size in a bend of a trapezoidal channel with essentially uniform flow. Bank slope is 1V on 2H and both the bed and banks will be protected with the same size of riprap. The bottom width is 140 ft, slope is 0.0017 ft/ft, and the design discharge is 13,500 cfs. Use $1D_{100}$(max) thickness and the same quarry as in Problem 1. Bend radius is 500 ft and bend angle is 120 deg.

b. Solution. In this problem the solution is iterative; flow depth, velocity, and rock size depend on each other. Use Strictler's equation $n = 0.036\,(D_{90}(\text{min}))^{0.166}$ to estimate Manning's resistance coefficient. Bend velocity is determined using Figure A.2. Assume trial gradation and solve for riprap size as shown in Tables A.2 and A.3. Use uniform flow computations to determine the following:

Table A.2 Uniform Flow Computations

| Trial $D_{100}$(max) (in.) | Manning's n | Normal Depth (ft)[1] | Water-Surface Width (ft) | Average Velocity (fps)[1] | Side Slope Depth (ft) |
|---|---|---|---|---|---|
| 12 | 0.034 | 10.6 | 182.4 | 7.9 | 8.5 |
| 18 | 0.036 | 11.0 | 184.0 | 7.6 | 8.8 |
| 24 | 0.038 | 11.3 | 185.2 | 7.3 | 9.0 |

[1] From iterative solution of Manning's equation $Q/A = (1.49/n)R^{2/3}S^{1/2}$.

Use velocity estimation and riprap size equations to obtain riprap size in Table A.3:

Table A.3 Velocity Estimation and Riprap Size

| Trial $D_{100}$(max) (in.) | $V_{SS}$[1] (fps) | Computed $D_{30}$[2] (ft) | $D_{30}$(min) of Trial[3] (ft) |
|---|---|---|---|
| 12 | 9.9 | 0.59 | 0.48 |
| 18 | 9.5 | 0.53 | 0.73 |
| 24 | 9.2 | 0.48 | 0.97 |

[1] From Figure A.2 using trapezoidal channel.
[2] From Equation (A.3).
[3] From gradation information given in Table A.2.

This example demonstrates that the increasing rock size for the three trial gradations results in increasing depth and decreasing velocity. The minimum acceptable routinely produced gradation is the 18-in. $D_{100}$(max).

(e) In braided streams and some meandering streams, flow is often directed into the bank line at sharp angles (angled flow impingement). Precise guidance is lacking on determining the imposed force for this condition. Until better guidance can be developed, a local velocity of 1.6 times the average channel velocity at the impingement point is recommended for use in riprap design. The discharge used for design conditions should correspond to a stage at or just above the tops of the mid-channel bars. A velocity distribution coefficient $C_V$ of 1.25 is typical of low R/W bends and should be used for impingement points in Equation (A.3).

(f) Transitions in channel size or shape may also require riprap protection. The procedures presented here are applicable to gradual transitions where flow remains tranquil. In areas where flow changes from tranquil to rapid and then back to tranquil, riprap sizing methods applicable to hydraulic structures should be used (Hydraulic Design Criteria, Chart 712). In converging transitions, the procedures based on Equation (A.3) can be used unaltered. In expanding transitions, flow can concentrate on one side of the expansion and design velocities should be increased. For installations immediately downstream of concrete channels, a vertical velocity distribution coefficient of 1.25 should be used due to the difference in velocity profile over the two surfaces.

## A.5 FILTER REQUIREMENTS

Guidance for filter requirements for riprap and other armors is given by USACE (1986), and Pilarczyk (1984).

## A.6 REVETMENT TOP AND END PROTECTION

### A.6.1 REVETMENT TOP

When the full height of a levee is to be protected, the revetment will cover the freeboard, i.e., extend to the top of the levee. This provides protection against waves, floating debris, and water-surface irregularities. Factors which determine whether protection can be terminated below design flowline are discussed in 6.2.5 of the main text. Figure A.6 provides general guidance for velocity variation over channel side slopes that can assist in evaluating the economics of reducing or omitting revetment for upper bank areas. Gradation and thickness reduction should not be made unless a sufficient quantity is saved to be cost effective. A horizontal collar or key at the top of the riprap can be provided to protect against escaping and returning flows by adapting the end protection methods illustrated in Figure A.9.

### A.6.2 REVETMENT END PROTECTION

The upstream and downstream ends of riprap revetment should be protected against erosion by increasing the revetment thickness T or extending the revetment to areas of noneroding velocities and relatively stable banks. The following guidance applies to the alternative methods of end protection illustrated in Figure A.9.

Method A. For riprap revetments 12 in. thick or less, the normal riprap layer should be extended to areas where velocities will not erode the natural channel banks.

Method B. For riprap revetments exceeding 12 in. in thickness, one or more reductions in riprap thickness and stone size may be adopted for a distance a (Figure A.9) in which velocities decrease to a noneroding natural channel velocity, if savings in stone quantity justify the extra expenses of using more than on gradation.

Method C. For all riprap revetments that do not terminate in noneroding natural channel velocities, the ends of the revetment should be enlarged, as shown in Figure A.9. The dimensions a an b should be 3 and 2 times the layer thickness, respectively. The decision to terminate in erosive velocities should be made cautiously, since severe erosion can cause the revetment to fail by progressive flanking.

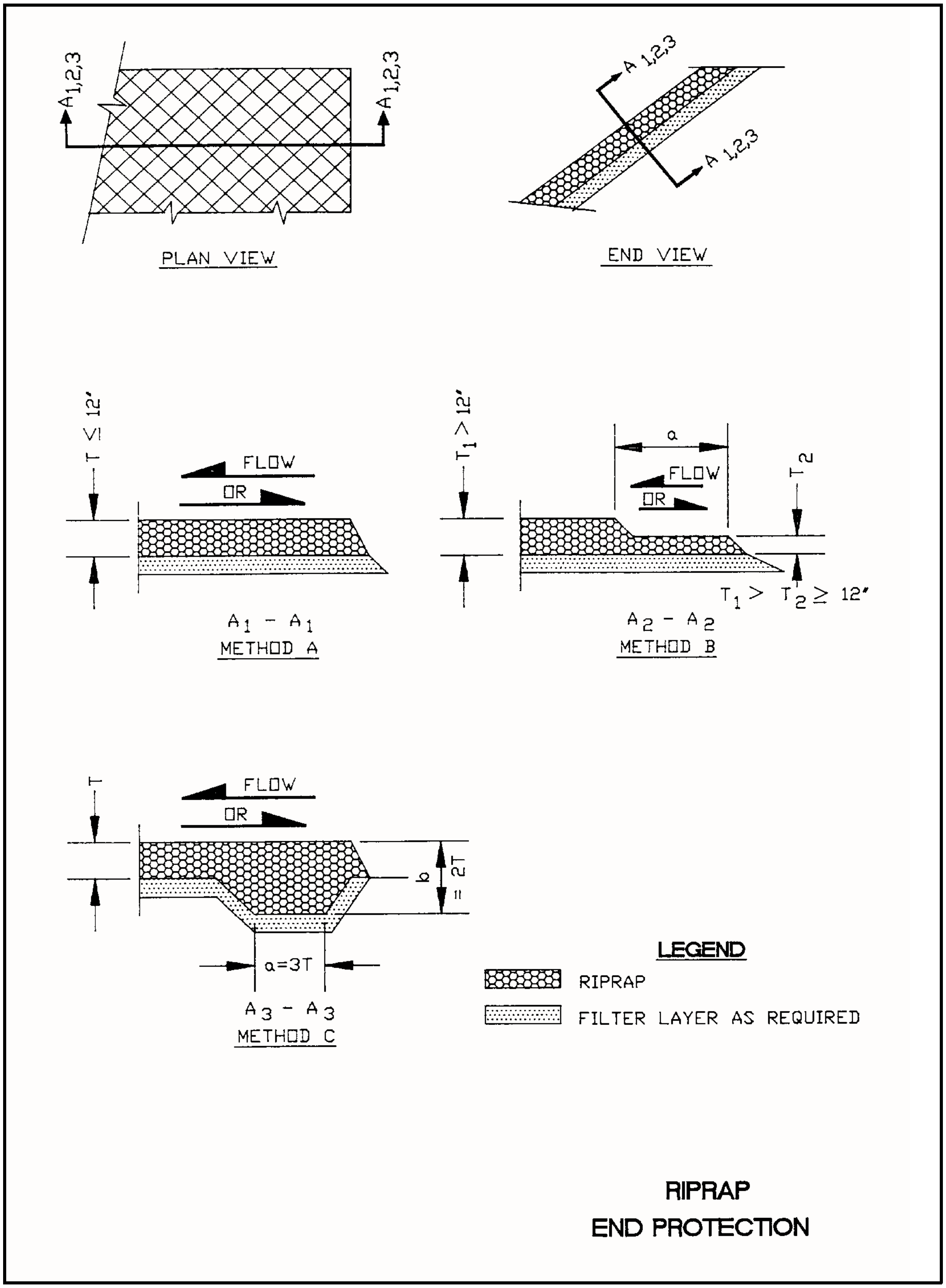


Figure A.9 Riprap End Protection

## A.7 REVETMENT TOE SCOUR ESTIMATION AND PROTECTION

### A.7.1 GENERAL

Toe scour is probably the most frequent cause of failure of riprap revetments. This is true not only for riprap, but also for a wide variety of protection techniques.

### A.7.2 REVETMENT TOE PROTECTION

Toe protection may be provided by either of two general approaches:

(1) Place the lower extremity below the expected scour depth or found it on noneroding material. This is the preferred method but it can be difficult and expensive when underwater excavation is required.

(2) Place sufficient launchable stone at the toe to arrest toe erosion before geotechnical instability occurs. This approach has been successfully used on many streams. It has experiences some failures where flow abruptly impinges on the bankline, perhaps as a result of the design not adequately accounting for this condition.

Four specific applications of these two general approaches are illustrated in Figure A.10. Methods A and B are intended to extend to the maximum depth of scour. Method C is suitable where significant toe scour is not expected. Method D can be adapted to a wide range of site conditions and scour depths. Constructability and the designer's judgement determine which method is preferable for a specific project.

Method A. When toe excavation can be made in the dry, the riprap layer may be extended below the existing groundline a distance exceeding the anticipate depth of scour. If excavation quantities are prohibitive, the concept of Method D can be adapted to reduce excavation.

Method B. When the bottom of the channel is non-erodible material, the normal riprap should be keyed in at streambed level.

Method C. When the riprap is to be placed underwater and little toe scour is expected (such as in straight reaches that are not downstream of bends), the toe may be placed on the existing bottom of height a and width c equal to 1.5T and 5T, respectively. This compensates for uncertainties of underwater placement (see A.8). When the anticipated erosion depth will undermine the stone toe, Method D should be used.

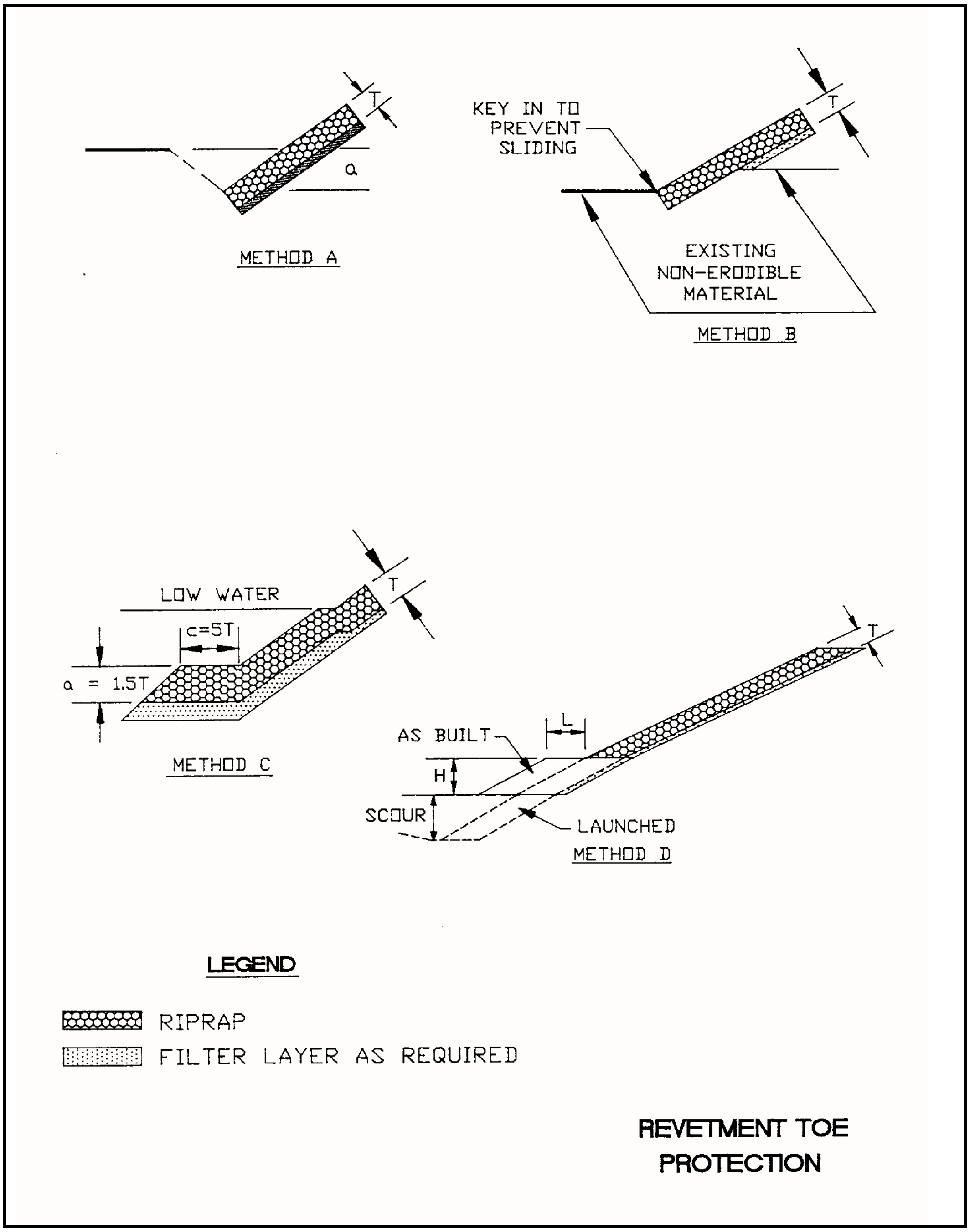


Figure A.10 Revetment Toe Protection

Method D. An extremely useful technique where water levels prohibit excavation for a toe section is to place a launchable section at the toe of the bank. Even if excavation is practicable, this method may be preferred for cost savings if the cost of extra stone required to produce a launched stone revetment is exceeded by the cost of excavation required to carry the design thickness T down the slope. This concept simply uses toe scour as a substitute for mechanical excavation. This method also has the advantage of providing a "built-in" scour gage, allowing easy monitoring of high-flow scour by visual inspection of the remaining toe stone after the high flow subsides, or by surveyed cross sections if the toe stone is underwater. Additional stone can then be added for reinforcement if necessary. This method is also readily adaptable to emergency protection, where high flow and the requirement for quick action make excavation impractical.

Shape of the stone toe is not critical. For trench-fill revetments, the height of the stone section is generally one-half to one times the length. For weighted riprap toes, heights of 2.5 to 4.0 times the bank protection thickness should be used. Providing an adequate volume of stone as discussed below is critical.

To compute the required launchable stone volume for Method D, the following assumptions should be used:

Launch slope = 1V on 2H. This is the slope resulting from rock launched on non-coehesive material in both model and prototype surveys. Launch slope is less predictable if coehesive material is present, since cohesive material may fail in large blocks.

Scour depth = existing elevation - maximum scour elevation.

Thickness after launching = thickness of the bank revetment T.

To account for the stone lost during launching and for underwater placement, the increases in stone volume listed in Table A.4 are recommended. Using these assumptions, the required stone volume for underwater placement for vertical launch distance less than 15 ft equals

$$\begin{aligned}\text{Required Stone Volume} &= 1.5\text{T times launch slope length}\\ &= 1.5\text{T times scour depth } \sqrt{5}\\ &= 3.35\text{T (scour depth)}\end{aligned}$$

A safety factor should be added if data to compute scour depth are unreliable, if cohesive bank material is present, or if monitoring and maintenance after construction cannot be guaranteed. Widely graded ripraps are recommended because of reduced rock voids that tend to prevent leaching of lower bank material through the launched riprap. Launchable stone should have $D_{85}D_{15}$ $ 2.

Table A.4 Increase in Stone Volume for Riprap Launching Sections

| Vertical Launch Distance (ft)[1] | Volume Increase Percent | |
|---|---|---|
| | Dry Placement | Underwater Placement |
| # 15 | 25 | 50 |
| > 15 | 50 | 75 |

[1] From bottom of launch section to maximum scour.

## A.8 DELIVERY AND PLACEMENT

The common methods of riprap placement are hand placing; machine placing, such as from a skip, dragline, or some form of bucket; and dumping from trucks and spreading by bulldozer. Hand placement produces the most stable riprap revetment if the long axis of the riprap particles are oriented perpendicular to the bank. It is the most expensive method except when stone is unusually costly and/or labor unusually cheap. Steeper side slopes can be used with hand-placed riprap than with other placing methods. This reduces the required volume of rock and may be an attractive alternative where rights of way are restricted. However, the greater cost of hand placement usually makes machine or dumped placement methods and flatter slopes more economical. Also, geotechnical considerations also dictate channel side slope design. In the machine placement method, sufficiently small increments of stone should be released as close to their final positions as practical. Rehandling or dragging operations to smooth the revetment surface tend to result in segregation and breakage of stone. Stone should not be dropped from an excessive height as this may result in the same undesirable conditions. Riprap placement by dumping and spreading is not recommended as significant segregation and breakage can occur. However in some cases, it may be economical to increase the layer thickness and stone size somewhat to offset the shortcomings of this placement method. Smooth, compact riprap sections have resulted from compacting the placed stone sections with a broad-tracked bulldozer. This stone must be quite resistant to abrasion. Thickness for underwater placement should be increased by 50 percent to provide for the uncertainties associated with this type of placement. Underwater placement is usually specified in terms of weight of stone per unit area, to be distributed uniformly in a “grid” pattern established by survey control.

## A.9 ICE AND DEBRIS

Ice and debris create greater stresses on riprap revetment by impact and flow concentration effects. Ice attachment to the riprap also causes a decrease in stability. One rule of thumb is that thickness should be increased by 6-12 in., accompanied by appropriate increase in stone size, for riprap subject to attack by large floating debris. Riprap deterioration from debris impacts is usually more extensive on bank lines with steep slopes. Therefore, riprapped slopes on streams with heavy debris loads should be no steeper than 1V on 2.5H.

## A.10 QUALITY CONTROL

Provisions should be made in the specifications for sampling and testing the quarry and in-place riprap as representative sections of revetment are completed. Additional sample testing of in-place and in-transit riprap material at the option of the Contracting Officer should be specified. The frequency of sample testing depends on the ease of producing riprap material that complies with the specifications. The size of the test samples should be sufficient to be representative of the riprap material. Truckload samples are usually satisfactory for in-transit material. The following tabulation provides two methods of determining the riprap sample (area versus volume) and should be used as a guide for in-place samples:

| Riprap Layer Thickness (in.) | Size of Samples | |
|---|---|---|
| | Bulk Volume (cu. yd.) | Area (ft) |
| 12 | 1 | 6 x 6 |
| 18 | 2 | 8 x 8 |
| 24 | 5 | 10 x 10 |
| 30 | 10 | 13 x 13 |
| 36 | 16 | 15 x 15 |

The primary concern of riprap users is that the in-place riprap meets specifications. Loading, transporting, stockpiling, and placing can result in deterioration of the riprap. Coordination of inspection efforts by experienced staff is necessary. Additional guidance can be found in EM 1110-2-2302.

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## APPENDIX B

# BIOENGINEERING FOR STREAMBANK EROSION CONTROL

### Guidelines

# 1 Introduction

## Background

Corps of Engineers (CE) and others are often restricted from using hard structures, such as riprap or concrete lined channels, for streambank erosion control partly because of environmental reasons and high cost. Within the last decade or so, increased demands have been placed upon the CE by environmental agencies and others to incorporate vegetation into their streambank erosion control projects rather than to rely completely on traditional methods. Complete bank armorment by various methods such as riprapped revetment, concrete revetment, bulkheads, concrete linings, etc. are considered by many to have little value for fisheries, wildlife, water quality, and aesthetic appeal. Bioengineering, in contrast, is receiving more emphasis from environmental agencies and conservation organizations. Bioengineering is the combination of biological, mechanical, and ecological concepts to control erosion and stabilize soil through the sole use of vegetation or in combination with construction materials. Both living and non living plants can be used. Non-living plants are used as construction materials, similar to engineered materials. The Planted vegetation controls erosion and serves as good wildlife and fisheries habitat in riparian systems.

A limited number of streambank erosion control projects have been designed and implemented by the CE where bioengineering has been purposely planned as a part of the project. The CE has historically relied on construction projects with design lifes of 50 to 100 years that require a minimum amount of maintenance. Therefore, the focus of development has been on hard structures that can be modeled and studied in hydraulic flumes and other test structures and are designed to stay in place a long time. The CE has been reluctant to design softer treatments, e.g., bioengineering, for erosion control because of a dearth of specific design guidance. For instance, under what velocity conditions will certain vegetative treatments work? This type of information has been slow to develop. In part, a lack of monitoring after streambanks have been treated with a vegetative method has led to unknown performance conditions and failure thresholds. In 1993, efforts were taken under the purview of the Environmental Impact Research Program (EIRP), sponsored by Headquarters, US Army Corps of Engineers, to develop and demonstrate bioengineering concepts for streambank erosion control and to determine hydraulic velocities and conditions for successful prototype performance and use.

## Purpose

This report synthesizes information related to bioengineering applications and provides preliminary planning and design guidelines for use of bioengineering treatments on eroded streambanks. It can be used by both planning and design elements, not as a cookbook, but as a guide with tools to accomplish bioengineering projects. It presents a bioengineering design model with examples in the text that describe specific case studies where certain stream conditions, such as velocities, have been provided. It also describes appropriate plants to use, their acquisition, and their handling requirements.

This report is divided into two volumes. The main report, Volume I, provides bioengineering guidelines for streambank erosion control. Volume II presents several case studies of bioengineering treatments applied to one or more streams in various geographic locations around the continental United States.

## Scope

The authors of this report do not attempt to assume that bioengineering for streambank protection is a cure unto itself. First, bed stability, another whole subject area, must be achieved before banks are addressed. If streambeds are not stable, it does little good to attempt bank stabilization. This report does not attempt to address the details of fluvial geomorphology, but the authors recognize that bioengineering must be done in consonance with good river bed and planform stability design and there are several texts and engineer manuals that address these subjects. Consequently, good bioengineering takes an interdisciplinary team approach with expertise representing engineering, physical, and biological fields, as well as others, a point re-emphasized throughout this report. The authors also recognize that causes of streambank erosion are complex and can often be related to land-use practices being conducted in the watershed and/or in the immediate vicinity of the erosion problem on the streambank. Therefore, careful study should be made of the causes of erosion before bioengineering is contemplated. Again, an interdisciplinary team is often required to develop an optimum plan. Bioengineering must be done within the context of a landscape approach, but erosion control must be addressed by reaches, from a practical standpoint. The report provides a planning sequence, or bioengineering design model, that is tailored to a zonal approach within reaches.

Vegetation, per se, is not a panacea for controlling erosion and must be considered in light of site-specific characteristics. When vegetation is combined with low cost building materials or engineered structures, numerous techniques can be created for streambank erosion control. This report summarizes a number of techniques that utilize vegetation. For understanding how vegetation can be used in bioengineering and as a basis for conceptualizing a bioengineering design model, it is important to understand both the assets and limitations of using planted vegetation.

## Assets of Using Planted Vegetation

Gray (1977), Bailey and Copeland (1961), and Allen (1978) discuss five mechanisms through which vegetation can aid erosion control: reinforce soil through roots (Gray, 1977); dampen waves or dissipate wave energy; intercept water; enhance water infiltration; and deplete soil water by uptake and transpiration. Klingman and Bradley (1976) point out four specific ways vegetation can protect streambanks. First, the root system helps hold the soil together and increases the overall bank stability by its binding network structure, i.e., the ability of roots to hold soil particles together. Second, the exposed vegetation (stalks, stems, branches, and foliage) 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. Third, the vegetation acts as a buffer against the abrasive effect of transported materials. Fourth, 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.

## Limitations of Using Planted Vegetation

Using planted vegetation for streambank erosion control also has limitations. These may include its occasional failure to grow; it is subject to undermining; it may be uprooted by wind, water, and the freezing and thawing of ice; wildlife or livestock may feed upon and depredate it; and it may require some maintenance. Most of these limitations, such as undermining, uprooting by freezing and thawing, etc., can often be lessened or prevented by use of bioengineering measures.

# 2 Bioengineering Design Model

A conceptual design model is offered below that leads one through the steps of planning and implementing a bioengineering project. It draws largely upon similar thought processes presented by Leiser (1992) for use of vegetation and engineered structures for slope protection and erosion control. Where appropriate, the report will reference examples in the main text (Vol. I) and case studies (Vol. II) that describe particular bioengineering treatments on selected and monitored stream systems. The model includes planning and its associated components that will be defined below; use of hard structures and bioengineering; a vegetative zonal concept; and various bioengineering fixes by zone. Monitoring, followup, and care should naturally follow.

## Planning

A bioengineering project may be primarily desired for erosion control, but often there are other considerations. Thought should be given to important functions that the bioengineering treatment can perform, such as habitat development, archeological site protection, water quality improvement, aesthetics, or a combination of these. The political and economical requirements or constraints of implementing any project must be considered. Any bioengineering streambank stabilization project should be planned within the context of the landscape in which the stream is located. Activities near the stream that is influencing its erosion must be identified. It is a wasted effort to install bioengineering treatments in an area where cattle are allowed access to the treated reach immediately after construction. The stream must be examined as a system, but the restoration must be accomplished at the reach level from a practical perspective. The planning part of the model should address potential functions of the treatment and the political and economical concerns (Figure 1).

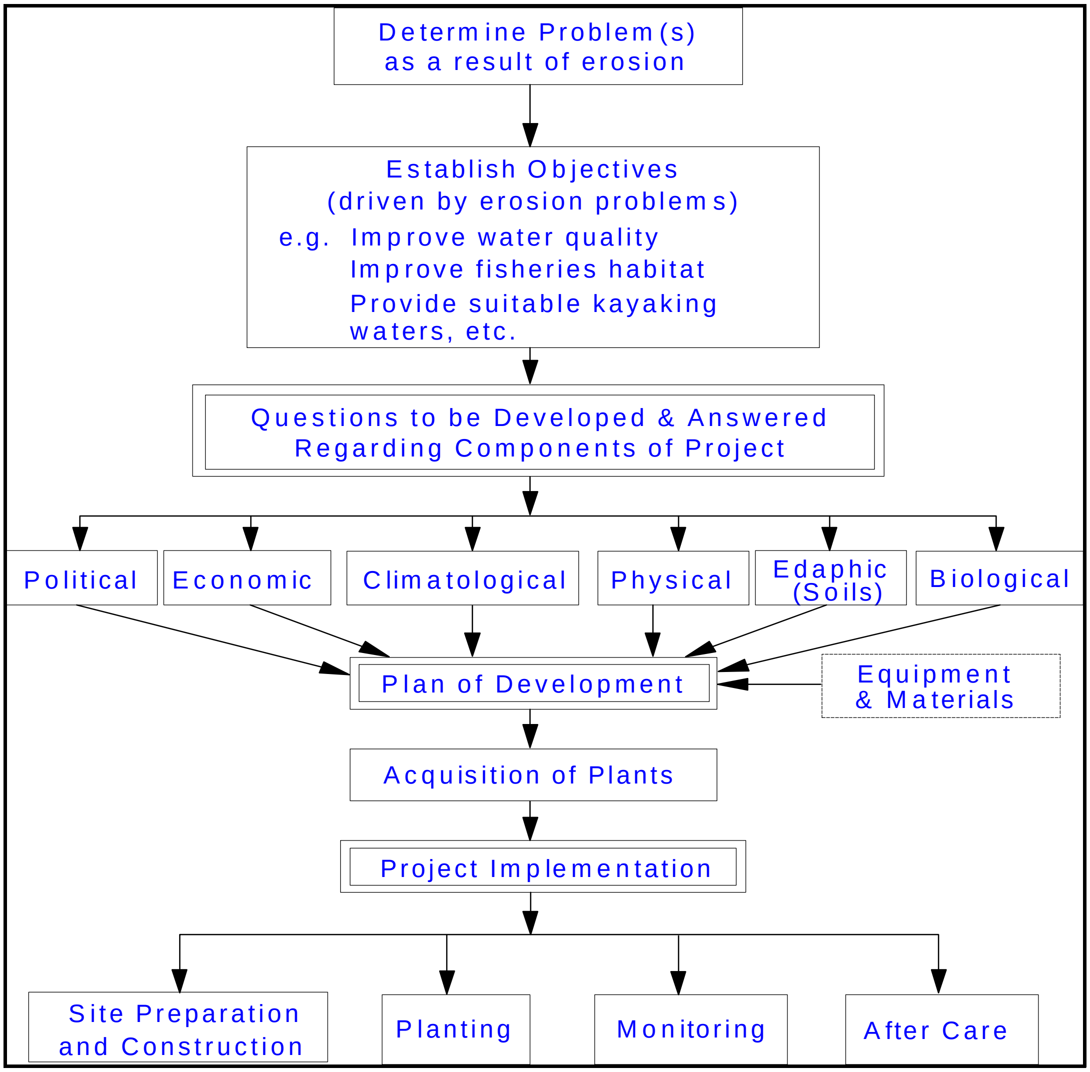


**Figure 1.** Steps of Planning and Implementing a Bioengineering Project

## Determine Problem(s) and Establish Objectives

Clear-cut objectives that are based on some perceived problem or problems are needed for any project. The problem or problems may be results of erosion, such as poor water quality, lack of fisheries, lack of suitable water for kayaks, and others. The objectives are then driven by these and may relate to primarily erosion control, but may also include providing fisheries or wildlife habitat, improving water quality, protection of cultural resources, or a host of other desired functions. Bowers (1992) established objectives on the Little Patuxent River, Maryland, that included not only erosion control, but also in-stream and riparian habitat enhancement. These objectives are often driven not only by the physical impacts of erosion on the landscape, but by legal mandates, such as mitigation for some action on the stream. Questions must be asked and answers provided before the project can proceed. This effort will require that an interdisciplinary team be developed consisting minimally of engineers, hydrologists, and life scientists with expertise in bioengineering approaches. Other disciplines, such as economists, sociologists, and attorneys can be consulted as needed during the planning stage of development.

## Questions to be Developed and Answered.

Any streambank erosion control project has several components. Each component may have constraints that have to be overcome. These components with associated constraints are interdependent and must be considered, thus generating an abundance of questions that should be answered, if possible. They include the political, economic, climatatological, physical, edaphic (soils), and biological components of the project. Both the asking and answering of these questions relative to these components lead to the Plan of Development. Once the plan is developed and permits acquired, procurement of plants will be required (See Part III). After or concurrent with this procurement, implementation of the plan can proceed. The political component includes governmental regulations, such as those presented in Section 404 of the The Clean Water Act (formerly known at the Federal Water Pollution Control Act, 33 U.S.C. 1344). It also includes public pressures, such as restricting bioengineering to the use of only native plant species or plants that are grown in a nursery as opposed to those harvested from the wild. Governmental regulations and/or public pressures may also mandate that certain vegetation species or types of species be used. If a certain species blocks the view of a river in an urban setting, for instance, public pressures may cause plans to change to use a different species or a different erosion control treatment altogether. Lack of grazing controls, limitations on use of chemicals for rodent, insect, or weed control or fertilizers are other examples of these constraints and must be considered in any bioengineering design criteria protocol. The political component also includes the negative human factors of vandalism and trespass by foot and off road vehicles as well as the positive factor of public pressure for improvement of the environment.

The economic component could be one of the more important factors to enable bioengineering erosion control efforts. Usually, bioengineering projects are less expensive than traditional engineering approaches. However, economics invariably affects the final decisions on the selection of plant species and planting densities, as well as pre-project experimentation and after care activities. A bioengineering design protocol must include funding for monitoring and allow for remedial planting and management of the site to meet the objectives of the project. Bioengineering projects will often require more funds early in the project's history for possible repair and assurance of effectiveness than traditional engineering, but will be more self sustaining and resilient over the long term. If traditional engineering projects need remediation over the life of the project (and they frequently do), the remediation occurs later in the life of the project but with higher overall costs.

The climatatological component includes several aspects of a project site: rainfall (amount and distribution), temperature (heat and cold, time, duration, and intensity), humidity, day length, etc. Climatatological components affect plant species selection, how those plants will be planted, and treatment after planting. With some exceptions, bioengineering projects in humid regions with ample rainfall and projects along permanent flowing streams will probably require less effort to establish vegetation than those along intermittent flowing streams in dry climates. In desert climates, where fewer plants in the inventory can be chosen than in humid climates, learning these plants' life requisites is essential for successful planting. The probability for bioengineering project failure is higher with fewer species planted and where growth stresses are greater.

The physical component includes physical parameters of a project: site stability such as subsidence or accretion; aspect (direction slope faces), which in turn influences environmental factors such as temperature (south and southwest facing sites are hotter and evapotranspiration is higher than on other directions); hydrodynamic aspects such as water sources (groundwater, surface water), and water frequency, timing, depth, duration and flooding relationship to bank height; fluvial geomorphology, such as historical stream meander, pattern, cross-sectional and longitudinal profiles, and energy sources such as wave and current action; and geomorphic features, such as landforms and terrain influences, e.g., impacts of off-site water sources.

From the above list of physical parameters, hydrologic and geomorphic factors are particularly important. For purposes of determining where to use vegetation on the bank and the kinds of vegetation to use and when to plant, one needs a knowledge of the stream's hydrographic and fluvial geomorphic characteristics. If stream guage data are not available, one will have to rely on high water marks, the knowledge of persons living in the areas, and any other data derived from local vegetation and soils that indicate flood periodicity. Table 1 gives an example of hydrologic characteristics of the upper Missouri River. It shows the frequency of various flows and their duration with 25,000 cfs being the normal flow from late spring through fall. A 40,000 cfs flow with a duration of 6 months can be expected to occur once every 10 years. Figure 2 subsequently shows the approximate water level corresponding to various river flows using the level of 25,000 cfs as the reference. At a flow of 40,000 cfs, the river level will be approximately 3 feet above the reference level. From other data, we also know that flows exceed normal usually in June or July; therefore, planting should occur in early spring or fall.

This data also gives information that leads to forming vegetation planting zones. We know that for this example, a daily high flow of 35,000 cfs translates to a zone 2-ft high on the bank that could occur once for 60 days every two years. This means that this zone will have to be vegetated with extremely flood-tolerant vegetation, e.g., emergent aquatic species, willow (*Salix* spp.), and is equivalent to a “splash zone” that will be discussed later.

**Table 1. Recurrence interval by discharge and duration on upper Missouri River***

| Discharge | Duration 6 Months | Duration 60 Days | Duration 1 Day | Probability Of Not Occurring (60 Days) |
|---|---|---|---|---|
| 60,000 CFS | --- | 1/100 Yrs. | 1/20 Yrs. | 99% |
| 50,000 CFS | 1/100 | 1/10 | 1/5 | 90% |
| 40,000 CFS | 1/10 | 1/3 | 1/2 | 67% |
| 35,000 CFS | 1/3 | 1/2 | 1 | 50% |
| 30,000 CFS | 1/2 | 1 | 1 | 1% |
| 25,000 CFS | 1 | --- | --- | --- |

(Normal Flow Levels) - occurs generally from April 15 - May 15 until No. 15

CFS = Cubic feet per second

* From Logan et al. (1979 op cit.)

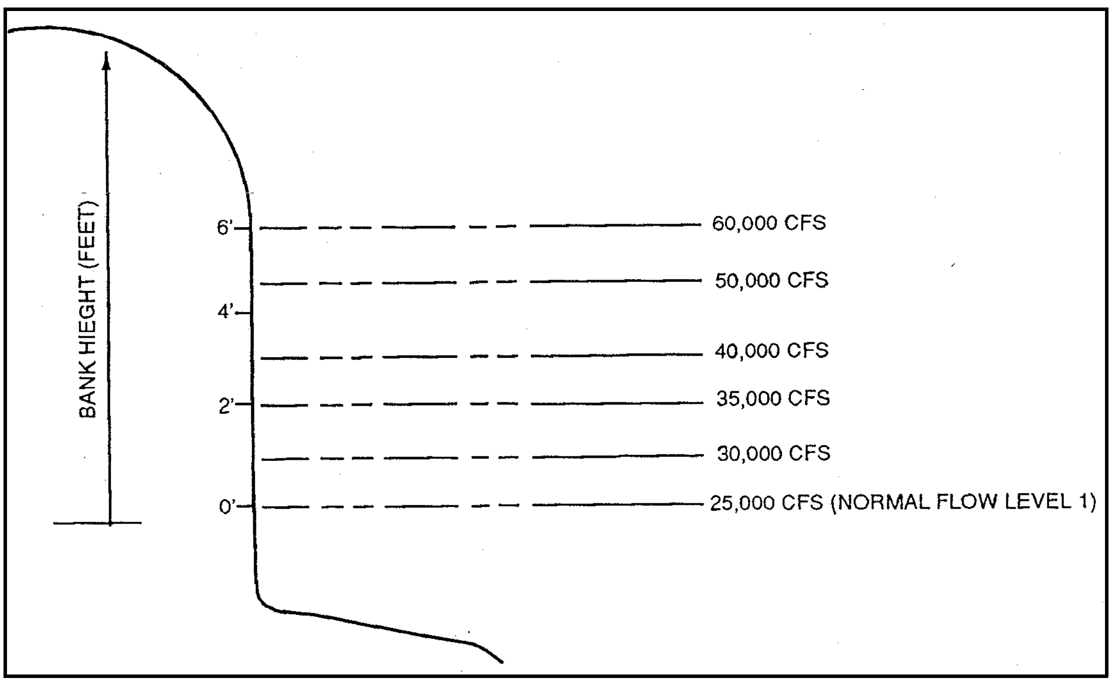


**Figure 2.** River levels and flows of upper Missouri River below Garrison Dam

Geomorphic characteristics such as bank geometry play a major part in the employment of bioengineering. Banks that have been eroded and undercut to a very steep, unplantable slope require grading prior to planting (Edminster et al. 1949 and Edminster 1949). The angle required varies with the soil, equipment used, and several other factors. Sand, for instance, has an angle of repose of about 30 degrees whereas clay can stand on a much steeper angle (Gray 1977). Most slopes that accommodate revegetation are less than 1-1.2 V:1 H. On steep banks where undercutting may be a problem, the toe of the bank may need protecting with riprap or other hard, structural treatments. Special structural treatments other than vegetation and drainage structures may be necessary where geomorphic features contribute to internal erosion of the bank, called piping or sapping. This is where water can seep into the bank from higher elevations through porous strata and cause bank failure when the erodible strata are gone. Sometimes, bioengineering with appropriate geotextile filters can treat piping problems, but not always.

The edaphic component includes all the soil parameters: texture, structure, fertility, erodability, chemistry, etc. Soil texture, structure, and depth all affect the water holding capacity of a soil and need to be considered when determining water retention requirements or supplemental irrigation requirements during dry periods of the year. In addition to ensuring proper bank slopes and bank toe protection, attention should be given to the edaphic component that may in turn require some site preparation activities. It is desirable to have slopes covered with at least a 10-cm layer of topsoil high in organic matter; this can be stockpiled prior to any grading. Movement of soil, however, is expensive and must be considered in light of the economic practicality. In lieu of moving rich topsoil, the existing substrate may be amended with fertilizer and mulch to help produce a better soil. In any case, plants need a growing medium that supports the plant and facilitates nutrient and water uptake. The site may require other soil amendments such as lime, gypsum, or other special nutrients depending upon the soil's pH and fertility. Soil tests should be conducted prior to revegetation to determine any amendments needed.

The biological component is one of the most important components and is interdependent with the other components. It includes habitat requirements of animal and plant species and the plan can be modified to some extent to meet these requirements if the life requisites of these species are known. This component also includes the availability of suitable plant species that, in part, make up the habitat for various riparian animals. Choices must be made between native and introduced species, plants obtained from commercial nurseries, or from the wild. This component also includes the propagation and cultural practice for the plants, planting, and aftercare. It includes plant diseases, insects, predators, and the presence or absence of grazing animals. An example of spider mite damage is presented in the case study of Court Creek, Illinois, Volume II, where willow had to be sprayed with an insecticide to control damage. If spraying had not occurred, streambank protection with living willow would not have been achieved. Protective screen sleeves or deer and grazing animal exclosures must be provided if these risks are present. The potential for damage from insect, rodent, deer, and other predation must be considered and protection provided to planted wetland vegetation.

The biological attributes of an area containing a bioengineering site are very important and plants are no exception. They are there because they have adapted to the ecological conditions of the area, such as climate, soils, etc. To use bioengineering effectively, one should learn to identify and evaluate plants that are growing in the area that have become adapted. These should include plants that are growing along all parts of the streambank, lower, middle, and upper. In bioengineering, these conditions and species should be emulated as much as possible. Native plants or plants that have become naturalized in the area should normally be used. Exotic plants should be avoided since there are species that may get out of control and become nuisances. One only has to look at examples such as purple loosestrife (*Lythrum salicaria*) to gain an appreciation of the problems exotic plants can cause.

Plants chosen should have some tolerance to flooding. Some will need to be highly tolerant (those planted lower on the bank) while others (those planted higher on the bank) can be less tolerant. Plants chosen also will have to withstand some dry conditions as well as flooded conditions because of the fluctuating nature of water levels in streams.

A mixture of grasses, herbs, shrubs, and trees should be used, if possible, to provide a diversity of wildlife habitats. Some legumes such as yellow sweet clover (*Melilotus officinalis)*, white sweet clover (*M. alba)*, and crownvetch (*Coronilla varia)* are possible choices because of their nitrogen-fixing attributes. These, however, should be used at an elevation subject to only intermittent and short periods of flooding, such as in the upper bank and terrace zones discussed below.

## Plan of Development

The plan of development is the culmination of answering all the questions in the various categories mentioned above. Many of the questions regarding the above components can be answered off site, but a site analysis is mandatory before plants can be procured or before project implementation can occur. In the site analysis, each component must again be examined to include the various factors or parameters and what will influence vegetation development for bioengineering and the stability of a streambank. A general guideline for the site analysis is to be a keen observer as to the conditions occurring at the project reach as well as upstream and downstream from it. From observations of a reference site, many answers can be found about what kinds of plants to use, invader species that are apt to occur, causes of problems, such as overgrazing, road construction upstream contributing to a high bed load of sediment, etc. The same or similar plant species that occur at the reference site should be acquired. In a site analysis, much of the data from a reference streambank area can be taken to answer the questions posed.

## Equipment and Materials

In the plan of development, consideration should be given to the equipment and materials required for vegetation handling and planting at the implementation stage. The tools required and the planting techniques will depend on the type of vegetation, i.e., woody or herbaceous, the size of plants, soils, and the size of the project and site conditions. Freshwater herbaceous plantings with low wave or current energy environments may call for tools like spades, shovels, and buckets. In contrast, high energy environments of waves and currents may require tools for bioengineering installations. Such tools include chain saws, lopping and hand pruners for the preparation of woody cuttings, and materials for woody bioengineering methods; or heavy hammers and sledges for driving stakes in bioengineering treatments such as wattling and brush matting. Specialized equipment may be required. This is true when moving sod or mulches containing wetland plants or plant propagules. It is also true since bioengineering projects often have the constraints of working in a pristine stream system where riparian corridors are extremely valuable, particularly in large, urban settings. It is in these settings that equipment size and type constraints are often placed upon the project. Thus, downsized front-end loaders and walking excavators are sometimes required to minimize disturbance of existing vegetation and soil. Other equipment and materials may include fertilizers, soil amendments, (i.e. lime), fencing for plant protection, and irrigation equipment for keeping plants alive during dry conditions. Other equipment and materials for keeping plants alive before they are planted may include shading materials such as tarps, buckets with water for holding plants, and water pumps and hoses for watering or water trucks.

## Permit Acquisition

After the site analysis and bioengineering actions are determined, necessary permits must be obtained, such as those governing any action impacting wetlands, water quality, cultural/historical resources, threatened and endangered species, and navigation. Usually and especially on smaller streams not requiring large structures or bank shaping as a part of the design, the permit process will not be very complex or time consuming. However, on large streams where deflection structures are employed or where banks are extensively shaped, navigation, cultural resource, and wetland permits can take several months to acquire.

## Acquisition of Plants

Prior to the implementation of the project, the plans for acquiring plants must be made well in advance (sometimes 1 - 2 years). To select vegetation for the project, vegetation existing on or near a site and on similar nearby areas which have revegetated naturally are the best indicators of the plant species to use. If commercial plant sources are not available (USDA, Soil Conservation Service, 1992), then on- or off-site harvesting can be considered. When acquiring plants, care must be given to local or federal laws prohibiting such plant acquisition and decimating the natural stands of wetland plants. Additionally, care must be taken to assure that pest species, such as purple loosestrife, are not collected and transferred to the project site.

The availability of plants of the appropriate species, size, and quality is often a limiting factor in the final selection and plant acquisition process. Some native plant species are very difficult to propagate and grow and many desirable species are not commonly available in commerce, or not available as good quality plants. As demand increases and nurserymen gain more experience in growing native plant species, this limitation should become less important (Leiser, 1992). Plant species composition and quantity can often be determined from the project objectives and functions desired. As a general rule, it is advisable to specify as many species as possible and require the use of some minimum number of these species. Maximum and minimum numbers of any one species may be specified. See Part III for additional information on plant acquisition, times of planting, and plant handling techniques.

## Implementation

Implementation is the natural followup to the plan of development and is integrated with the planning process. It cannot really be separated from it. It is the final stage of the conceptual and detailed design but may require feedback into design plan formulation for possible on-site corrections. It includes site preparation and construction, planting, monitoring, and aftercare. For the bioengineering design to be successful, it must have close supervision throughout by someone familiar with implementation of bioengineering projects. This stage requires close attention to detail. It is important in this stage to maintain continuity of the same interdisciplinary team who planned and designed the project and keep them involved in this part of the project. Those capable of actually carrying out the project should be a team of persons with knowledge and experience of both stream morphology and mechanics, hydraulic and geotechnical engineering, and bioengineering. Regarding vegetation, the person should possess both training and experience in wetlands plant science and development. It is mandatory that person be on site intermittently at least during project construction and especially planting. All of the efforts to address the various components of design will be in vain unless plants are handled and cared for properly when planted and even after planting in many cases.

## Planting Techniques

There are several planting techniques for bioengineering ranging from simple digging with shovels or spades and inserting sprigs (rooted stems) or cuttings to moving large pieces of rooted material, such as sod, mulch, and root pads (large rooted shrubs). Other methods consist of direct seeding, hydroseeding, or drilling individual seeds such as acorns of wetland oak species. All of the above methods capitalize on combining the attibutes of plants with some kind of engineered material or structure or relying on the plant itself to form a resistant structure to erosion, such as a live willow post revetment. Various techniques will be discussed in detail below.

## Monitoring and Aftercare

Most importantly, monitoring and necessary aftercare must be a part of any bioengineering design and must be included in the plan of development and the implementation stage. The intensity and frequency of monitoring and aftercare will depend on site conditions, such as harshness of climate, probability of animal disturbance, high wave or current conditions, etc., and on established success criteria.

On many sites, it is essential to protect plantings from damage by animals, such as Canada geese (*Branta canadensis*), or beaver (*Casta canadensis*) and other mammals. The use of irrigation may be required during aftercare and will improve growth and survival of plantings that are installed during dry seasons and in dry soils. The decision about irrigation must be made based on economics contrasting the need to irrigate with the cost of possible mortality and the consequences of failing to obtain the desired erosion control and other functions. See Part IV for more detail on monitoring.

## Hard Structures and Bioengineering

Generally speaking, bioengineering is considered "a soft fix." This is not necessarily the case. On first or second order streams, the sole use of vegetation with perhaps a little wire and a few stakes for holding the vegetation until it is established makes bioengineering more of a soft treatment. However, bioengineering is used also in combination with hard structures. These hard structures are used to protect the toe of the bank from undercutting and the flanks (ends of treatment) from eroding. The larger the stream or stronger the flow, the more probable that hard structures will be incorporated into the bioengineering design model. This is also true when risks become greater, such as when an expensive facility is being threatened. As an example, a utility tower along a stream in Georgia[1] was being threatened by erosion. A rock revetment had previously been used in front of the tower, but was washed out. A bioengineering treatment that incorporated live willow whips and a log crib were installed to control erosion. Crib logs controlled undercutting and flanking while the live willow whips installed between the log stringers developed and strengthened the overall structure and gave it a "green" appearance.

In most of the case studies presented in Volume II, and in the references made to other bioengineering streambank erosion control, hard structures such as rock riprap, log/tree

[1] Ms. Robin Sotir, personal communication, President, Robin Sotir and Associates, Marietta, Georgia

revetments, tree butts, and deflection dikes were used to protect toes from being undercut or flanks at the upper and lower ends from being washed out. In these cases, water currents are prevented from undercutting the bank either through direct protection of the lower bank with some hard structure or material or through some kind of deflection structure that deflects the currents off of the bank. Deflection structures may be some kind of spur dike, vane, transverse dike, or bendway weir. Figure 3 shows two timber cribs serving as deflection structures on the upper Missouri River to direct current away from the bank. In the case of hard toes on the lower bank, plants and engineered materials to hold them in place are positioned above the hard toe. Rock riprap keyed into the bank at both the upper and lower ends of a bioengineering treatment are called refusals (Figure 4) and prevent currents getting behind the structure, called flanking. In the case of a deflection structure, these are usually placed in a series at critical points of scour and plants with engineered materials are placed in between them to help hold the bank. With the aid of these structures and time, the planted vegetation establishes roots and stems in the bank to hold it together and trap sediment. This sedimentation, in turn, leads to spread of the planted species and colonization by other opportunistic plants.

**Figure 3.** Timber cribs serving as deflection structures on the upper Missouri River to direct current away from the bank where there are bioengineering treatments.

**Figure 4.** Rock refusal used on an upper Missouri River bioengineering project. Note that it is keyed back into the bank to prevent flanking of the upper and lower end sections of the project.

## Bioengineering by Zones

Plants should be positioned in various elevational zones of the bank based on their ability to tolerate certain frequencies and durations of flooding and their attibutes of dissipating current- and wave-energies. Likewise, bioengineering fixes should be arranged by zone, which will be discussed below. The zone definitions given below correspond to those used by the U.S. Army Corps of Engineers, Omaha District, and have been used in preparing guidelines for the use of vegetation in streambank erosion control of the upper Missouri River (Logan et al. 1979). These zones are not precise and distinct since stream levels vary daily and seasonally--they are only relative and may be visualized as somewhat elastic depending on the bank geometry. If one carefully copied nature in the planning process, plant species can be chosen that will adapt to that specific zone or micro-habitat. Mallik and Rasid (1993) lend credence to this zonal concept in a study on the Neebing-McIntyre Floodway, parts of the Neebing and McIntyre River Complex near the Intercity area of Thunder Bay, Ontario, Canada. They describe four micro-habitats: bank slope, scarp face, above-water bench, and under-water depositional shelf. Each one had distinctively dominant plant species that generally correspond to the types of plants adapted for this report. Figure 5 illustrates the location of each bank zone for the upper Missouri River example. A description of each and the types of vegetation and appropriate species examples associated with them is given below. This zonal concept can be expanded to other streams to

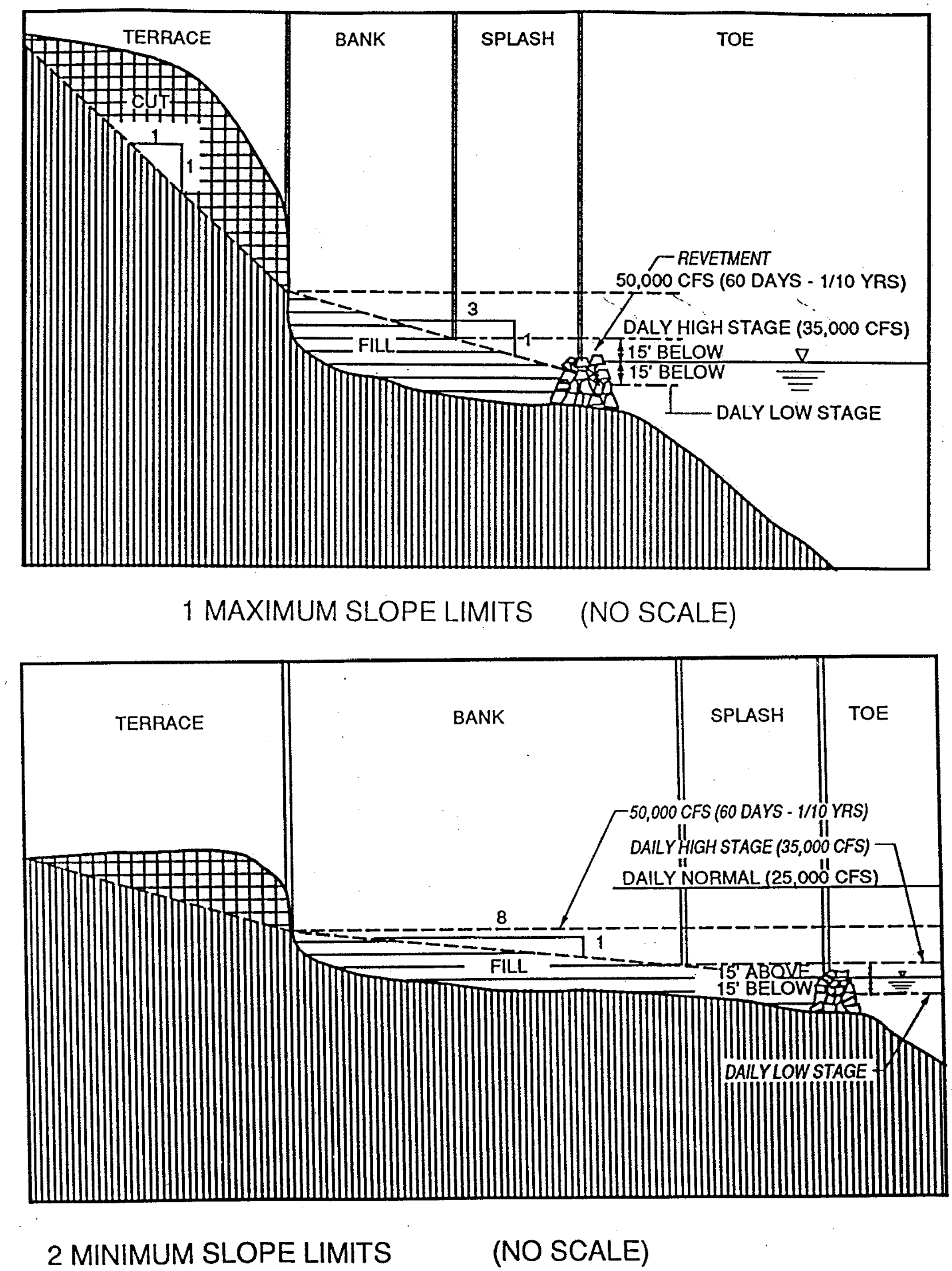


**Figure 5.** Bank zones defined on constructed slopes.

facilitate prescription of the erosion control treatment, and plants to use at relative locations on the streambank.

<u>*Toe zone.*</u> That portion of the bank between the bed and average normal stage. This zone is a zone of high stress and can often be undercut by currents. Undercutting here will likely result in bank failure unless preventative or corrective measures are taken. This zone is often flooded greater than 6 months of the year.

Figure 6 illustrates the plant species prescribed for each streambank zone on the upper Missouri River, except for the toe zone. The toe zone would likely have to be treated by some hard material, such as rock, stone, log revetments, cribs, or a durable material such as a geotextile roll (to be discussed later).

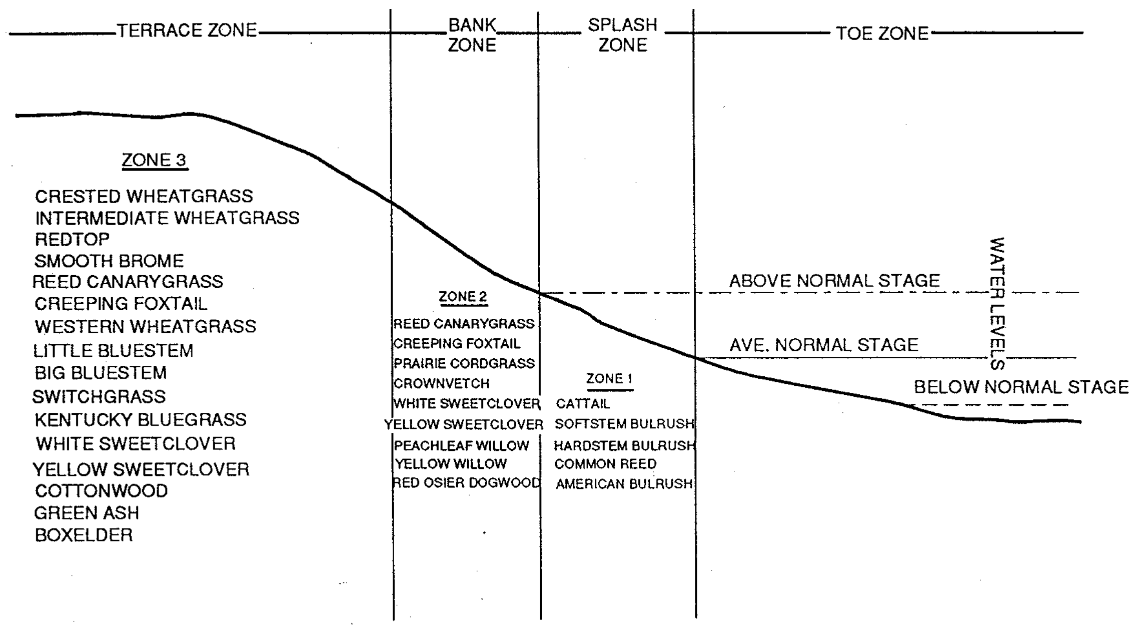


**Figure 6.** Possible species to plant by zone on the Missouri River.

<u>*Splash zone.*</u> That portion of the bank between normal high-water and normal low-water flow rates. This and the toe zone are the zones of highest stress. The splash zone is exposed frequently to wave-wash, erosive river currents, ice and debris movement, wet-dry cycles, and freezing-thawing cycles. This section of the bank would be inundated throughout most of the year (at least 6 months/year), but note that a large part of this inundation may occur in the dormant season of plants. The water depths will fluctuate daily, seasonally, and by location within the splash zone.

Only herbaceous emergent aquatic plants like reeds, rushes, and sedges are suggested for planting in the splash zone (Figure 6). These types of plants can tolerate considerable flooding and are more likely to live in this zone. They possess aerenchyma, cells with air spaces, in roots and stems that allow the diffusion of oxygen from the aerial portions of the plant into the roots (Mitsch and Gosselink, 1986). Therefore, they can extend roots into deeper water than many other types of plants, such as woody plants. Reeds, such as common reed (*Phragmites australis*), and sedges, such as bulrushes (*Scirpus* spp.), also protect streambanks in various ways. Their roots, rhizomes, and shoots bind the soil under the water, sometimes even above the water (Seibert 1968). In the reed zone, as Seibert (1968) defines it, they form a permeable underwater obstacle which slows down the current and waves by friction, thereby reducing their impact on the soil. Active protection of the bank can be ensured by reeds only in an area which is constantly submerged (Seibert 1968).

It should be mentioned that common reed is often considered a pest in the U.S. where it has been observed as a monotypic plant that does not offer habitat diversity. The authors would submit that this is true where there is not much of an elevation and hydrologic gradient. In other words, on shallow flats that become periodically inundated, it can thrive. However, when it is on a shoreline and becomes inundated over about 18 inches, it is often replaced by other more water tolerant species. One should use caution on where this plant is used and match it to one's objectives.

Various wetland grasses, sedges, and other herbs were used in this zone as a part of a coir geotextile roll in an urban park setting in Allentown, Pennsylvania. The main vegetative components of erosion control of the stream embankment are: lake sedge (*Carex lacustris),* stubble sedge (*C. stipata),* and woodland bulrush (*Scirpus sylvaticus*). Other minor components used for diversity and color included: rice cut-grass (*Leersia oryzoides*), other sedges (*C. lata, C. lanuginosa, C. hysterina, and C. prasina)*, softstem bulrush (*Scirpus validus),* blue flag iris (*Iris versicolor)*, and monkey flower (*Mimulus ringens*). The latter two species were provided primarily for additional diversity and color (Siegel, 1994a). Siegel reported that these plants, along with bioengineering methods such as the coir roll, stabilized a streambank that was subjected to storm events. In fact, the methods were designed to accentuate and enlarge the existing floodplain to act as a buffer zone for floods associated with storms greater than the 25-yr event (Siegel, 1994b). The vegetation list above only gives one examples of types of species that were used for erosion control in the splash zone, i.e., flood-tolerant and fast growing grasses and sedges. Care should be exercised in selecting species that are adapted to the project's geographic area. Local university botanists and USDA Natural Resources Conservation Service (NRCS, formerly Soil Conservation Service) district personnel can be consulted for suitable species.

Herbaceous emergent aquatic plants, like those shown in Figure 6, must be used on a streambank that has a geometric shape conducive to such plants. Caution must be used on streams that have heavy silt loads that could suffocate plants. These plants must grow in fairly shallow water, from +45 to -152 cm (Allen et al., 1989). Sometimes, it is impossible or impractical to find or shape a stream to match those conditions. Then, flood-tolerant woody plants, like willow (*Salix* spp.*)*, dogwood (*Cornus* spp.), and alder (*Alnus* spp.) are used in the splash zone. Again, a good rule of thumb is to look at the natural system and observe what is growing there and try to duplicate it.

Bank zone. That portion of the bank usually above the normal high-water level; yet, this site is exposed periodically to wave-wash, erosive river currents, ice and debris movement, and traffic by animals or man. The site is inundated for at least a 60-day duration once every two to three years. The water table in this zone frequently is close to the soil surface due to its closeness to the normal river level.

In the bank zone, both herbaceous (i.e., grasses, clovers, some sedges and other herbs) and woody plants are used. These should still be flood tolerant and able to withstand partial to complete submergence for up to several weeks. Allen and Klimas (1986) list several grass and woody species that can tolerate from 4 to 8 weeks of complete inundation. This list, should not be considered exhaustive, however. Whitlow and Harris (1979) provide a listing of very flood-tolerant woody species and a few herbaceous species by geographic area within the United States that can be used in the bank zone.

Skeesick and Sheehan (1992) report on several other herbaceous and woody plants that can withstand tens of feet of inundation over 3 to 4 months in two different reservoir situations in Oregon. These same species are often found along streambanks. Local university botanists and plant material specialists within the NRCS should be consulted when seeking flood-tolerant plants . Various willows can be used in this zone, but they should be shrublike willows such as sandbar willow (*S. exigua*) and basket willow (*S. purpurea* var. *nana*). Edminster et al. (1949) and Edminster (1949) describe successful use of basket willow for streams and rivers in the Northeast. Shrub-like willow, alder, and dogwood species have been used in Europe successfully (Seibert 1968). Red-osier dogwood (*Cornus stolonifera*) and silky dogwood (*C. amomum*) also have been used in the Northeast (Edminster et al. 1949 and Edminster 1949). Seibert (1968) notes that in periods of high water, the upper branches of such shrubs reduce the speed of the current and thereby the erosive force of the water. The branches of these have great resilience, springing back after currents subside.

Terrace zone. That portion of the bank inland from the bank zone; it is usually not subjected to erosive action of the river except during occasional flooding. This zone may include only the level area near the crest of the unaltered “high bank” or may include sharply sloping banks on high hills bordering the stream.

The terrace zone is less significant for bank protection because it is less often flooded, but can be easily eroded when it is flooded if vegetation is not present. Vegetation in this zone is extremely important for intercepting floodwaters from overbank flooding, serving to reduce super-saturation and decrease weight of unstable banks through evapotranspiration processes and for tying the upper portion of the streambank together with its soil-binding root network. Coppin and Richards (1990) provide a detailed explanation of plant evapotranspiration, but summarize by saying, “ Apart from increasing the strength of soil by reducing its moisture content, evapotranspiration by plants reduces the weight of the soil mass. This weight reduction can be important on vegetated slopes where the soil may be potentially unstable.”

As denoted in Figure 6, the terrace zone can contain native grasses, herbs, shrubs, and trees that are less flood tolerant than those in the bank zone, but still somewhat flood tolerant.

The tree species also become taller and more massive. Trees are noted for their value in stabilizing banks of streams and rivers (Seibert 1968; Leopold and Wolman, 1957; Wolman and Leopold, 1957; Lindsey et al. 1961; and Sigafoos 1964). The trees have much deeper roots than grasses and shrubs and can hold the upper bank together. The banks of some rivers are not eroded for durations of 100 to 200 years because heavy tree roots bind the alluvium of floodplains (Leopold et al. 1957; Wolman and Leopold 1957; and Sigafoos 1964). A combination of trees, shrubs, and grasses in this zone will not only serve as an integrated plant community for erosion control, but will improve wildlife habitat diversity and aesthetic appeal.

## Bioengineering Treatments

The entire streambank should be treated to furnish a maximum array of plants capable of providing proper ground cover and root penetration for erosion protection, wildlife habitat, water quality improvement, and many other benefits. At times, the planting sites or zones may be quite narrow in width or difficult to distinguish depending on the geomorphology of the stream. The entire bank in these cases should be treated as a systematic arrangement of plants and treatment practices.

### Toe Zone

This is the zone that will need to be protected from undercutting with treatments such as stone or rock revetments, gabions, lunkers, log revetments, deflector dikes, cribs, rock and geotextile rolls, root wads, or a combination of materials. The zone rarely has vegetation employed in it alone, but when vegetation is employed, it is used in combination with materials that extend below the normal flow of water and above it. The principle is to keep high velocity currents from undercutting the bank either through armoring the bank or deflecting the currents away from the site of concern. Vegetation can then be used either above the armor or in between and above the deflecting structure.

Stone or rock revetments in a bioengineering application are used at the toe in the zone below normal water levels and up to where normal water levels occur. Sometimes, the stone is extended above where normal flow levels occur depending on the frequency and duration of flooding above this level. Then, vegetation is placed above it in a bioengineering application. Stream guage information helps in making this judgement. Unfortunately, there are no set guidelines for how far up the bank to carry the revetment except to say that it should be applied below the scour zone up to at least the level where water runs the majority of the year. Engineering Manual 1110-2-1601, Change 1 (Corps of Engineers, 1994) gives guidelines for riprap toe protection.

One such rock revetment for toe protection was used in conjunction with vegetation above it on Crutcho Creek, Tinker Air Force Base, Oklahoma (Figure 7). In this example, the creek is flashy and often reaches or exceeds the top of bank during the spring of each year for a few days. The rock toe extended from the bed to about 1/3 the height of the bank (Figure 8). This treatment has been successful in this type of setting after several floods exceeding the top of the bank.

Rock toes are also used streamward or just below other materials such as hay bales or geotextile rolls. In one example, Omaha District recently used rock riprap below a large hay bale cylinder covered with a fabric (rope mesh) made from woven fibers of coconut husks called coir. The riprap weighed about 1.5 Tons/ft and was about 3.5-ft deep. Then, vegetation in the form of dormant willow poles (discussed below) was placed above this (Figures 9 and 10).

In another example, a rock roll (Figure 11) was used on the Rhine River in Dusseldorf, Germany, below an installation of wetland vegetation grown in geotextile mats made from coir. The large rock was wrapped in a polyethylene type of rope mesh and installed in the following fashion: 1) a ditch is dug; 2) the rope mesh is placed in the ditch so that enough of it is overhanging the ditch on the riverward side to wrap around the rock and have some left on the shoreward side on which to place more rock; 3) the rock is placed on the rope mesh; 4) the rope mesh is wrapped around the rock with a portion of it running up the shoreward side; and finally 5) more rock is backfilled on top of the rope mesh to hold it all firmly in place. This rock roll serves to protect the treatment from undercutting. The rope mesh contains smaller rocks and strengthens the system and is similar to the function of gabions which are discussed below. It should be mentioned that this whole system of rock rolls and geotextile mats with wetland grasses or grass-like plants, such as sedges, were placed in between large rock transverse dikes (Figure 12). The dikes were already there before this treatment was installed and divert river currents away from the banks. The rock roll (toe protection), the transverse dikes, and the geotextile coir mats, work together to obtain wetland plant establishment and erosion control. Prior to the installation of plants, even though the transverse dikes were present, an asphalt revetment used to control erosion failed because water got behind the asphalt and pushed it out. This system has been in place from 1991 to present and has withstood a large flood in 1994, the largest in the last decade, with more than a 7 m fluctuation above normal flow. The flood overtopped the treatment for several months. Because of the wetland plants' flood tolerance, the rock toe, and transverse dikes, they survived and are still controlling erosion. A key wetland plant species instrumental in the treatment's success was a sedge, *Carex hirta*[2].

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[1] Herr Lothar Bestmann, personal communication, President, Ingenieurbiologie, Wedel,Germany, May 9, 1996.

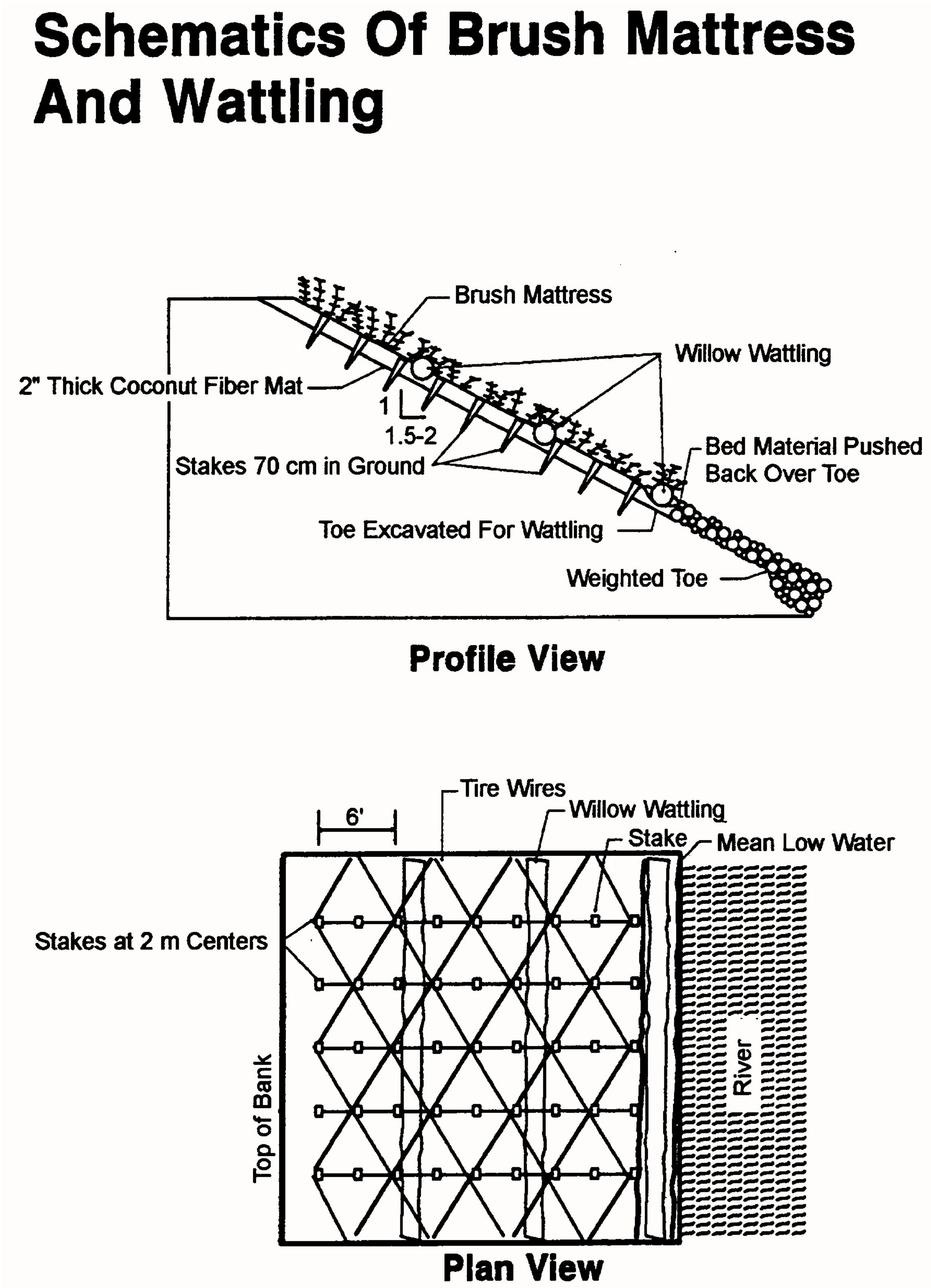



**Figure 7.** Schematics of bioengineering treatment used with a weighted rock toe with vegetation in the form of a brush mattress (to be discussed later) used above it.

**Figure 8.** Photo of weighted rock toe revetment extending up the bank. It extends about 1/3 the distance up the bank. This photo shows the stream above low-flow conditions.

**Figure 9.** Photo of bioengineering project on upper Missouri River where large rock (1.5 Tons/lin ft) was used as toe protection below large coir-covered haybales, also forming part of the toe.

**Figure 10.** Vegetation in the form of dormant willow posts (discussed later) was placed landward of the rock and haybale toe.

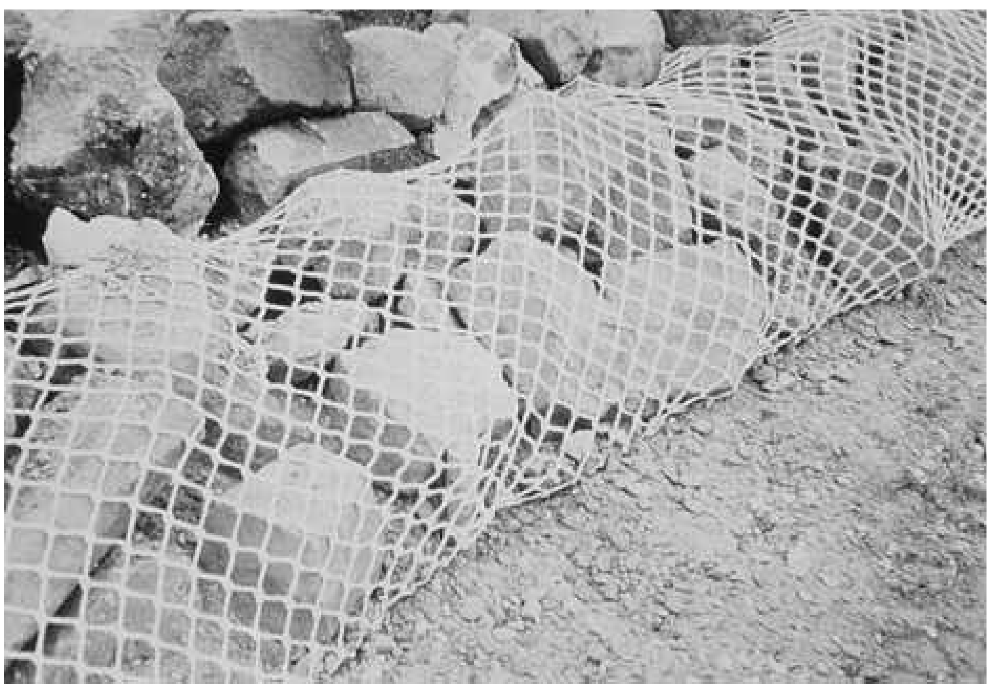


**Figure 11.** Rock roll used as toe protection on a bioengineering project, Rhine River, Germany, in the city of Dusseldorf.

**Figure 12.** System of bioengineering treatments such as geotextile coir mats with planted vegetation on them placed above a rock roll toe and between large rock transverse dikes.

Gabions are wire mesh baskets filled with rock and formed as boxes of various dimensions. The wire is either galvanized or covered with a plastic coating to increase durability. Gabions are tied together to become large, flexible, structural units and can be stacked in tiers. They can be installed in the toe zone to prevent undercutting and can be stacked or run as a revetment of gabion mattresses up into the splash and bank zones (Figure 13). They can be used in conjunction with vegetation in several ways. Often times, live, willow whips are placed in between the tiers of boxes back into soil that facilitates sprouting. When they are used as a gabion revetment and rock toe, vegetation can be placed in the splash and bank zones above them. Gabions should be used with caution in streams that have high bed load movement with cobbles and gravels that may break the wire mesh. Also, they are susceptible to vandalism and to undercutting/overturning. If used in a stacked fashion, a geotechnical engineer should be consulted to determine stability; otherwise, overturning and sliding may be a problem.

Figure 14 is two schematics (two different versions) of a hard stabilizing structure for a toe. This structure is called a LUNKERS, which is an acronym for "Little Underwater Neighborhood Keepers Encompassing Rheotactic Salmonids." The LUNKERS is designed to provide overhanging shade and protection for fish while serving to stabilize the toe of a streambank. They were first used by the Wisconsin Department of Natural Resources and described in detail by Vetrano (1988). They have since been adapted for use by the Illinois State Water Survey. They are made from treated lumber, untreated oak, or materials made

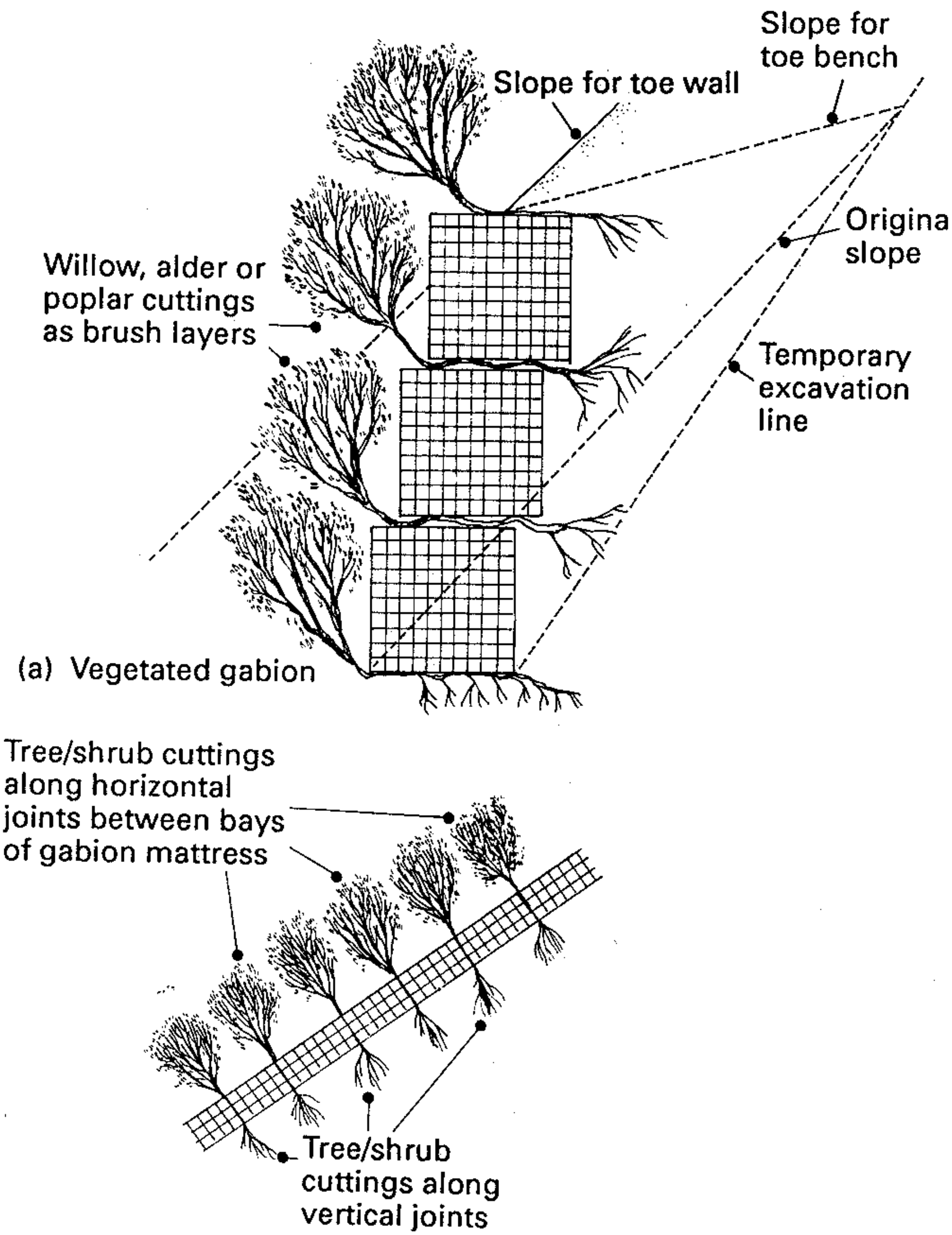
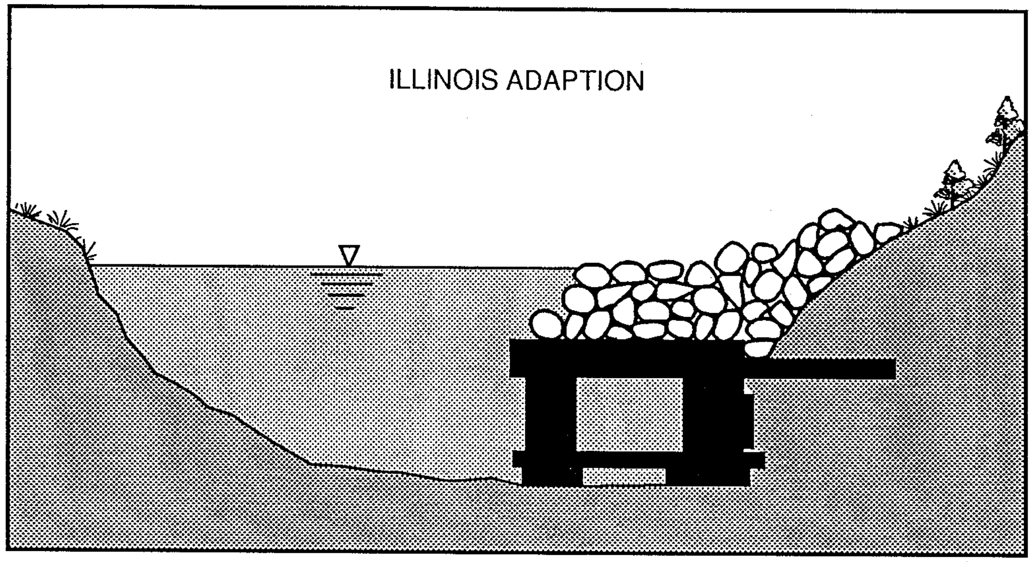



**Figure 13.** Schematic of gabions used with woody plants to form a hard structure to prevent undercutting and flanking. Can be used in the toe zone or installed higher in the splash and bank zones. (from Coppin and Richards, 1990)



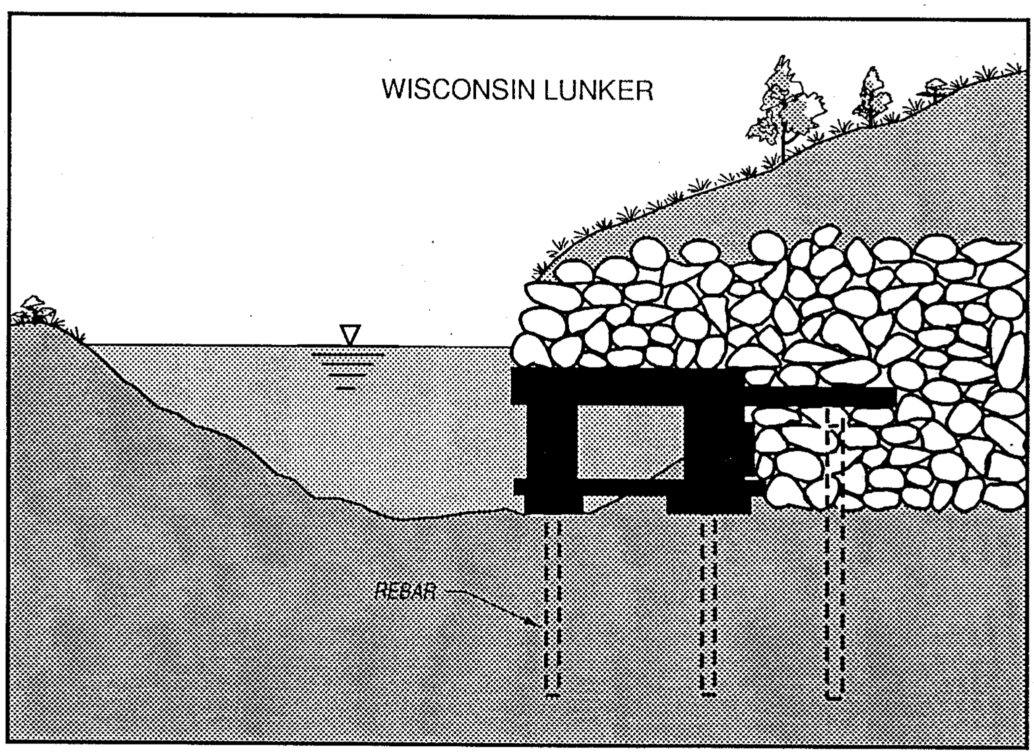




**Figure 14.** Two schematics (two different versions) of a LUNKERS structure designed to provide overhanging shade and protection for fish while serving to stabilize the toe of a streambank. Both versions use rebar although rebar is not shown on the upper schematic.

from a combination of plastic and wood. They are constructed by nailing planks to the top and bottom of 15- to 20-cm spacer logs. These planks form stringers, which tie into the streambank at right angles. Planks are nailed to the top and bottom stringer boards and run parallel to the streambank. The entire structure forms a crib, which can be constructed onshore and moved by a loader or backhoe to the installation site. Once in the stream, the LUNKERS is placed in position and anchored by driving 1.5-m lengths of steel-reinforcing rod through predrilled holes in the structures and then into the streambed. These structures are set in a line that simulates the outside bend of a meander. After the structures are in place, the area behind them is filled with rock , which also is used to cover the structure, and then the entire area is covered with soil (Hunter, 1991). Often, the soil is planted with various kinds of vegetation, either woody or herbaceous. Care must be taken to tie the ends into the bank with a transition of rock or into a hardpoint to prevent flanking.

Another hard structure placed in the toe zone to stabilize the toe is a "Bank Crib with Cover Log" (Figure 15). This is described by the USDA Forest Service (1985). Like the LUNKERS, it is used to protect unstable streambanks at the toe while at the same time providing excellent overhead cover for fish. The design is a simple crib with abutment logs extending as far back into the bank as necessary to assure structural stability (1.3 to 1.8 m in stable soils and 3 m or more in unstable soils). The lower abutment logs should be near water level and should extend 45 to 60 cm from the bank. The cover log can then be pinned to the crib log and the lower abutment. The structure can be from one to several logs high, depending upon bank height. The only materials required are logs on site and 1.6 cm rebar to join the logs. Installing structures is fairly time consuming, due to the amount of digging required. One crew should be able to install 6 to 9 m of crib (two crib logs high) per day if logs are reasonably close to the site. Water adjacent to some eroding banks requiring abutment work is sometimes too shallow to make effective use of cover logs. It has been noted by some that rocks need to be added below the crib log and upstream and downstream from the structure to avoid scour and flanking respectively.

Log revetments are similar to bank cribs with cover logs except these are used to harden the toe and continue up the bank by lining the bank with logs (Figure 16). Then, flood-tolerant plants are placed at the top of and shoreward to the revetment. Depending on the height of the revetment, this may be in the splash, bank, or terrace zones. They are placed with butt ends facing upstream and are overlapped in a shingle fashion. They are secured with cables that are looped around the logs and then are fastened to dead men in the bank. Care must be taken to ensure their longevity by placing rock on both the upstream and downstream ends to prevent flanking of the structure. Rock should also be placed at the toe of the structure to prevent scour.

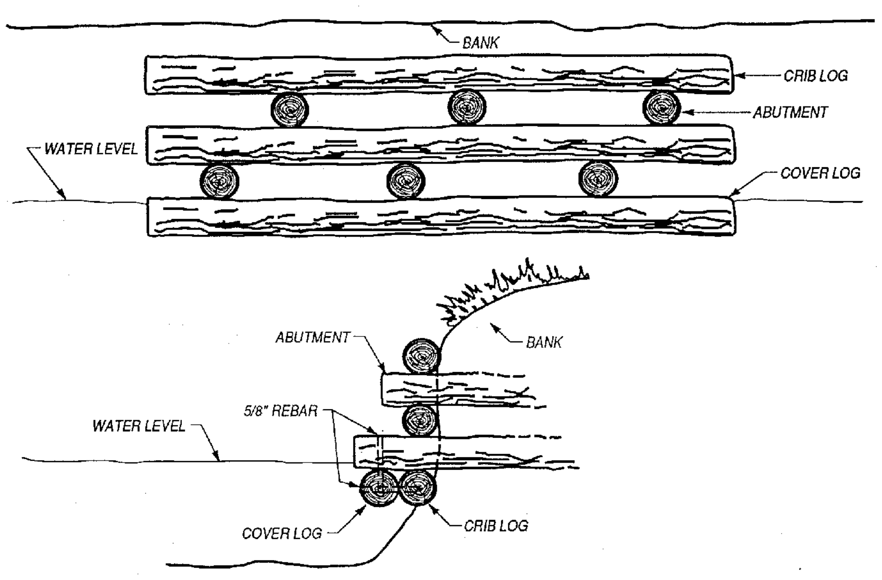



**Figure 15.** Bank crib with cover log used to protect unstable streambanks while concurrently providing excellent overhead cover for fish. (Courtesy of US Forest Service)

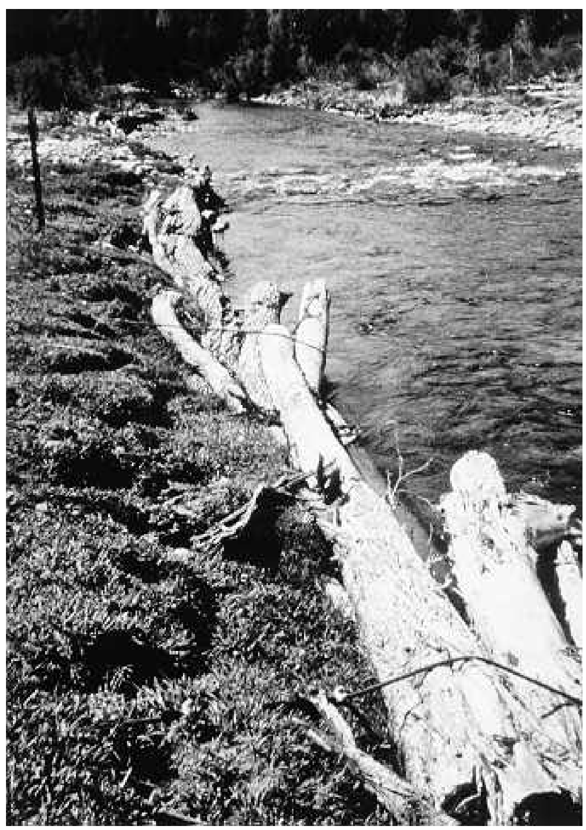


**Figure 16.** Log revetment, Roaring Fork River, Colorado. Note the cable wrapped around the logs and buried and secured to dead men in the bank.

Figure 17 shows a schematic of a log revetment used on the Roaring Fork River, Colorado, near Basalt. A geotextile coir roll, called a Vegetations-Faschinen in Germany, where it originated, is placed above the top log in the revetment so its top is just even with or slightly above the normal water level. The roll is often referred to in this country under various trade names such as Fiber Roll, Fiberschine, and Bio-log. It is used in conjunction with a geotextile mat which is placed shoreward of the roll, backfilled with soil, and planted or seeded with wetland plants. The geotextile roll and mat trap sediment, allow plants to be planted in them, and are biodegradable. Note that the top log is placed in an overhanging fashion with the coir roll on top to provide shade and cover for fish. Figure 18 shows an installed log revetment on the Roaring Fork River. Volume II presents a case study that includes evaluation of such a treatment, among others on western Colorado rivers and streams and notes local velocities to which this treatment and others were subjected. On one reach of the Roaring Fork, this structure failed because it was not keyed into the bed of the stream. Scour at the toe caused structure failure. On another reach, it worked just fine. These structures must be properly protected at the toe and at the upper and lower ends with rock and hard points, respectively.

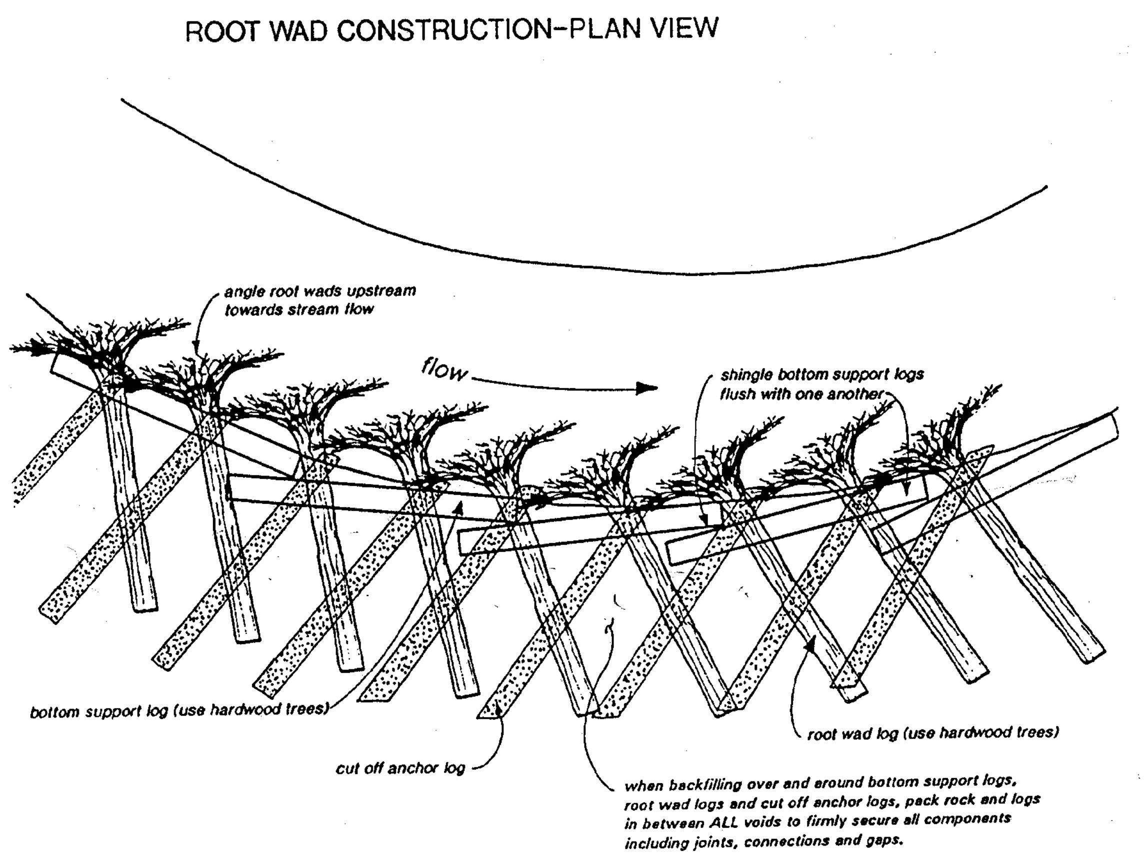



**Figure 17.** Schematic of root wad construction (from Bowers, 1992).

**Figure 18.** Installed log revetment with coir geotextile roll combination, Roaring Fork River, Colorado. Wetland vegetation is seeded or planted in backfilled soil placed in a depression between the revetment and the land. Rock is placed on top to prevent scour.

Root wads are live or dead logs with root masses attached (Figure 19, see Bowers, Land and Water, 1992). These are also used in the toe zone to protect it from undercutting, but must be used in combination with other materials. The fans of the root wads provide an interlocking wall protecting the streambank from erosion. The voids within and between the root wads are filled with a soil mix and planted with live, willow clumps or root pads. The root wads are laid on top of a keyed-in shelf of stone and support logs. This shelf includes a layer of bottom support logs flush with one another, shingled together, and running parallel to the streambank. The root mass should be a minimum of 5-ft in diameter and angled slightly upstream towards stream flow. This treatment should be placed at a base elevation that is consistent with water levels during the major part of the growing season, i.e., June through September. The bottom two-thirds of the root wad should be in water during that period of time. The upstream and downstream ends of the root wad treatment should be tied into hard points made from rock or some natural hard feature so as to prevent flanking.

Figure 20 shows a treatment using root wads on the Upper Truckee River in California near South Lake Tahoe, where this treatment and others were monitored for a couple of growing seasons (see also Volume II). Various local flow velocities were measured along the treatment on the fall of the hydrograph. These ranged from 1.6 to 4.0 fps at .6 depth of flow and 4 ft out from the right bank. The root wads sufficiently reduced local flow velocities so that vegetation had a chance to get established and stabilize the bank despite a major flood in the spring and summer of 1995 where floodwaters overtopped the bank. Rosgen[3] noted that on a root wad treatment on the Blanco River in Colorado, that local

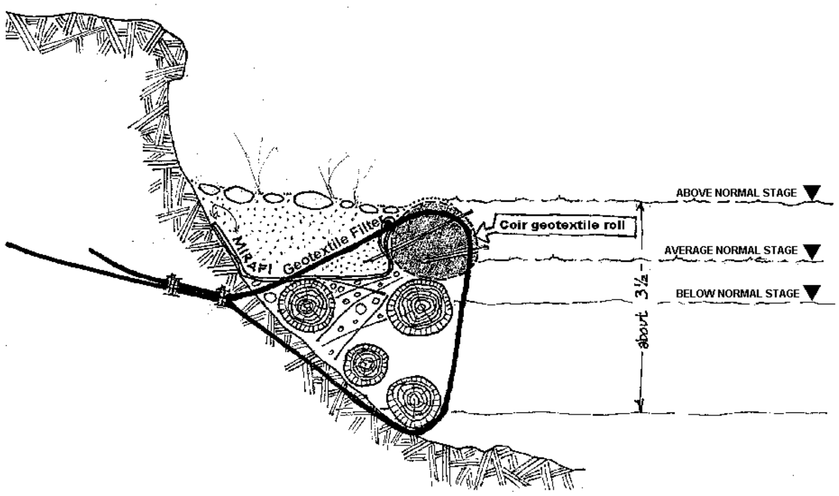



**Figure 19.** Schematic of log revetment with coir geotextile roll and plantings on top of backfill soil over a geotextile filter. (Designed by Alan Czenkusch, Colo. Division of Wildlife)

**Figure 20.** Root wads soon after installation on the Upper Truckee River, California, near South Lake Tahoe. The voids within and between the root wads are filled with a soil mix and planted with live, vegetative clumps or root pads, such as willow.

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[3] Rosgen, Dave. Pres., Wildland Hydrology, Pagosa Springs, CO, Jul 1996, pers. comm.

velocities in the vicinity of the root wads were 12 fps and yet willow clumps installed in with the root wads and the root wads themselves did not fail.

Deflector dikes are any constructed protrusion into the water that deflect the current away from the eroded bank. These consist of: transverse dikes, hardpoints, groins, bendway weirs, and stream barbs. They are usually made of rock, but other materials such as logs or trees can be used. As mentioned above in the Dusseldorf, Germany, example, bioengineered treatments often use vegetation between deflector dikes. The dikes and the bioengineered treatments work as a system to stabilize the streambank. Transverse dikes differ from hardpoints or groins by projecting further out into the stream. Bendway weirs and stream barbs are low rock sills. Flows passing over them is redirected so that the flow leaving the structure is perpendicular to the centerline of the structure. Derrick (1996) describes the construction and use of bendway weirs both on the Mississippi River and on smaller streams in northern Mississippi. In the latter case, bendway weirs were successfully used, in part, with a dormant willow post method of stabilizing the streambank (to be discussed below). Shields et al. (1995) describe the benefits to aquatic habitats on small streams in northern Mississippi by use of such weirs. The structures increased pool habitat availability, overall physical heterogeneity, riparian vegetation, shade and woody debris density. To design deflector dikes with vegetation, persons are needed with training both in hydraulic engineering and bioengineering working as a team. Hydraulic engineers should be consulted for design, construction, and placement of the deflector dike and bioengineers or someone with training in botany should be consulted for use and placement of the vegetation.

A combination of materials, as mentioned above, can be used in the toe zone. Deflector dikes can be used with plants incorporated in the dike system for erosion control as well as fisheries habitat. Figure 21 shows a schematic of a coir geotextile roll. As illustrated in the figure, it is used in combination with rock at the base and around the ends with some openings for the ingress and egress of fish and other aquatic organisms. The coir is stuffed into a rope mesh material made either out of coir itself or of polyethylene. The roll is planted with emergent aquatic plants. The coir accumulates sediment and biodegrades as plant roots develop and become a stabilizing system. Figure 22 shows several on a German stream. Each structure serves to redirect the current away from the bank so that vegetation can be installed in between. The plants in the structure furnish shade and cover for aquatic life. While the rock of the structure would be in the toe zone, the roll and the aquatic plants would be on top of the rock and abreast of it. The roll would actually grade into the next higher zone, the "Splash Zone."

*Splash Zone*

The coir roll mentioned above can also run parallel to the bank with rock in the toe zone providing the foundation and additional protection at the base of the roll itself. Sometimes, the coir roll is all that is used in the toe zone when currents or waves are not strong or big

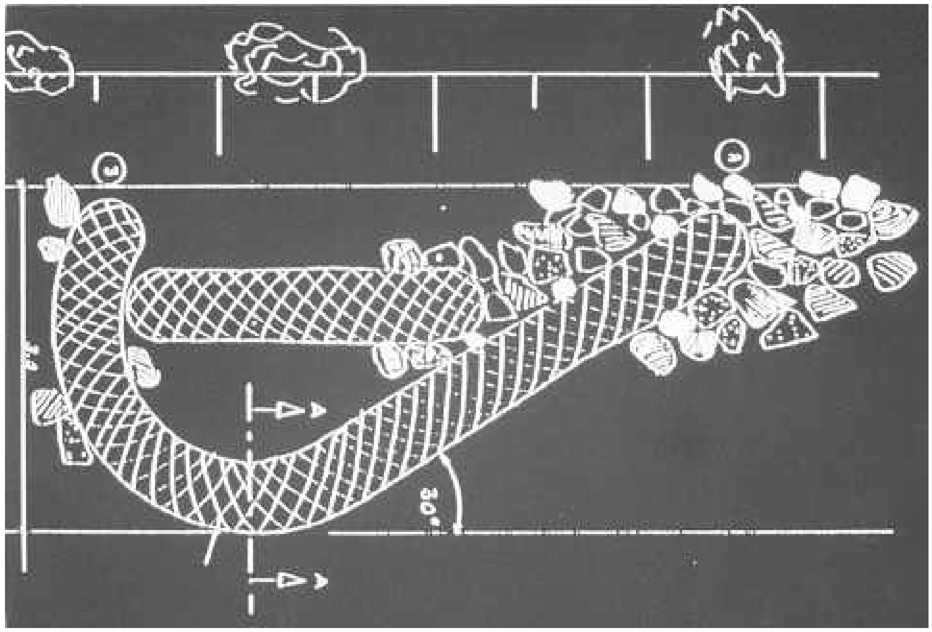


**Figure 21.** Schematic of a coir geotextile roll and rocks. The roll is planted with wetland vegetation. Used as a deflector system while serving as aquatic habitat. (Photo courtesy of Bestmann Ingenieurbiologie, Wedel, Germany)

**Figure 22.** Photo of coir geotextile roll and rocks with wetland plants serving as a deflection system and providing aquatic habitat on a German stream. Note that two can be seen on the opposite bank also. (Photo courtesy of Bestmann Ingenieurbiologie, Wedel, Germany)

enough to justify rock. Then, vegetation is planted or grown in the roll to form part of the splash zone. Figure 23 is a schematic of a coir roll abutted to an unshaped bank with some backfill. Figures 24 a-d show such a treatment in a stream in Germany and planted with emergent aquatic vegetation, such as bulrushes, iris, and sedges. Vegetation can be grown in the roll at a nursery and then transferred to the planting site with vegetation almost established.

Coir rolls and emergent aquatic vegetation have also been used in this country recently. One such use was on the North River near Colrain, Massachusetts. It was monitored as a part of this work unit for two growing seasons. That case study is presented in Volume II. Both single and double coir rolls were used in different sections of the streambank. In the latter case, another roll was placed upslope from the first one. Both were planted by inserting clumps of emergent aquatic plants in them. Where overhanging banks occurred and were void of woody vegetation, an evenly sloped bank was achieved by shaping and backfilling using a small front-end loader. Shaping, however, was minimized where possible in an effort to prevent disturbance of the bank and existant vegetation. It should reiterated that the coir rolls should be keyed well into the upper and lower ends of the reach being treated. The authors discovered after the two-year formal monitoring period, that the coir rolls had apparently been flanked at the upper end as a result of flooding in the fall of 1995 and that sections of the project unraveled.

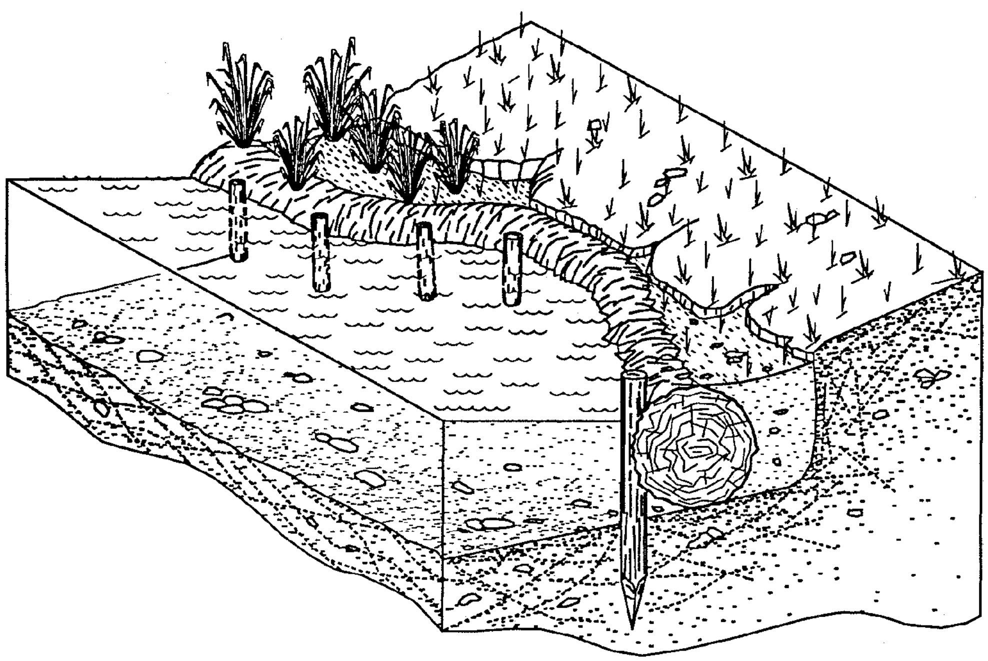

**Figure 23.** Coir geotextile rolls are used to stabilize streambanks and permit planting of wetland vegetation within them. The coconut fiber accumulates sediment and biodegrades as plant roots develop and become a stabilizing system. (From Bestmann Ingenieurbiologie, Wedel, Germany)

**Figure 24a.** Coir geotextile roll being installed along a streambank in Germany.

**Figure 24b.** Coir roll a month or so after planting.

**Figure 24c.** Coir roll a few months later.

**Figure 24d.** Closer view of coir roll a few months after plant establishment.

**Figure 24.** Wetland plant development in a coir geotextile roll within the splash zone at a stream in Germany. (Photos of Figures 24 a-d courtesy of Bestmann Ingenieurbiologie, Wedel, Germany)

The clumps of emergent aquatic plants mentioned above that were placed in the coir rolls were grown from seedlings placed in a coir wrapping and allowed to develop hydroponically (in water without soil, but with nutrients added). This leads to a well-developed, but light and easily transportable plant unit with roots readily established and poised to grow in a planting medium, such as the coir roll or in a soil substrate.

Coir fiber mats made in various thicknesses are also used in the splash zone. These are often prevegetated at the nursery with emergent aquatic plants (Figure 25 a-c) or sometimes sprigged (use of single or multiple rooted stems inserted into substrate) with emergent aquatic plants harvested from local sources. When prevegetated at the nursery, the fiber mats have the advantage of being light and can be lifted in rolls or smaller mats and transferred directly to the planting site where immediate establishment is required. They are usually tied into or keyed into whatever is used as the toe material. In the example on the North River above, 1-inch thick mats were prevegetated and tied into the coir rolls. Coir fiber mats have the attributes of high tensile strengths, the ability to trap sediment, they are pH neutral, they facilitate root development because of the fiber network, and they are slow to biodegrade. These types of vegetated coir mats have also been used on dredged material in coastal environments with wave environments. Knutson et al. (1990) reported successful trials of sprigging emergent aquatic plants into such mats. This success was attributed, in part, to the attributes mentioned above, such as sediment entrapment. The blankets trapped sediment very well on the North River which aided plant establishment.

Single-stemmed sprigs and clumps of emergent aquatic plants and flood-tolerant grasses or grass-like plants, e.g., rushes, sedges, can be planted shoreward of hard rock toes, coir rolls, and fiber mats. They can even be used in lieu of the fiber mats if the site-specific conditions are appropriate. This may mean that the soils are more cohesive, i.e., have more clay in them, the stream discharges at that level are not as high.

Our focus in the splash zone, so far, has been on use of emergent aquatic and other herbaceous plants. Woody plants are also used in the splash zone. For these, wetland plants are used that can also withstand periods of dryness. The woody plants should be those that can sprout roots and branches from the stem. These include willow, some species of alder, dogwood, and several other species. Several possible species are listed by the Georgia Soil and Water Conservation Commission (1994) and Gray and Sotir (1996). Sometimes, woody plants may be all that are suited to the splash zone. At times, the bank geometry is very steep down to the normal flow level without a shallow water zone for emergent aquatics or, the stream system has extreme fluctuations and large silt loads that would drop sediment on emergent aquatics and bury them.

Bioengineering techniques that utilize woody plants include: brushmattress, brush layering, vegetative geogrids, dormant post method, dormant cuttings, and dormant root pads. All of these are usually used in combination with hard structures or materials that either deflect the current away from the bank or protect the toe and upper and lower ends.

**Figure 25a.** Emergent aquatic plants in WES greenhouse nursery that were seeded on coir fiber mat.

**Figure 25b**. Emergent aquatic plants established on a coir fiber mat being rolled up in the WES nursery ready for transport to the bioengineering site.

**Figure 25c**. Coir geotextile mat in a roll planted with emergent aquatic plants being carried to the planting site.

**Figure 25.** Coir geotextile mat being prevegetated in the nursery and transported to the field site ready for immediate growth. Roots and stems of the plant have already been established in the mat.

For instance, dormant root pads are used with root wads that were discussed above for the toe zone.

*Brushmattress*. A brushmattress, sometimes called brush matting or a brush barrier, is a combination of a thick layer (mattress) of interlaced live willow switches or branches and wattling. Both are held in place by wire and stakes. The branches in the mattress are usually about 2 to 3 years old, sometimes older, and 1.5 to 3 m long. Basal ends are usually not more than about 3.5 cm in diameter. They are placed perpendicular to the bank with their basal ends inserted into a trench at the bottom of the slope in the splash zone, just above any toe protection, such as a rock toe. The branches are cut from live willow plants and kept moist until planting. The willow branches will sprout after planting, but care should be taken to obtain and plant them in the dormant period, either in the late fall after bud set or in the early spring before bud break. A compacted layer of branches 10 to 15 cm thick is used and is held in place by either woven wire or tie-wire. Wedge-shaped construction stakes (2 X 4 X 24 " to 2 X 4 X 36", diagonal cut) are used to hold the wire in place. A guage and type suitable for tie-wire is No. 9 or 10 galvanized annealed. It is run perpendicular to the branches and also diagonally from stake to stake and usually tied by use of a clove-hitch. If woven wire is used, it should be a strong welded wire (2- by 4-in mesh). The wedged-shape stakes are driven firmly through the wire as it is stretched over the mattress to hold it in place. The wedge of the stake actually compresses the wire to hold the brush down. Wattling is a cigar-shaped bundle of live, shrubby material made from species that root very quickly from the stem, such as willow and some species of dogwood and alder. These bundles are laid over the basal ends of the brushmattress material that was placed in the ditch and staked. The procedure of making wattling bundles and installing them over the brushmattress material is presented in more detail below (These procedures are modified after Leiser (1994).

Wattling bundles may vary in length, depending on materials available. Bundles taper at the ends and this is achieved by alternately (randomly) placing each stem so that about one-half of the basal ends are at each end of the bundle. When compressed firmly and tied, each bundle is about 15 to 20-cm in diameter in the middle. Bundles should be tied with either hemp binder twine or can be fastened and compressed by wrapping "pigtails" around the bundle. Pigtails are commonly used to fasten rebar together. If tied with binder twine, a minimum of two wraps should be used in combination with a non-slipping knot, such as a square knot. Tying of bundles should be done on about 38-cm centers. Wattling bundles should be staked firmly in place with vertical stakes on the downhill side of the wattling not more than 90 cm on center and with the wedge of the stake pointing upslope. Also, stakes should be installed through the bundles at about the same distance, but slightly off-set and turned around so their wedge points downslope. In this way, the wedged stakes, in tandem, compress the wattling very firmly. Where bundles overlap, an additional pair of stakes should be used at the midpoint of the overlap. The overlap should be staked with one pair of stakes through the ends of both bundles while on the inside of the end tie of each bundle. Figures 26 a-b show a schematic of a brushmattress and wattling. Figures 27 a-c show a sequence of installing a brushmattress with wattling at a workshop. It should be noted that because of the workshop setting at a mild time of the year, non-dormant vegetative material is being used. Normally, one would preferably use dormant material.

Both brushmattress and wattling should be covered immediately with soil and tamped. Soil should be worked into both the brushmattress and wattling by by both tamping and walking on it. All but the edges of the brushmattress should be covered with soil and about 75 percent of the wattling should be covered leaving some of each exposed to facilitate sprouting of stems rather than roots.

A brushmattress without any rock toe was used on the North River, Massachusetts, and performed quite well for two growing seasons until unraveling started to occur, again because of a lack of toe and upper and lower end protection. This was in a reach where a bankfull discharge was experienced with an associated average bankfull velocity

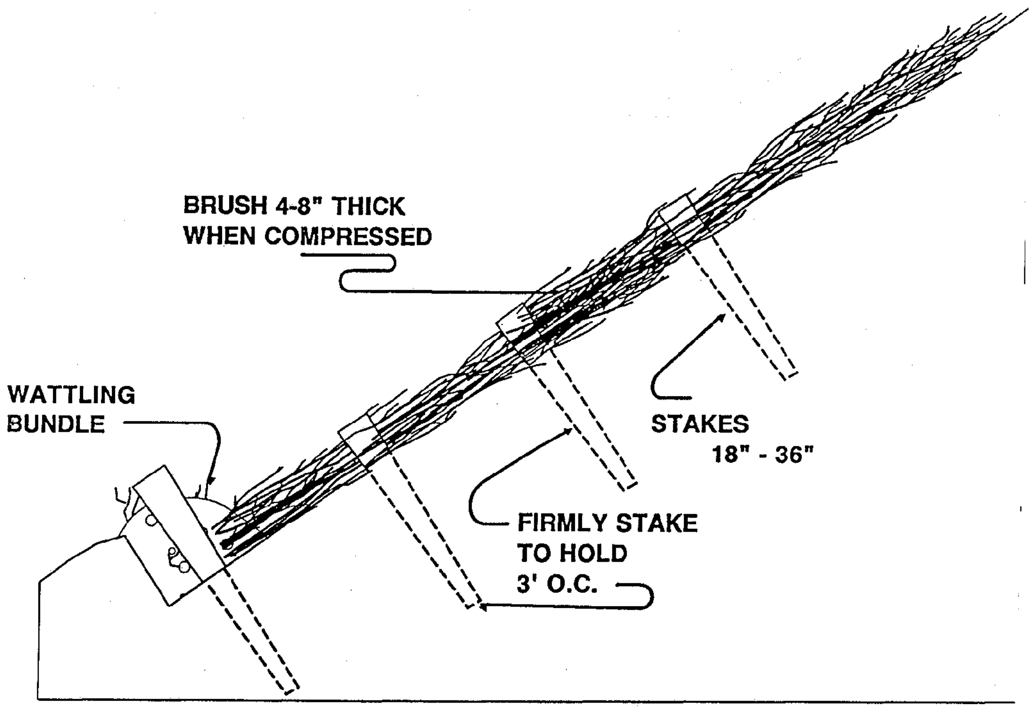


**Figure 26a**. Profile view.

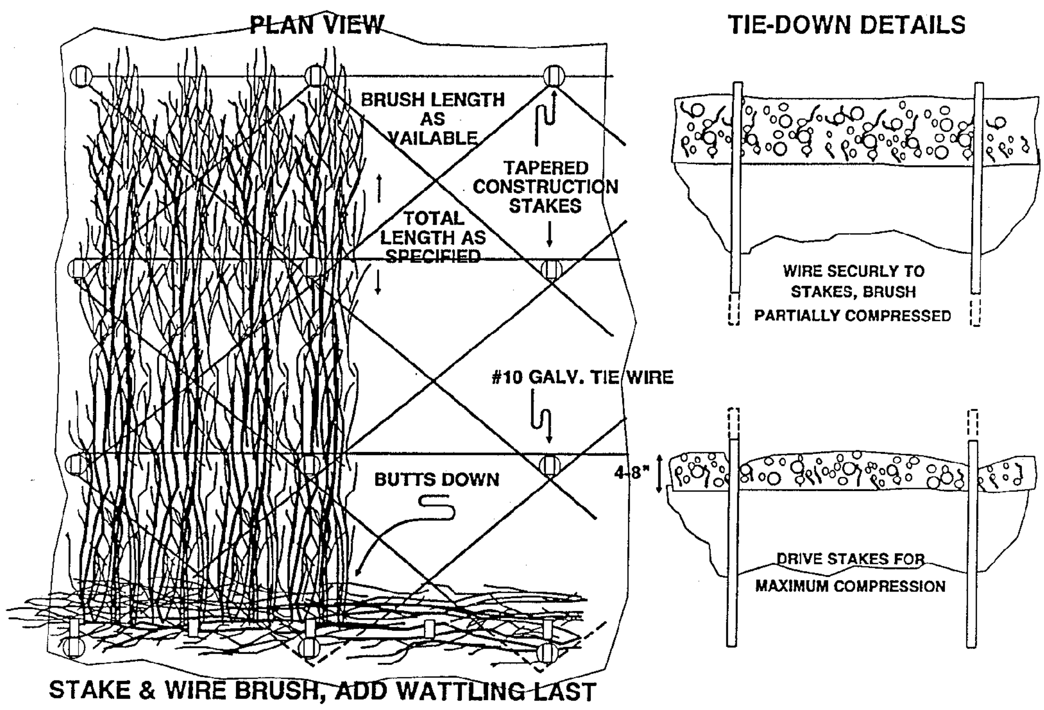


**Figure 26b.** Plan view.

**Figure 26.** Schematics of brushmattress and wattling combination. (from Leiser, 1983)

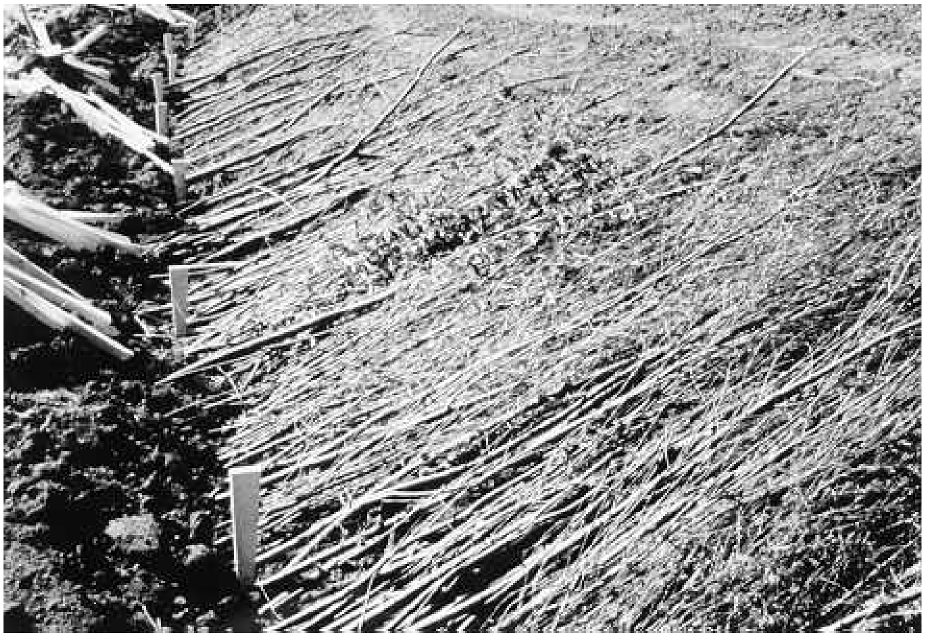

**Figure 27a**. Laying down the brush (basal end first) into a previously dug trench marked by row of wedge-shaped stakes.

**Figure 27b.** Placing woven wire over the willow brush.

**Figure 27c**. Stretching the woven wire tight and securing by wedge-shaped stakes. Also, the wattling bundles are then laid over the top of the basal ends of the willow in the trench and secured tightly with wedge-shaped stakes.

**Figure 27.** Sequence of brushmattress and wattling bundle installation. Note that this was done in dormant season in the fall even though some leaves remain on branches.

estimated at 6.5 fps. The 350 ft radius of curvature in the project reach, as measured off of a 1981 aerial photograph, results in increased localized velocities (Goldsmith, 1993). A more detailed explanation of this example appears in the case study in Volume II.

*Brush layering*. Brush layering, also called branch layering, or branch packing, is used in the splash zone, but only in association with a hard toe, such as rock riprap, in the toe zone. It can also be used in the bank zone as discussed later. This is a treatment where live brush that quickly sprout, such as willow or dogwood species, are used in trenches. Trenches are dug 2-6 feet into the slope, on contour, sloping downward from the face of the bank 10 to 20 degrees below horizontal (Figures 28-29). Live branches are placed in the trench with their basal ends pointed inward and no more than 6 inches or more than 18 inches of the tips extending beyond the fill face (Leiser, 1994). Branches should be arranged in a criss-cross fashion. Brush layers should be at least 4 inches thick (Leiser, 1994) and should be covered with soil immediately following placement and the soil compacted firmly.

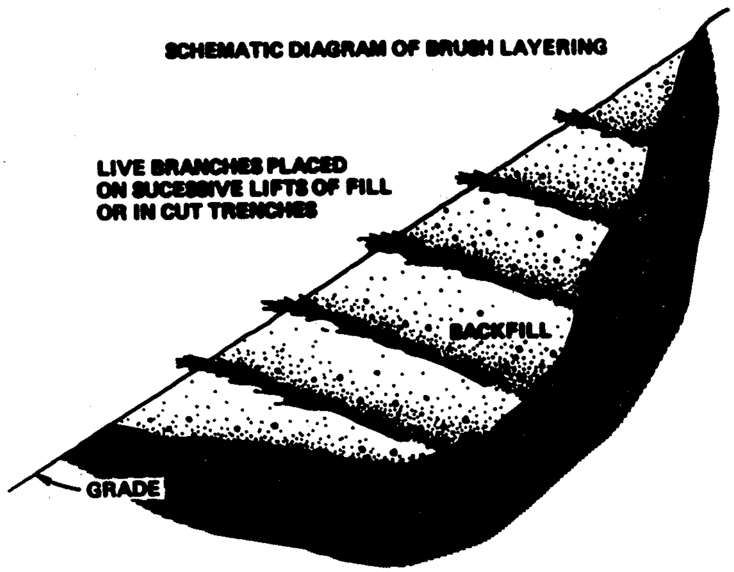


**Figure 28.** Schematic diagram of brush layering. (from Leiser, 1983)

**Figure 29.** Installed section of brush layering. Note that brush has leaves because of a workshop setting. Normally, the brush would be without leaves because of installation during the dormant season.

Brush layering (branch packing) was used successfully on the Little Patuxent River in Maryland (Figure 30). There, it was used in combination with live facines (wattles) and live pegs (Bowers, 1992). Rock riprap was placed at the toe of the streambank for added protection. Bowers (1992) reported that the top growth of the live facines, live branches in the branch layering, and live pegs (live stakes or cuttings) provide coverage of and protect the streambank during storm events. The species used included black willow and silky dogwood. Branch layering and live facines were used in the low energy zones of the river, i.e., along the beginning and end of outside meanders. For the areas where the thalweg came in contact with the streambank on the outside of the meander, root wads were used for protection and stabilization (Bowers, 1992).

*Vegetative geogrid*. This is a system that can be used in the splash zone and actually extend further up the bank into the bank and possibly terrace zones. The system is sometimes also referred to as "fabric encapsulated soil." It consists of successive walls of several lifts of fabric reinforcement. In between the lifts are placed 5- to 10-ft long live willow whips. This system is described by Miller (1992) and was used successfully on Acid Brook in New Jersey. It was also used on the Upper Truckee River near South Lake Tahoe along with a few other treatments and will be discussed in more detail in Volume II. The design, according to Miller, is based on a dual fabric system modeled after synthetic fabric retaining walls used by engineers for road embankments and bridge abutments. The generic system is shown in Figure 31. Two layers of coconut fiber-based fabric provide both structural strength and resistance to piping of fine material. Piping is that process where internal erosion of soils occur; that is, water seeps in from above through a porous layer of soil, such as sand lenses, and erodes that layer from where it enters to where it exits further down slope. The inner layer is a loose coconut fiber blanket held together by synthetic mesh netting and is used to trap finds and prevent piping. The outer layer is a strong, woven coir fabric to provide structural support. Sometimes, the latter fabric is substituted by even stronger, more durable synthetic materials, that are formed by a matrix of geosynthetic bands. The disadvantage of the latter materials, however, is that they are not very biodegradable. Of course, vegetation would mask the materials so they are not visible.

Miller (1992) describes building the lifts of fabric-reinforcement as follows:

> "To build the streambanks, we would first lay down a layer of each fabric in the appropriate location. We'd place fill material, compact it, and wrap the exposed fabric over the face of the fill. The fabric would be keyed back under the next layer with wooden stakes. We'd progress upwards from layer to layer, whether the slopes were vertical or at a 3:1 slope."

**Figure 30.** Brush layering with willow and dogwood branches after one growing season; installed above a rock toe (to prevent undercutting) on the Little Patuxent River, Maryland. (From Bowers, 1992)

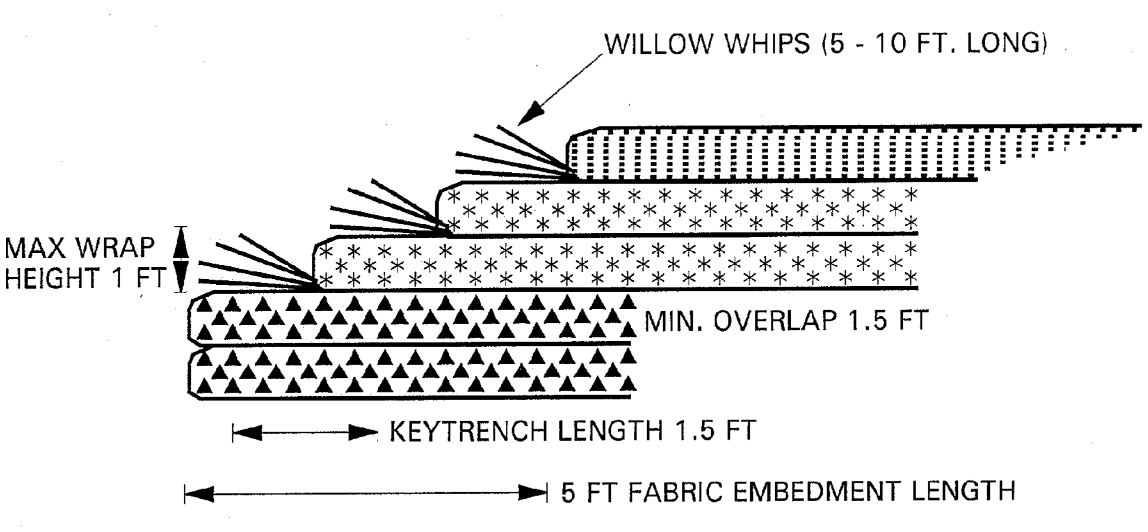


**Figure 31.** Cross-section of Vegetative Geogrid, also called Fabric-Encapsulated Soil with vegetation. (adapted from Miller, 1992)

Figures 32 and 33 show photographs of the Upper Truckee River site both before and after construction. The latter figure was taken in July 1995 after an extended high flow period from May 21 through July 21. There, Mr. Matt Kiese[4] (pers. communication) described building the lifts with the use of long angle iron forms. The angle irons were 8 ft long and were fashioned to form a frame into which plywood boards were inserted. Then, the forms were wrapped with two fabrics similar to those described above and soil dumped into the forms and compacted. The fabrics were wrapped back over the soil and the forms removed. Willow whips were laid on top of each lift and then the next lift was prepared. The installation at the Upper Truckee was no more than five feet tall and 123 ft long. Care must be taken to provide rock or some other hard material at each upstream and downstream end to prevent flanking of the treatment. For instance, one may either tie into existing vegetation, such as trees, or create hard ends by placing rock. Also, it is important to prevent scour at the bottom lift and to provide a good footing by creating a ditch and filling it with cobble or rock. The first lift is placed on top of the cobble ditch. The ditch at the Upper Truckee River site was about 2-ft wide by 2-ft deep.

This treatment was very successful on the Upper Truckee River despite the 5-yr flood event in May 1995 that produced overbank flows. The treatment remained in place since since October 1993. Further discussion about this treatment can be found in Volume II.

Dormant Post Method. This treatment consists of placing in the splash zone and perhaps the lower part of the bank zone dormant, but living stems of woody species that sprout stems and roots from the stem, such as willow or cottonwood. Willows are normally used and are cut into 10-14 ft posts when the leaves have fallen and the tree is dormant. The dormant posts store root hormones and food reserves (carbohydrates) that promote sprouting of stems and roots during the growing season. According to Roseboom (1993), dense stands of 4-6 year old willows make the best harvesting areas. He also uses posts that are 4-6 inches in diameter at the base. His examples are based on fast-growing eastern species, however, and smaller willow may have to be used in the western states.

Roseboom (1993) prescribes shaping a bank to a 1:1 slope with the spoil placed in a 6-inch deep layer along the top of the bank. In major erosion sites, post holes are formed in the bed and bank so that the end of the post is 2 ft below maximum streambed scour (that portion of the streambed that is subject to movement). Hoag (1993) suggested that for bank stabilization, the cutting (post) should extend 2-3 ft above ground so as it leafs out, it can provide immediate bank erosion protection. He also recommended the cutting should be planted as much as 3-5 feet into the ground. If they are not this deep, moving water can erode around the cutting and rip it out of the ground. Roseboom places the posts four feet apart up the streambank. The posts in one row are offset from the posts in adjacent rows.

[4]Matt Kiese, Interfluve Inc., October 1993, personal communication

**Figure 32.** Vegetative geogrid during construction on the Upper Truckee River, California, near South Lake Tahoe. Note rock toe that was keyed into channel bed and bank to prevent undercutting. Photo was taken in October 1993.

**Figure 33.** Vegetative geogrid in July 1995, after two growing seasons and an estimated 5-yr flood during the spring of 1995. Note that live willow whips which were placed between the layers of COIR fabric are sprouting and spreading. (Photo courtesy of Ms. Catherine McDonald, Calif. State Parks)

Both Roseboom (1993) and Hoag (1993) advised that willow posts should be long enough and placed deep enough to reach wet soil during dry summers. Hoag (1993) noted that plantings can occur at the water line, up the bank, and on top of bank in relatively dry soil, as long as cuttings are long enough to reach into the mid-summer water table.

An excavator that is either fitted with a long, steel ram or an auger is typically required for installation. Roseboom (1993) reported that a steel ram on an excavator boom is more efficient at depths of 6 feet in clay soils. In contrast, an auger on an excavator boom forms deeper and longer lasting holes in stoney or sandy streambeds. The ram on the excavator is for creating a pilot hole in which to place the willow post. The willow post is fitted with a cap that goes over the post and then the heel of the bucket on the excavator is used to push the post down into the hole. Care must be taken to ensure that the post comes in contact with the soil so that no air pockets exist. In the case of the auger, this can be done by backfilling the sides of the hole in lifts and then tamping. In the case of the ram, the ram can be placed out a few inches from the post and run along the side of it into the soil so as to close the hole containing the post, especially toward the bottom of the hole.

Roseboom (1993) reported that in larger streams with non-cohesive sand banks, large cedar trees cabled to the willow posts along the toe of the bank can reduce toe erosion. The cedars not only reduce bank scour while root systems are growing, but retain moisture during drought periods. Another material used for the same purpose is a coir roll mentioned earlier. In addition to trapping sediment, the coir roll can be planted with either emergent aquatic vegetation or other willow cuttings. The cedar trees and the coir roll were used in combination with willow poles on Court Creek, Illinois, along a 600-ft reach. Figures 34 and 35 respectively illustrate work in progress and bank conditions four months after planting. This is described in a case study in Volume II. Velocities were measured at this site during a major 1995 flood and ranged between 1.23 to 3.11 fps. They were measured at distances immediately in front of the treatment to 3.5 ft in front and at both the surface and 0.6 d. It is suspected that the willow contributed substantially to reduced velocities near the bank.

Hoag (1994a) and Hoag (1994b) provided specifications for and description of another type of implement that is used to make a pilot hole for the dormant willow post. It is called "The Stinger" and has been used by the USDA Natural Resources Conservation Service (NRCS) and the Bureau of Reclamation for establishing willow in riprapped revetments on shorelines of reservoirs and streambanks. According to Hoag (1994b), woody vegetation has been planted in rock rip-rap in the past, but the methods have concentrated on planting the cuttings first and dumping rock on top of them or planting through the rock riprap with a steel bar or water jet (Hoag 1994b cites Schultze and Wilcox 1985).

Hoag (1994b) states: "Neither of these methods are very efficient nor have achieved great success. 'The Stinger', however, builds upon these methods and utilizes the power of a backhoe to plant much bigger diameter and much longer cuttings than was possible before. "The Stinger" can plant cuttings right through rock riprap with minimal effort to better stabilize the rock, allow the cutting to be above the ice layer, and to improve the aesthetics of the riprap." "The Stinger" can plant through 2 to 3-ft riprap, but it must penetrate the moist soil below in which to push the dormant willow pole."

**Figure 34.** Dormant willow posts, coir geotextile roll, and cedar trees being installed at Court Creek, Illinois, April 1993. (Photo courtesy of Mr. Donald Roseboom, Illinois State Water Survey)

**Figure 35.** Court Creek site above after one growing season. Note that this is after one major flood in spring and summer, 1994, that overtopped the banks. (Photo courtesy of Mr. Donald Roseboom, Illinois State Water Survey)

"The Stinger" was used on a bioengineering project on the upper Missouri River by the Omaha District, Corps of Engineers, in April 1996, to place dormant willow posts between and landward of large haybales used in the toe zone, as mentioned briefly above. "The Stinger" was used for efficiency and ease of construction (Figure 36).

**Figure 36.** Use of "The Stinger" to create pilot holes for dormant willow posts on the upper Missouri River (CE project, Omaha District).

There are constraints in using willow posts and several questions to be addressed in the process of planning if this method is considered. These are noted by Roseboom (1993), but have been modified here:

a. Does sunlight fall directly on the eroding bank? Willows must have at least partial sunlight to grow.

b. Is bedrock close to the surface? The soil should be at least 4 ft deep; this can be checked with a probe.

c. Are lenses of fine sand exposed in the eroding bank? If so, piping may be a problem and other methods of controlling piping need to be addressed for the dormant post method to be successful. This may be done through the brushmattress technique mentioned above in combination with a geotextile filter or it could be done by use of the vegetative geogrid technique mentioned above.

d. Is the stream channel stable upstream of the erosion site? If the stream cuts behind the upper end of willow posts, the entire bank will erode.

e. How deep is the stream along the eroding bank? Willow posts must penetrate to a depth that is deeper than the water near the eroding bank. There should be a shelf or at least a sloping bank that allows willow posts to penetrate at least 2 feet deeper than the deepest water at the shore or the posts will be undercut below the root zone. If this cannot be achieved by the willow posts, then some kind of hard toe, like a rock revetment, should be used to prevent scour beneath the posts. The length of the willow posts will depend on the water depth as well as the dryness of the soil above the stream level.

f. How wide is the stream channel at the erosion sites when compared to stable channels upstream and downstream? The channel with vegetation at the erosion site(s) should not be narrower than stable channels upstream or downstream; otherwise, vegetation could choke the channel and cause other erosion problems.

g. Do you have a source of large willows close to the site? Costs are less when willow stands are close because of less transportation costs. Also, there is less chance of mortality due to long durations of handling and possible drying of the willow.

h. Will the site be wet during dry summers? Willow posts require considerable water while the roots are becoming established from the root primordia on the stems. For dry sites, such as in the western states of the United States, tops of willow posts should be only 1-2 feet above ground and they should penetrate into at least the capillary zone of the groundwater table. Figure 10 shows willow posts being used in eastern Montana on the upper Missouri River in combination with a line of coir-covered haybales for toe protection. In similar cases, care should be taken to ensure the posts are cut off not more than two feet above ground and that they penetrate the groundwater.

i. Can you keep cattle and other animals, domestic or wild, away from the posts during the first summer? Willows and other plants produce food for regrowth from leaf photosynthesis. If these sprouting branches with leaves continue to be browsed or if the tops of the plants continue to be cut off by beaver during the first growing season, they could die. It is best to prevent this by keeping cattle off of the area and either trap beaver off the area or spray the willow stems with organic beaver deterrent sprays, made with such constituents as mountain lion urine. It should be noted, however, that beaver damage during subsequent years of development may only promote resprouting of branches from the main stem and actually promote a shrubby-like plant. This is a positive effect from a surface roughness perspective whereas the many branches slow the current and promote sedimentation that can lead to other plant colonization.

j. Have debris jams or trees and logs forced floodwater into the eroding bank? These must be removed at least to the point where they are not directing water into a bank. Trees and logs can be moved parallel to the bank and cabled to dead men.

Care should be taken, however, to ensure the upstream end is not flanked by currents, thus possibly jeopardizing that bank reach.

The dormant post method using willow provides a low-cost bank stabilization method with both wildlife and fisheries benefits. Roseboom (1993) reported that the method has received widespread support by both the agricultural and environmental communities: Farm Bureau, Soil and Water Conservation Districts, American Fisheries Society, and the Nature Conservancy. The willows hold the soil together long enough for other plants to become established on the bank through succession. Together, they provide a natural system of food and cover. More can be found on this method in the case study provided in Volume II.

*Dormant Cuttings*. Dormant cuttings, sometimes called “Live Stakes,” involves the insertion and tamping of live, rootable cuttings into the ground or sometimes geotextile substrate. In higher velocity streams, such as over 5 fps, this method usually is applied in the splash zone with a combination of other methods, such as the brushmattress and root wad methods. Dormant cuttings can be used as live stakes in the brushmattress and wattling as opposed to or in combination with the wedge-shaped construction stakes previously mentioned. Or, they can be placed adjacent to the brushmattress. They can also be used in the matrix openings of the root wad logs along with root pads of other vegetative materials. If cuttings are used alone in the splash zone, the toe should be very stable and velocities should be less than 5 fps. Also, the soil in which they are placed should be fairly cohesive. Figures 37 a-c show an application of bankers (*Salix* X *cotteti)* and streamco (*S. purpurea* 'streamco'*)* willow cuttings that was installed on Irish creek in North Carolina by the NRCS. These willow were installed on a fairly cohesive bank on a staight reach with a stable toe.

Dormant cuttings can vary in size, but are usually a minimum of 1/2 inch in diameter at the basal end (Hoag, 1994b). Cuttings can be used that are up to 2 to 3 inches in diameter and have been noted by Hoag (1993) to have the highest survival rates. Cutting length is largely determined by the depth to the mid-summer water table and erosive force of the stream at the planting site (Hoag 1993). Plantings can occur at the water line as in the splash zone, up the bank into the bank zone, and on top of the bank (terrace zone) in relatively dry soil, as long as cuttings are long enough to reach into the mid-summer water table (Hoag 1993).

Cuttings should have their side branches cleanly removed and the bark intact so that the cutting is one single stem. Care should be taken to make clean cuts at the top and the bottom so that the bark is not separated from the underlying woody tissue. Also, be sure they are cut so that a terminal bud scar is within 1 to 4 inches of the top because cuttings put out their greatest concentration of shoots and their strongest ones just below an annual ring (formed from a terminal bud scar). At least two buds and/or bud scars should be above the ground after planting (Gray and Leiser, 1982). Tops are normally cut off square so they can be tamped or pushed easily into the substrate. The basal ends are often angled for easy insertion into the soil. When selecting material from a natural stand, care should be taken to see that the harvest material is free from insect damage, disease, and splitting.

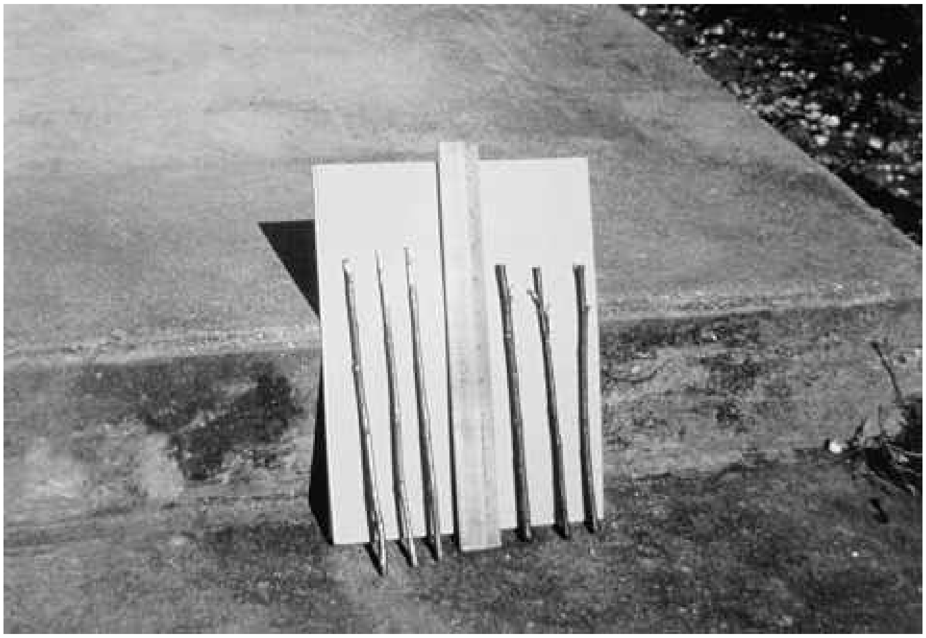

**Figure 37a**. 8 inch live cuttings of streamco and bankers willow used to stabilize Irish Creek.

**Figure 37b**. Photo of Irish Creek during the first growing season.

**Figure 37c**. Reach of Irish Creek stabilized with cuttings of willow. Photo taken 4 growing seasons after planting.

**Figure 37.** Irish Creek, North Carolina, stabilized with cuttings of bankers and streamco willow. (Photos courtesy of USDA Natural Resources Conservation Service)

*Root pads.* Root pads are clumps of shrubbery composed of such species as willow (shrubby forms), redosier dogwood, european alder (*Alnus glutinosa*), and others. It is often used in the splash zone as a part of root wads where the root pads are positioned in between them. Root pads can also be used further up the slope into the bank and terrace zones. Caution should be exercised in planting these during the dormant season. They can be removed from harvest areas and placed at the project site with front-end loaders. “Veimeer” type spades are sometimes used on root pads where species have deep penetrating roots whereas front-end loaders are used on species whose roots spread out more at the surface. Placement of root pads on slopes greater than 1V:6H should include securing the root pads by driving 2-in diameter, 18 to 24-in long wooden stakes through the pads at 2 to 3-ft intervals (Logan et al., 1979)

### ***Bank Zone***

This zone may be exposed to considerable flooding and current and wave action. If only mild current and wave action is expected, sodding of flood-tolerant grasses like reed canagry grass, buffalo grass (*Buchloe dactyloides*), or switchgrass (*Panicum virgatum*) can be employed to provide rapid bank stabilization. Usually, the sod must be held in place with some kind of wire mesh, geotextile mesh such as a coir fabric, or stakes. A soilless system for growing wetland plants in coconut fiber mats (coir mats) was discussed above for the splash zone and can be extended up into this zone as well.

Instead of using sod in this zone, the California Department of Parks used seed from wetland plants, such as various sedges and grasses, in combination with burlap and a coir woven fabric (0.8 lbs/sq yd) laid over the seed (Figure 38). This whole system was placed in the bank zone above root wads and willow clumps that were installed in the toe and splash zones, respectively. The combination of root wads, willow clumps, and this seeding and burlap/coir combination was stable in most reaches where it was installed although vegetative cover from the planted seed was less than expected. This treatment, along with others, are described in Volume II.

**Figure 38.** Burlap and coir woven fabric laid over sedge and grass seed, Upper Truckee River, California. Note that the fabrics were keyed in at the top and bottom in trenches and securely staked with wedge-shaped stakes. (Photo courtesy of Interfluve, Inc.)

To augment the sodding practice for this milder energy regime, shrub-like willow, dogwood, and alder transplants or 1 year-old rooted cuttings are effectively used in this zone (Edminster et al. 1949; Edminster 1949; and Seibert 1968). These transplants or cuttings should be planted about 0.5 m apart and in rows. Further planting practices can be found in Edminster et al. (1949) and Edminster (1949). Newly planted banks are usually subject to additional erosion and the shrub plantings should have mulch placed over them to serve as temporary protection. Mulch of woody plant branches are best for this and should be the heaviest on outside curves of the stream where the current strikes the bank. The mulch should be tied down with chicken wire or wire laced between stakes since the mulch may float away when flooded (Edminster 1949).

Where severe erosion is expected and currents on the bank are expected to exceed 8 fps, methods such as the brushmattress discussed for the splash zone above should be carried up into the bank zone. Additionally, two other methods using woody materials are appropriate for this zone. They include contour wattling and brush layering.

*Contour Wattling*. Contour wattling was discussed above as an integral component of the brushmattress. In the bank zone, and in this context, it may be used independent of the brushmattress along contours. Sometimes, you will see the term "fascine" in lieu of the term wattling. They are buried across the slope, parallel or nearly parallel to the stream course, and supported on the downhill side by stakes (Figures 39 a-c). They also have stakes driven through the bundles and can be either living or constructed from wood as previously described. The sprouting attributes of the brush species used, such as willow, combined with the supportive attributes of the structure itself provide an integrated system of stems, roots, wire, and stakes that hold the soil in place. When used on slopes, they protect against erosion caused by downward water flow, wind action, trampling caused by wildlife and livestock, and the forces of gravity. Further descriptions of wattling (fascine) construction can be found in Edminster (1949), Schiechtl (1980), Gray and Leiser (1982), Allen and Klimas (1986), Coppin and Richards (1990), and Georgia Soil and Water Conservation (1993).

Contour wattles (fascines) are often installed in combination with a coir fiber blanket over seed and a straw mulch. In this way, slopes between the wattles may be held firmly in place without development of rills or gullies. Figure 40 illustrates this and was prepared by Robbin B. Sotir and Associates for the Corps of Engineers Nashville District and successfully used on the Tennessee River near Knoxville, Tennessee. It should be noted that there was significant toe protection in the toe zone with rock riprap; however, there was also overbank flooding shortly after installation of the contour wattles and the treatment was stable.

*Brush layering*. Brush layering can be used in the bank zone as it was in the splash zone except with some modifications. Geotextile fabrics, such as coir woven fabrics, should be used between the layers and keyed into each branch layer trench, so that unraveling of the bank does not occur between the layers (Figure 41). Before the geotextile fabric is applied, the areas between the branch layers should be seeded with flood-tolerant grasses or grass-like plants, like sedges, and then covered with a straw mulch. This method was used to stabilize levees in low-lying areas of fen districts in England (from Gray and Leiser, 1982 who cited Doran, 1948). Slope heights, the vertical distance between the layers, should not exceed 3 times the length of the longest brush in the trench. This would be similar in principle to a sloping reinforced earth revetment (from Gray and Leiser, 1982 who cited Bartos, 1979) where metal strips are placed essentially horizontally in successive layers up the face of a

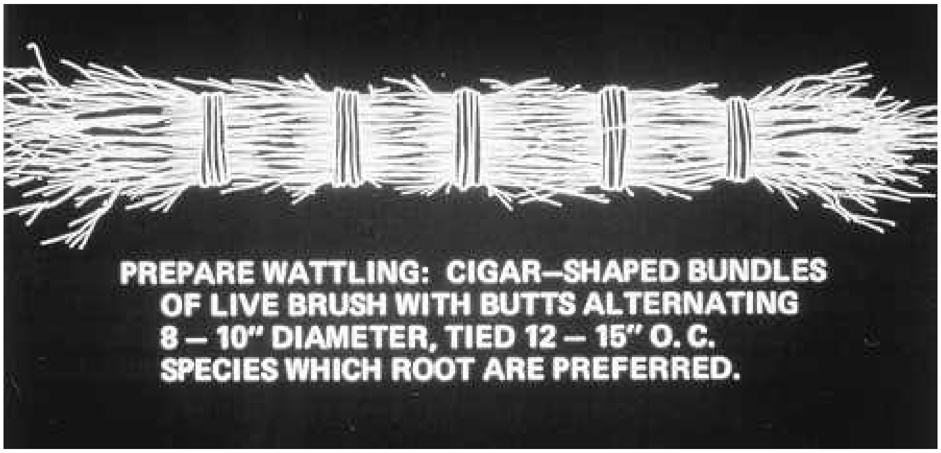


**Figure 39a**. Schematic of wattling bundle with preparation specifications. (from Leiser, 1983)

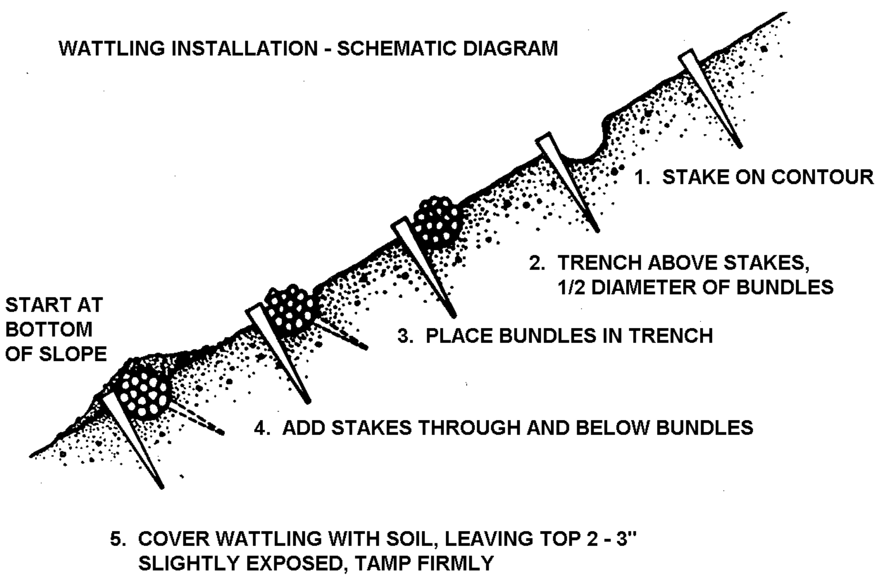


**Figure 39b**. Procedures for installing wattling bundles on slope in the bank zone. (from Leiser, 1983)

**Figure 39c.** Wattling (fascine) bundle being installed in the bank zone. Note that wattling should not be covered completely with soil; leave top 2-3" exposed for sprouting purposes. (Photo courtesy of Ms. Robin Sotir, Robin Sotir & Associates)

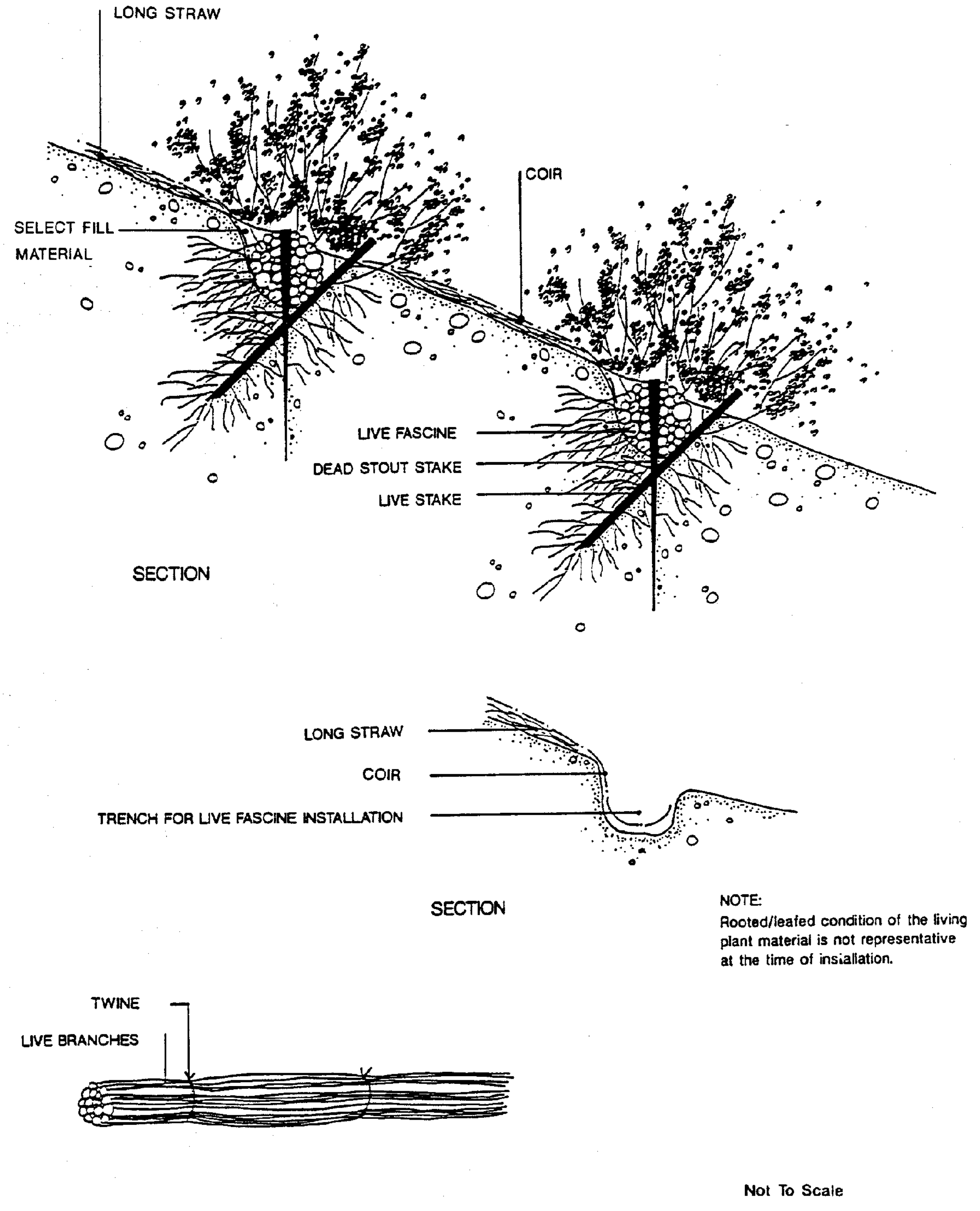


**Figure 40.** Schematic illustration of live fascine bundles with coir rope mesh fabric and long straw installed between the bundles. (from US Army Corps of Engineers, Nashville, 1993; schematic drawn by Robin B. Sotir & Associates, 1993)

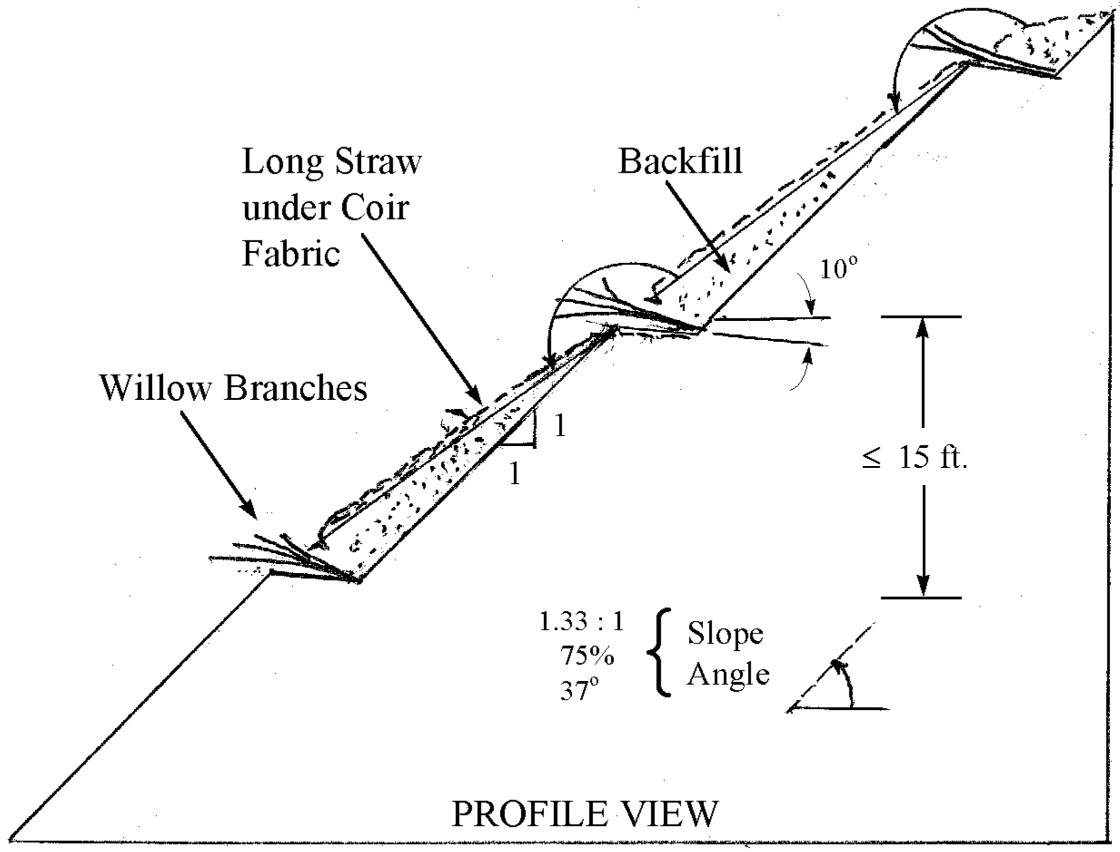


**Figure 41.** Brush layering with coir woven fabric and long straw under fabric. Coir fabric and straw help control rillying and gullying between layers. (Adapted from Gray and Leiser, 1982)

slope. In a reinforced earth revetment it is common practice to make the strip length (or width of reinforced volume ) about one-third the slope height (Gray and Leiser, 1982).

Brush layering lends itself to partial mechanization because the benches can be excavated with a small backhoe or grader. Regular construction equipment, such as a front-end loader with a clasp on the bucket, can be used for hauling and placing the brush. Backhoes or similar equipment can also backfill.

The choice between wattling and brush layering, according to Gray and Leiser (1982), should be based on economics, the potential stability of the fill (in this case, stability of the streambank), and the availability of suitable plant materials. Generally speaking, brush layering is considered to be less expensive than contour wattling. Brush layering stabilizes a fill or bank to greater depths, but more plant material is required than for contour wattling. However, if the streambank is disturbed to the extent that rebuilding and reshaping is necessary, brush layering may be the better alternative, because of its ability to stabilize a bank to greater depths.

Again, as it was in the earlier parts of this report, emphasis should be placed on prevention of flanking of the bioengineering treatment. In this case, either contour wattling or brush layering treatments should be protected with some kind of hard structure both upstream and downstream of the treatment. If natural hard points, such as large boulders, rock outcroppings, or hard geological strata, are not present, then one should consider use of a rock refusal. This would be rock riprap that starts at the bottom of the bank, continues up the bank, and is keyed into the bank (Figure 4).

*Terrace Zone*

This zone, as mentioned earlier, is rarely flooded and usually not subjected to erosive action of the stream except during occasional flooding. When flooded, it receives overbank flooding with return flows that can cause gullying and rilling to occur on the fall of the hydrograph. It is in this zone that vegetation is needed with deeply penetrating roots to hold the bank together, such as larger flood-tolerant trees. Grasses, other herbs, and shrubs can be planted in between the trees, depending on their shade tolerance. Bioengineering, per se, is not normally used in this zone unless there are deep gullies that have occurred as a result of return flows or slopes still occur in this zone that are 3H:1V or greater. In these cases, branch layering or contour wattling treatments are often employed across the gully or on the contours of the slope.

Care should be taken in using large trees in this zone. They should be planted far enough back from the bank that their shade does not kill out the vegetation in the splash and bank zones. Narrow channels, especially, can be completely shaded from one side. When trees are planted in this zone, they are planted either as container-grown (potted) or bare-root plants. Suggestions vary on the size of container-grown plants. Leiser (1994) suggests using containers with a minimum size of 9 cubic inches with a depth of 8 inches and a maximum size of no larger than one quart milk carton. Plants in larger containers increase the cost for purchase and planting substantially. Survival is frequently reduced because of limited root systems in relation to size of the tops of the plants (Leiser, 1994). The important thing to remember is to have a container with growing medium well filled with roots so that the roots and medium form a cohesive unit when removed from the container.

Woody materials (Hoag 1994b), whether they be grown in containers or derived from cuttings, should be used only in the bank and terrace zones when the following conditions exist:

a. where long periods of inundation or water erosion are minimized;
b. where adequate moisture is available, i.e., natural precipitation is adequate for species selected or plants are irrigated;
c. where there is no competing vegetation or a 30" diameter area around plant is scalped of competing vegetation at planting time;
d. where plants have a low risk of physically being pulled or eroded out due to shallow rooting system during the first year after being planted.

Hydroseeding and hydromulching can be a useful and effective means of direct seeding in the terrace zone, particularly on slopes greater than 3H:1V and places where it is difficult to get equipment. Sometimes, it is possible to work from a small barge and use hydroseeding and hydromulching equipment on the barge (Figure 42) and blow them onto the bank. If seeds are blown on in a water slurry, a generic type mix is suggested by Leiser (1994):

| | |
|---|---|
| Grass seed | 50 pounds/acre |
| Woodfiber mulch | 500 pounds/acre |
| Water | As needed |
| Fertilizer (if not broadcast) | 250 pounds/acre |

According to Leiser (1994), the slurry should be continuously mixed as ingredients are

**Figure 42**. Hydroseeding and mulching operation from a barge.

added and mixed at least five minutes following the addition of the last ingredients before application begins. The slurry should be continuously mixed until used and application must be completed within two hours of the last addition. Water should be potable or at least filtered so as not to clog spraying equipment. The slurry should be applied at a rate that is non-erosive and minimizes runoff.

On level areas and slopes of less than 3H:1V, seed should be broadcast by mechanical hand or power-operated spreaders or drilled on contour with a Brillion or range drill as site conditions permit. Broadcasted seed should be covered by raking or dragging with a chain, chainlink fence, or other approved means unless previously planted with cuttings or transplants (Leiser, 1994).

Sometimes surface drainage water intercepts the terrace zone from inland areas and can cause gullying not only in the terrace zone, but in the other zones on the bank. This water should be diverted or controlled with a small furrow or trench at the top of the bank. This trench should be sodded to prevent erosion.

## Velocities for Design Purposes

The purpose of this section is to provide some velocity information that bioengineering systems have been noted to sustain so that planners and designers have a basis for choosing bioengineering systems and the particular kind of system. Some of the velocity information was derived from the literature while other information was measured at local points at case study locations where bioengineering treatments were installed. Velocities vary so much within a stream that local velocities near the treated section are the most valuable. Admittedly, the measured velocities are much lower than considered maximum threshold values that could be sustained by the installed structures. This is because when measurements were made, they were made with current meters in the local vicinity of the bioengineering treatment on the fall of the hydrograph when water levels and currents during flood events were not a safety hazard. Remote current meters exist, but would have been silted in or damaged by debris flow during these flood events.

Most of the velocity information in the literature concerns itself with turf grass cover that was designed for erosion control ditches or waterways. Little information exists on combinations of systems, i.e., bioengineering treatments, containing both herbaceous and woody species. Engineer Manual, EM 1110-2-1205 (US Army Corps of Engineers, 1989), states that herbaceous or woody vegetation may be used to protect channel side slope areas (depending on the frequency of inundation, velocity, and geotechnical constraints to infrequent flooding) and other bank areas where velocities are not expected to exceed 6 to 8 feet per second (fps). Information concerning influence of vegetation (bermuda grass) or variation of velocity with depth below water surface is shown in Henderson and Shields (1984) who cites Parsons (1963).

The splash and bank zones will be the principal focus for bioengineering applications. It is in these zones that the designer must tailor vegetation types and bioengineering structures to be commensurate with velocities that they can sustain. Hoag (1993) suggests that maximum flow velocities should not exceed 3 fps for herbaceous plantings, 3-5 fps for woody and herbaceous mixed plantings, 5-8 fps for woody plantings alone, and that maximum flows above 8 fps require soil-bioengineering approaches.

For the case studies examined and monitored for this report, measured velocities for local flow conditions around the bioengineering treatment never exceeded 10 fps. Maximum velocities sustained and recorded by bioengineering treatment types are shown in Table 2. As previously mentioned, these may not represent the maximum velocities encountered, as they were usually taken on the fall of the hydrograph. Also, local roughness imparted by the bioengineering treatment would have slowed velocities in its vicinity.

**Table 2. Local flow velocities sustained by and recorded for various bioengineering treatments monitored by this project.**

| Location | Type of Bioeng'r Treatment | Maximum Velocity (fps) Recorded | Notes |
|---|---|---|---|
| Roaring Fork River, CO | Log revetment with coir geotextile roll and grass seeding above roll (See Figures 17 & 18) | 10.0 | Logs anchored in the bank with heavy duty cables. Rock jetties used for hard points at strategic points |
| Snowmass Creek, CO | Root wads with large root pads (clumps) of willow (See Figures 19 & 20) | 8.7 | Lack of maintenance during spring, 1994 (additional root wads at scour points) caused partial washout of the upper meander during spring flood of 1995. |
| Upper Truckee River, CA | Root wads with large clumps of willow (Figures 19 and 20) | 4.0 | Lower velocities measured in and around bioengineering treatment than further out into channel; this can be attributed to larger roughness coefficient |
| Court Creek, IL | Dormant willow posts with rock toe (Figures 34 and 35) | 3.1 | 4 rows of willow posts on 4-ft centers; 10-15 -ft long cedar trees between 1st two rows of willow; coir geotextile roll and riprap placed at toe along meander apex. |

Notes: These are local flow velocities noted in this table and were measured by a flow meter; All treatments were in their second growing season after major flood events when these measurements were taken.

Table 2 shows maximum local flow velocities around a root wad structure with willow root pads to be 4.0 and 8.7 fps for two different treatments at two geographic locations, Upper Truckee River, CA, and Snowmass Creek, CO. It is suspected that these kind of structures, if properly installed, could sustain velocities much higher than these. It was noted earlier in this report that D. Rosgen[5] measured local flow velocities around root wads on the Blanco River, CO, to be 12.0 fps.

Some of the treatments noted in Table 2 had some partial failures even though at least half of the reaches where these were installed remained intact and the treatments continued to function. The treatment containing the log revetment with coir geotextile roll on the Roaring

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[1] Rosgen, Dave. President, Wildland Hydrology, Pagosa Springs, CO, Jul 1996, personal communication.

Fork River, CO, experienced some failure. The lower half of the reach in which it was installed washed out after a major flood in the spring of 1995. This was due to the problem of insufficiently burying and keying in the bottom-most log of the revetment into the streambed. Consequently, scour undermined the structure and it failed along the lower half of the reach.

The root wad structure on Snowmass Creek, CO, had a partial failure. After the spring runoff in 1994, the sponsor noticed minor damage around certain critical points that needed maintenance, the addition of more root wad logs. The contractor instead placed rock at inappropriate places. Consequently, the creek flooded during the spring runoff of 1995 and the outside of the lower section of the upper meander washed out and eroded about six feet of bank. In these two cases, it points to the need for properly keying in structures for toe and end protection and to monitoring and possible maintenance early in the life of a bioengineering project. This early monitoring and maintenance should be included in the construction contract at the outset.

# 3 Plant Acquisition And Handling

Almost all of the plants used in bioengineering can be considered wetland plants, either obligative or facultative. Some of the exceptions would occur in the terrace zone that is infrequently flooded; however, all must be somewhat flood-tolerant. Both herbaceous and woody plants are used. Herbaceous plants may be emergent aquatic plants like rushes and sedges or grasses and other forbs that require non-aquatic, but moist conditions at least part of the year. The herbaceous plants are usually acquired as vegetative material such as sprigs, rhizomes, and tubers. Sometimes seed is acquired, but is used when the threat of flooding is low in the bank and terrace zones. Otherwise, they would wash out quite easily unless they are seeded underneath or in a geotextile mat or fabric that is securely anchored.

Woody plants used for bioengineering purposes usually consist of stem cuttings, those that quickly sprout roots and stems from the parent stem. These are plants such as willow, some dogwood, and some alder. They can be supplemented by bare-root or containerized stock, particularly in the bank or terrace zones where they are not subjected to frequent flooding. Gray and Sotir (1996) list several such plants that can be used in bioengineering and relate their flood tolerances, along with some other characteristics.

There are three suitable methods to acquire plants for bioengineering treatments. Each has, according to Pierce (1994), noteworthy advantages, but critical disadvantages that make plant acquisition and handling an important and complex process. The three methods are to: a) purchase plants, b) collect plants from the wild; and c) propagate and grow plants.

Regardless of the method chosen, it is necessary to conduct the following steps (Pierce, 1994):

a. Determine the available hydrologic regime and soil types. General positioning of the plant type, e.g., emergent aquatic, shrubby willow, should be in accordance with the plant zone (splash, bank, and terrace) defined in Part II.

b. Prepare a list of common wetland plant species in the region and more preferably, in the watershed containing the stream of concern, and match those to the hydrology and substrate of the target streambank reach to be addressed.

c. Select species that will match the energy of the environment and the hydraulic conveyance constraints that may be imposed by the situation. For instance, one must be careful to use low-lying and flexible vegetation that lays down with water flows if hydraulic conveyance must be maximized. In such cases, use flood-tolerant grasses or grass-like plants and shrubby woody species.

d. Select species that will not be dug out or severely grazed by animals, especially muskrat (*Ondatia zibethieus*), nutria (*Myocastor coypes*), beaver, Canada geese, and carp (*Cyprinus carpio*). Other animals may influence plant growth and survival. If plants chosen are unavoidably vulnerable to animal damage, then plant protection measures must be used, such as fencing, wire or nylon cages around them, or use of repellents.

e. Determine additional special requirements and constraints of the site. For instance, some sites may be prone to sediment deposition or have a bank geometry that is almost vertical. In such cases, it may be difficult to obtain success with emergent aquatic plants that may become covered with sediment and suffocate or which have too deep of water in which to grow unless the bank is reshaped. The former situation may necessitate the use of willow that can be planted as cuttings or posts and be less susceptible to complete coverage by sediment.

f. Prepare a suite of species that would be suitable. This may be limited to those currently available from commercial sources if there is no possibility to collect in the wild or have plants contract grown.

Pierce (1994) also gives a number of steps and advantages and disadvantages of the three methods of acquiring plants and these have been adapted with some modifications below. Each project will have unique situations, but the following will serve as a guide.

*Purchasing Plants*

a. Acquire a list of wetland plant suppliers, such as "Directory of Plant Vendors," (USDA Soil Conservation Service, 1992). Request vendors' catalogs and plant availability lists.

b. Determine in what condition the plants from each supplier are delivered, potted, bare root, rhizomes and tubers, or seed. This is important because if the plants are to be used in the splash zone where they may be partially covered with water, seed of emergent aquatic plants will not germinate under water.

c. Match the plant list against species availability, and do not assume that all species advertised will be available in needed quantities.

d. Order samples, if available, and verify plant condition and identification.

e. Negotiate a flexible delivery schedule allowing for unpredicted delays in planting.

f. Some suppliers may grow plants on contract but it will be necessary to contact them several months to a year before the plants are needed.

*Advantages of Purchasing Plants*

a. Plants are readily available at the planting location in predicted quantities and at the required time.

b. No special expertise is required to collect or grow the plants.

c. No wild source for the plants must be found and there are no harvesting permits to obtain from state or local governments.

d. Cost can be more readily predicted and controllable than harvesting from the wild or growing your own.

*Disadvantages of Purchasing Plants*

a. Plants may arrive in poor condition.

b. Selection of species is limited.

c. Plants may not be adapted to the local environment. Contract growing may solve this problem.

d. Cost may be high and shipping cost needs to be considered.

e. Quantities may be limited.

f. It may be necessary to store large quantities of plants and consequently necessitate procurement of adequate and appropriate storage facilities.

*Collecting Plants from the Wild*

Collecting plants from the wild may be very demanding because of “hard-to-reach” plants that are off main access routes. Wild plants must then be moved immediately to a nursery or hold-over site or to the project site. Logistical and plant handling problems need to be carefully assessed and solutions planned well ahead of time. Care should be taken if this method is selected because of the possibility of contaminating the harvested donor plants with unwanted weedy species that could become a problem at the project site. Samples should be collected ahead of time in order to determine what kind of problems will be encountered in collecting, transporting, and storing each species. Caution should be exercised in collecting plants from harvesting areas so that the plant community is not extirpated, left functional, and the ecosystem not damaged. This can be done by not harvesting in one spot, but dispersing the harvest areas. Care should be taken by harvesting only fairly common plants. Certainly, rare plants should be avoided.

*Advantages of Collecting Plants from the Wild*

a. Plants are likely to be ecotypically adapted to the local environment.

b. Plants can often be collected at a low cost.

c. Plants can be collected as needed and will not require extended storage.

d. Availability of species is very flexible and can be adjusted as the need arises.

e. No special expertise is required to grow the plants.

f. A very wide diversity of plants is available.

*Disadvantages of Collecting Plants from the Wild*

a. Weedy species may contaminate the source area and be inadvertantly transplanted.

b. A suitable area must be found, and more than one donor area may need to be located.

c. Plants may not be in an appropriate condition for planting. For instance, they may be highly stressed, diseased, or insect infested.

d. Species must be accurately identified or rare plants or weeds may be harvested by mistake.

e. Cost of collection and logistics may be very high.

f. Outdoor hazards such as snakes, adverse weather, noxious plants, e.g., poison ivy and stinging nettles, parasites, and other inhibiting items may interfere with collection efforts.

g. It is often necessary to procure a permit for collecting from native plant sources and wetlands, in particular.

*Growing Plants*

Plants to be grown for planting can be grown in a greenhouse or other enclosed facility or in the case of emergent aquatics, outdoor ponds or troughs containing water. In either case, the plants must first be acquired from the wild or other growers, and propagated. If seeds are used for propagation, they must first be stratified (subjected to various treatments such as soaking and temperature differences), but germination requirements for most wetland plant seeds are unknown. If a greenhouse is to be used, a number of limitations and constraints must be overcome, such as room for pots, adequate ventilation, and requirements or problems associated with fertilizing, watering, and disease and pest control.

Plants can be grown in coir carpets (Figures 25 a-c), mats, or rolls, to facilitate early establishment, ease of transport, and rapid development. Emergent aquatic plants, especially, may be hydroponically grown in the greenhouse or in outside troughs. Then, they can be transported to the planting site ready to grow with roots already established in the carpet, mat, or roll. The Waterways Experiment Station used a coir carpet for this purpose in 1983 for growing and transporting ready-to-grow plants to a site in Mobile Bay for erosion control of dredged material. This same concept can be used along streambanks and can be used to an advantage when one is in an area with short growing seasons or where rapid installation is mandatory.

*Advantages of Growing Plants*

a. All of the advantages of purchasing plants can be realized.

b. The variety of species available can be as diverse as for plants collected in the wild and plants can be planted in large quantities.

c. Plants can be available earlier in the season than purchased or collected plants.

d. Low cost is one of the primary reasons to grow stock for planting.

*Disadvantages of Growing Plants*

a. Space and facilities must be dedicated to growing plants.

b. Personnel with time and expertise to grow the plants may not be available.

c. There is an up-front investment in both fixed and variable overhead items in order to establish a growing facility and it may not be justified unless there is a large and continuing need for planting stock.

Handling of Plant Materials

Plants need to be handled carefully to ensure their survival between the phases of acquisition (purchasing, growing, or harvesting from the wild) and transplanting because they will undergo transportation and planting shock. Many problems associated with poor plant survival occur from the handling of the plants between the nursery or collection site and the project planting site. Generally, the plant material needs to be kept cool, moist, and shaded (Hoag, 1994). They must be treated as living material; if the living attributes are lost, then the project is much more prone to fail even though dead plant materials in a bioengineering treatment can offer some erosion control through their physical attributes, e.g., acting as bank armor, runoff retention through checkdam effects, current and wave deflectors. Plants are most easily collected when dormant. When plants are dormant, there is substantially more forgiveness in how they are handled.

**Woody Plants.** Woody plants, particularly cuttings, should be collected when dormant; their survival decreases a lot if they are harvested and planted in a non-dormant state. With bareroot or unrooted cuttings, keep them cool, moist, and in the dark until they are ready to be planted (Hoag, 1994b). They can be stored in a large cooler at 24-32 deg F until just before planting. Cuttings can be stored in this manner for several months (Platts et al. 1987). The cuttings can be kept in a cooler, root cellar, garage, shop floor, or any place that is dark, moist, and cool at all times (Hoag, 1994b). Often, cuttings are placed on burlap and covered with sawdust or peat moss and then covered with burlap after being moistened.

Hoag (1994b) advocates soaking of cuttings for a minimum of 24 hours, whether they are coming out of storage or directly after harvesting in the late winter to early spring (Hoag et al. 1991a; Hoag et al. 1991b; Hoag 1992). Some research recommends soaking the cuttings for as much as 10-14 days (Briggs and Munda 1992; Fenchel et al. 1988). The main criteria is that the cuttings need to be removed from the water prior to root emergence from the bark. This normally takes 7 to 9 days (Peterson and Phipps 1976). Soaking is important because it initiates the root growth process within the inner layer of bark in willows and poplars (Hoag, 1994b).

When woody plants are moved from the nursery, holding, or harvesting area, to the project site, they should continue to receive careful handling by keeping them moist and free from wind dessication. The latter can be achieved by ensuring they are covered with a light-colored (to reflect heat) and moist tarp. In the case of cuttings, they can be moved to the project site by moving them in barrels with water in them or some similar method. Actual planting of the plants shall follow the digging of holes as soon as possible, preferably no longer than 2-3 minutes, so that the excavated soil does not dry out. Use only the moist, excavated soil for backfill of the planting hole. Backfill should be tamped firmly to eliminate all voids and to obtain close contact between the root systems and the native soils. When using containerized or balled and burlap stock, excess soil should be smoother and firmed around the plants leaving a slight depression to collect rainfall. Plants should be placed 1 to 2 inches lower than they were grown in the nursery to provide a soil cover over the root system (Leiser, 1994).

**Herbaceous Plants.** Plant handling requirements of herbaceous plants are even more rigorous than woody plants as a general rule because they are usually obtained in the spring when nurseries have them ready to ship or when they are readily identified in the wild for collection. At those times, they are very susceptible to dessication mortality. Consequently, they must be kept in a moist, shaded condition, or even better, in water-filled containers from the time of collection from the wild or receipt from the nursery to the time of transplanting. If herbaceous plants are identified and tagged for collection in the spring or summer, they can be collected when dormant in the late fall or winter. During those times, they can be handled more freely, but should still be prevented from drying out. When transporting from the nursery, holding, or harvesting area to the project site, this should be in a covered vehicle. If the weather is very hot, cooling from ice or refrigeration may be necessary. Exposure to high winds should be avoided. Plants can be placed in a water-filled ditch and covered with soil in a shaded area for storage of several days while awaiting planting. It is best not to store plants longer than necessary, and delivery should be scheduled to match planting dates.

If herbaceous plants are to be grown, they will need to be grown from seed or from collected rhizomes, tubers, or rooted stems or rootstock from the wild. Most wetland plant seed needs to be stratified and will not germinate under water even after stratification. An experienced wetlands nursery person should be consulted before attempting to grow wetland plants from seed. Often, a cold treatment under water is necessary for stratification (Pierce, 1994). There are various other stratification methods of wetland plants, such as hot and cold temperature treatments and treatments with various fertilizers. Rhizomes, tubers, and rooted stems and rootstock of wetland herbaceous plants can be grown out in wet troughs or ditches and ponds containing fertilized sand and peat moss. Only enough water is necessary to keep the rhizomes, tubers, etc. from drying out. Plants can be grown out in the greenhouse over colder months, but will require hardening before transfer to the project site.

Hoag (1994b) stated that hardening off can be accomplished by removing the plants from the greenhouse and placing them in a cool, partially shaded area for 1-2 weeks. This is generally a lathe or slat house. Some are constructed with snow fencing which has wooden slats woven together with wire. According to Hoag (1994b), this type of structure allows a small amount of direct sunlight and solar radiation through the slats to the plants, but not enough to burn them. A partially shaded spot near the planting site will also work. It is important to keep the plants well watered and misted during the hardening off period.

# 4 Monitoring and Aftercare

The Philosophy of Monitoring and Aftercare

Most agencies and private entities cannot afford extensive monitoring in an operational setting in contrast to very definitive monitoring in a research and development setting. This discussion focuses on the operational setting. Bioengineering projects continue to grow stronger and stronger, once bed degradation is controlled, toe undercutting and scouring at upper and lower ends of reach have been arrested, and plants become established. Deeply penetrating plant roots hold the soil together and upper stems deflect current and wave energy and slow local flow velocities. Then, sedimentation takes place and other pioneer plants start to invade and further contribute to stability. The key, however, is to ensure that this early-on establishment of plants takes place and this requires early monitoring and possible remediation. Thus, early maintenance may be called for if this establishment is jeopardized. In contrast, traditional projects such as riprapped revetment, may not require maintenance early in the project life, but may need major maintenance at a much higher cost a few years later. So, bioengineering may require early-on monitoring and remediation with the trade-off being no maintenance or little maintenance in later years. Figure 43 (from Coppin and Richards, 1990) illustrates this point.

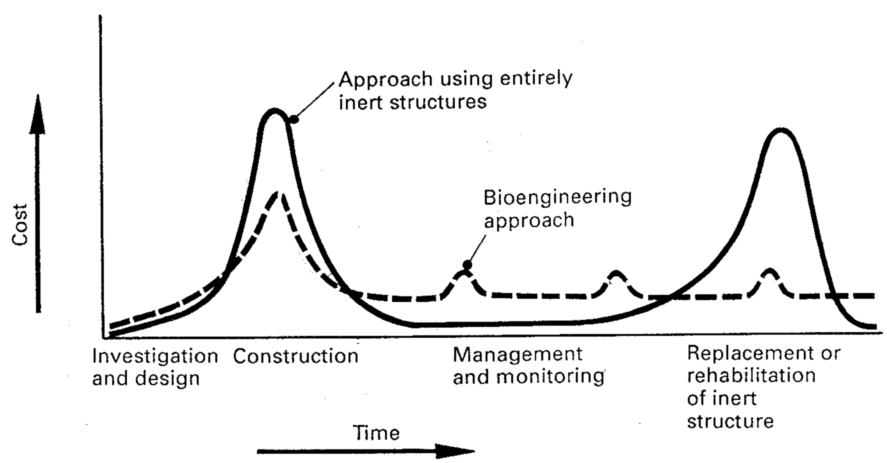


**Figure 43**. Illustrations of different expenditure profiles and maintenance (implied) of inert structures and bioengineering treatments. (from Coppin and Richards, 1990)

Bioengineering projects need to be observed early after project construction for signs of plant survival and development, as well as for streambank integrity. At least qualitative monitoring should be done to assure that detrimental phenomena do not jeopardize the project. For instance, Court Creek, Illinois, one of the project case studies discussed in Volume II, had an infestation of spider mites. Within a month or so after planting, spider mites had damaged almost all of the leaves on the willow that were being used for stabilization. Without remedial spraying, project failure could have resulted. In another case study, North River, Massachusetts (Volume II), a drought occurred the first year after planting and killed much of the planted emergent aquatic vegetation. Remedial planting had to be done the following year to compensate for drought mortality. Also, along with vegetative development, streambank integrity needs to be observed to ensure that unraveling of the bank is not occurring from such actions as undercutting of the toe or flanking at the upper or lower ends of the treated section. If this is occurring, then corrective measures need to be taken immediately, such as placing more rock or some other hard structure in those places. Projects should be monitored at least a couple of years after development at a minimum. Preferably, they should be monitored through 1-2 flood events where currents are directed on the treated bank. One can then assess whether the site remains stable or unravels. In the latter case, remediation can occur. Site monitoring in bioengineering projects should be written into the contract specifications so that early remediation does not become a part of operational and maintenance costs, which often have to be budgeted separately within many agencies.

## Direct Documentation of Erosion Protection

Aerial Photographic Monitoring. Each bioengineering reach and associated treatment, e.g., rock toe with brush matting, vegetative geogrid, should be monitored for erosion directly by use of aerial photogrammetric techniques. This will allow evaluation of changes occuring at the land-water interface providing the procedures discussed below are used.

Aerial photo coverage should be flown at least twice a year for the first 2-3 years or immediately after a flood event. Suggested times are in the spring and in the fall. Low-water periods are preferable. Photo flights should be highly controlled; that is, the scale of repeated flights must be the same. A suggested scale is 1:1,000. Also, three ground control points of known location and dimensions should be used per frame to provide accurate photogrammetric measurements and these should be orthogonally corrected when processed to negate distortion. Recommended film type in priority order is: (1) color infrared and (2) color. To allow comparisons of repeated photo coverage, flights must be made during low water periods and when river water levels correspond to each other; that is, at or below previous photographic periods. Overlays can be made on the photos which will delineate the water-interface boundary. Subsequent overlays can be compared showing any changes in the water-interface boundary (see Figure 44). Photographic measurements can then be made on the overlays to determine amount of surface area lost to erosion.

<u>Ground Photographic Coverage</u>. Monitoring, at a minimum, should be an array of photographs taken from the same photo point in the same directions so that later comparisons of streambank development or degradation can occur very readily. Preferably, this will be used to supplement the aerial photo coverage and measurements mentioned above. Photos should be taken at established photo points with photos taken periodically for a given azimuth. These should be taken at the same time the aerial photos are taken, again at low water periods, if possible; however, others can be taken at intermittent times if deemed necessary.

<u>Ocular Description</u>. As a further effort to document erosion, a description of any erosive processes must be made at the same time the ground photos are made. Processes that must be documented and particularly noted include such things as slumping from geotechnical failures, rilling, gullying, toe undercutting or launching, flanking at upper or lower ends of treatment, and scouring at other areas within the reach from either current or wave action. Descriptive estimates of degree of severity for each of the above processes per treatment and reach with backup photos should be made.

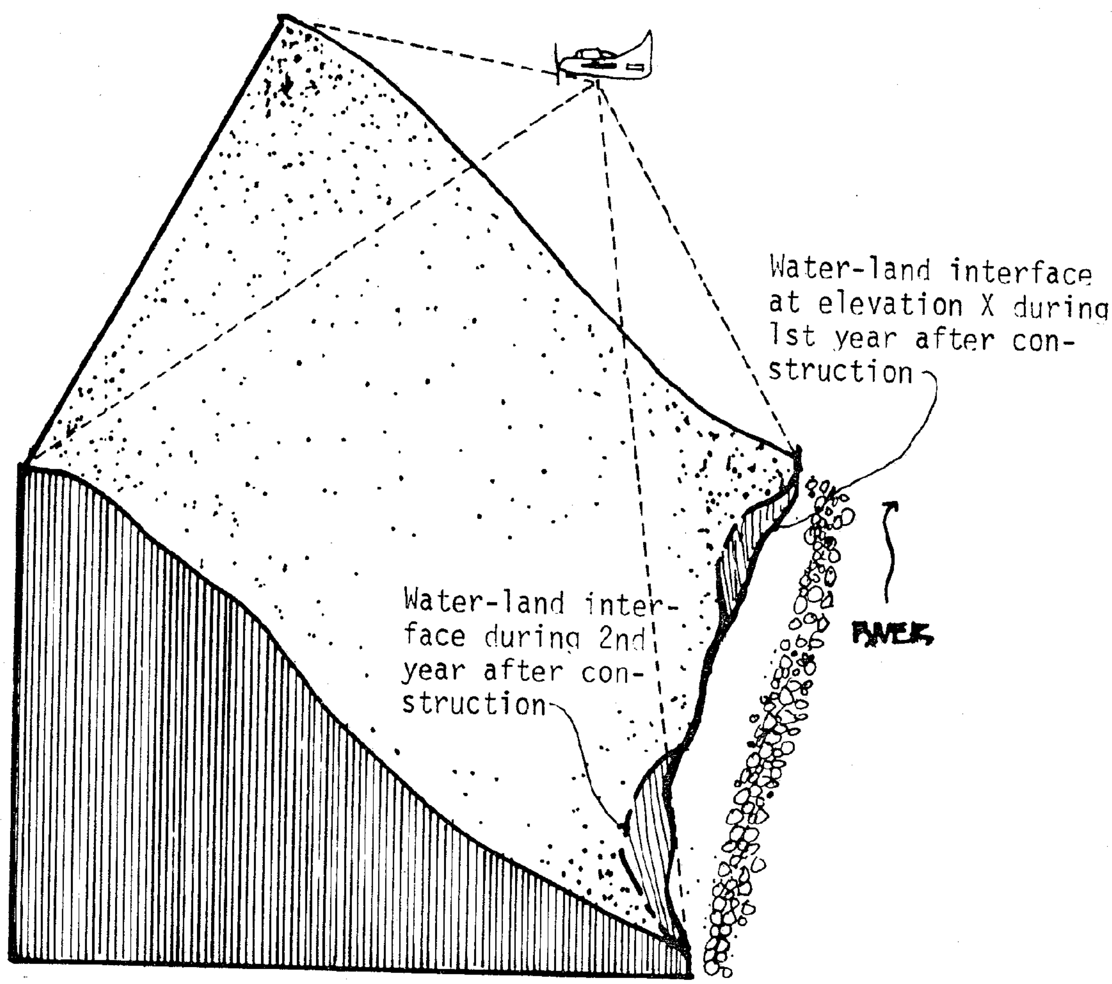


**Figure 44.** Aerial monitoring of bioengineering treatment. (from Logan et al. 1979)

Indirect Documentation of Erosion Protection

Erosion protection is assumed to be offered by the vegetation if the plants are surviving and developing; that is covering the site. The development of the vegetation needs to be monitored and possibly correlated, at least from a visual standpoint to the degree of erosion or lack of erosion taking place on the treated streambank. One would assume, for example, that vegetative plantings are doing a good job if the vegetation is growing well in all elevation zones in the project area and if the stream is not undercutting the treatments, flanking them, or scouring them to the point of failure.

Aftercare

As mentioned above, early monitoring may mean some early remediation and maintenance just to ensure long-term viability. What does this early remediation and maintenance mean? Does this constitute periodic irrigation or repeated fertilizer application? Not as a regular rule. However, plants should be well watered immediately after planting. Bioengineering projects are normally installed at a time of the year, such as early spring, where precipitation is sufficient to allow the planted vegetation to sprout roots and stems and obtain a foothold in their environment. Or, they are installed in the late fall during dormancy. Repeated irrigation is not needed then. Hopefully, fertilizer and other soil treatments were applied before or during planting, if needed, and they should not be required again, unless unusual circumstances prevail.

Possible aftercare requirements may mean bolstering a particular treatment with additional plant or even inert materials after an immediate flood event. Flooding may have caused some plants to wash out before they had a chance to secure themselves with their roots. Hopefully, engineered materials, such as wire, stakes, geotextile coverings, rock toes, etc. would have helped hold the plants and soil until the plants become established, but sometimes any one of these materials, either plants or inert materials, may need bolstering.

Other aftercare measures, as mentioned above, may mean treating plants with an insecticide or fungicide if insects or disease is widely prevalent. Usually, this will be the exception rather than the rule. One can overcome widespread insect or disease damage by emphasizing a wide diversity of plants in the plant mix so that if one species is attacked, the whole vegetative treatment will not be jeopardized. Beaver and herbivores, such as geese, may be a problem in some cases by feeding on woody and emergent aquatic plants, respectively. Beaver will often chew off the upper part of willow and poplar cuttings, but these can resprout and still perform satisfactorily if the complete cutting or stem is not chewed off or dislodged. In some cases, where beaver are known to be in the area, then a trapping program may be advised. Waterfowl, such as geese, like to grub out emergent aquatic plants as well as feed on the upper parts. Temporary fence corridors made out of wooden slats with tiered twine attached to the slats have been shown to prevent geese from feeding on emergent aquatic plants. They do not like to feel trapped inside narrow confines where they cannot escape quickly.

# 5 Costs of Bioengineering

Bioengineering treatments are normally much less expensive than traditional methods of streambank erosion control, e.g., riprapped revetment, bulkheads, but not always depending on the environmental setting and the project objectives. Costs can vary tremendously by availability of materials, hauling distances, prevailing labor rates for the geographic area, and a host of other factors. Table 3 illustrates cost comparisons of actual costs for a couple of bioengineering installations compared to estimated costs of riprapped revetment for the same locality under similar conditions. You will note that the first method, the dormant post method, installed in northwestern Illinois, was about one-fourth the cost of riprapped revetment. The vegetative geogrid installed in California was about 4 times the cost of riprapped revetment. In the first case, riprap was in short supply and cost much more which, in part, contributed to a higher cost than in the California example. Also, the dormant post method required cheaper materials and less labor than the vegetative geogrid in California. Riprap in the California example was fairly cheap and the slope distance to cover the bank was not great, contributing to a cheaper installation than the vegetative geogrid. Also, the vegetative geogrid was fairly labor intensive. Labor accounted for 66 percent of the overall costs. However, what is not shown in the California example is that the site is next to a valuable golf course and the sponsor is also trying to provide shaded riverine aquatic (SRA) habitat for native brown trout. The vegetative geogrid can be installed on nearly a vertical slope without much sacrifice to the adjacent land and it will provide the SRA habitat by providing willow that overhang the banks. The riprapped revetment option does not provide overhanging vegetation for good SRA habitat and does require more land to accommodate shaving the bank to an acceptable construction standard for riprap. It would have required eliminating some of the valuable golf course land. Thus, one must consider the project objectives and potential benefits and impacts when considering comparison of bioengineering methods with other traditional techniques.

When comparing bioengineering methods with traditional engineering applications, Coppin and Richards (1990) stated that each must be considered on its merits, comparing life-cycle costs, i.e. the net present value of investigation, design and construction, plus future management and replacement. As mentioned earlier, bioengineering will require a higher investment early in the project life to ensure that the living system is established. Then, maintenance drops off and the vegetation in the bioengineering treatment continues to grow, spread, and strengthen the streambank through its various attributes mentioned early

**Table 3. Comparisons of actual costs of bioengineering treatments with estimated costs of traditional erosion control (riprapped revetment) under similar conditions in same area.**

| Location & Conditions | Type of Treatment | Costs ($/linear ft) |
|---|---|---|
| **Court Creek, IL** | | |
| 10-ft bank height;<br>3.1 fps local velocity;<br>1V:1H graded side slope | Dormant post & rock toe | $15.19 (actual) |
| 10-ft bank height;<br>1V:2H side slope;<br>1.5 ft total rock thickness,<br>(0.5 ft bedding material);<br>300# stone size;<br>1.5 Ton/ft;<br>$40.00/Ton delivered & placed | Riprapped revetment | $60.00 (est.) |
| **Upper Truckee River, CA** | | |
| 6-ft bank height;<br>4 fps local velocity;<br>stacked soil lifts | Vegetative geogrid | $104.00 (actual) |
| 8-ft bank height (2-ft buried);<br>1V:2H side slope;<br>18 sq ft rock/ft;<br>$20.00/Ton delivered & placed | Riprapped revetment | $27.00 (est.) |

in this report. Some maintenance costs may be associated with the bioengineering treatment later in the project life, but these costs will be rather small. In contrast, the traditional treatment using inert structures, such as riprapped revetment, will have a high construction cost, a finite serviceable life with an element of maintenance, and then a substantial replacement or refurbishment cost (Coppin and Richards, 1990). Figure 43, again, illustrates this cost comparison very nicely (Coppin and Richards, 1990).

Costs are also difficult to compare when strictly looking at currency per unit of measure. The most common denominator for arriving at costs seems to be labor in terms of person hours it takes to build and install the particular treatment. Then, material costs and equipment rental, etc., have to be added onto this. The authors could not document time for all of the bioengineering methods mentioned in the text, but some man-power estimates are given in the following paragraphs. Also, man-power costs are given for general applications of seeding and vegetative plantings to supplement the bioengineering treatments.

## Man-hour Costs of Bioengineering Treatments

### Brush Mattress or Matting

The cost of the brush mattress is moderate according to Schiechtl (1980), requiring 2 to 5 man-hours per square metre. In a training session that WES conducted, a crew of 20 students using hand tools installed about 18 sq m of brush mattress at a rate of about 1 man-hour per square metre. This rate included harvesting the brush, cutting branches into appropriate lengths, and constructing the mattress. This rate of production compares favorably to an average rate of .92 sq m of brush mattress per man-hour by a leading bioengineering firm in the United States.

### Brush Layering

There are few references on the cost of brush layering. Schiechtl (1980) reported the cost to be low, presumably in comparison to techniques using riprap or other similar materials. In the training session mentioned earlier, a crew of 20 students using hand tools installed about 20 m of brush layering along one contour-slope in about 30 min. This equates to 2 m per man-hour. Often, costs can be reduced if machinery such as bulldozers or graders can gain access to the shoreline site and reduce the hand labor required in digging the trenches. Then, this would only require workers to fill the trenches with brush, which can also be covered with machinery.

### Wattling Bundles (Fascines) and Cuttings

Leiser (1983) reported man-hour costs for installing wattling and willow cuttings at Lake Tahoe, California (Table 4). These man-hour costs can be extrapolated to streambanks as well and run about 6 linear ft of wattling per man-hour and 46 small willow cuttings per manhour. Robin Sotir[1] quoted an average installation rate of 5 linear ft of fascine production per man-hour. Obviously, if one were to place a coir fabric between contours of wattling bundles, production rates would decrease substantially. According to Ms Sotir[6], who has done this extensively, it would probably half the amount of linear ft per man-hour.

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[1] Ms. Robin Sotir, President, Robin Sotir and Associates, Marietta, GA, 2 Aug 1996, personal communication.

**Table 4. Man-hour costs of installing wattling and willow cuttings at Lake Tahoe in 1973. (Leiser 1983)**

| Prepare and install wattling (1,140 linear ft) | |
|---|---|
| **Labor** | **Man-hours** |
| C Scaling or cutting back the bank or slope | 2 |
| C Cutting willow whips | 27 |
| C Prepare (stack, tie, load) | 28 |
| C Layout | 9 |
| C Install | 75 |
| C Downtime (rain) | 10 |
| C Travel (from Sacramento, Marysville) | 42 |
| | 193 |
| Unit Man-hour Cost: 1,140/193 = 5.9 linear ft per man-hour | |

| Prepare and plant willow cuttings (8,000 cuttings) | |
|---|---|
| **Labor** | **Man-hours** |
| C Scaling | 2 |
| C Cutting | 9 |
| C Prepare | 34 |
| C Plant | 76 |
| C Downtime (rain) | 10 |
| C Travel (from Sacramento, Marysville) | 42 |
| | 173 |
| Unit Man-hour Cost: 8,000/173 = 46.2 cuttings per man-hour | |

## Dormant Willow Post Method

Roseboom (1995) reported that for bioengineering work on a 600-ft reach at Court Creek, Illinois, it took 5 men two 8-hr days to install 675 willow (12- ft tall) posts on 4-ft centers. This also included installation of a rock toe (20 tons of 10" riprap) with a coir geotextile roll along 300 ft. Also, 60 cedar trees were laid and cabled along the the toe of the slope to trap sediment. This included an excavator operator along with the 4 other men previously mentioned. This equates to about 17 posts per man-hour that includes harvesting and installing the willow posts plus the other operations mentioned above, e.g., shaping site, cedar tree installation.

### Vegetative Geogrid

Man-hour costs for 123 ft of a 6-ft high vegetative geogrid installed on the Upper Truckee River that was previously mentioned included the following:

Three days time of:

1 foreman/equipment operator
1 equipment operator
2 laborers
1 supervisor/project manager

Thus, 120 man-hours were expended on the above project assuming an 8-hr day. This equates to about 1 manhour per linear foot of treated bank. About 66 percent of the costs of this treatment can be attributed to labor.

## Man-hour Costs of Standard Vegetation Establishment Techniques to Supplement Bioengineering Treatments

### Standard Seeding

The cost for broadcast seeding per square metre can vary considerably according to some literature sources. Reported costs in man-hours per square metre vary from 0.004 (Kay, 1978) to 0.07 (Schiechtl, 1980) depending on the degree of slope and the type of seeds used.

### Hydroseeding

Depending on the material used and the distance to adequate water, 4,000 to 20,000 sq m can be hydroseeded by one hydroseeder machine per day (Schiechtl, 1980). A hydroseeder normally uses a two-man crew.

### Hydromulching

Mulching is often applied over seeds by a hydromulcher similar to a hydroseeding machine. For hydromulching or mechanical mulching without seeds, about 0.12 to 0.50 man-hours per square metre is estimated (Schiechtl, 1980). Mulching after seeding increases the cost per square metre considerably. Hydromulching with a slurry of wood fiber, seed, and fertilizer can result in a cost of only 0.008 man-hour per square metre, according to calculations derived from Kay (1978), who reviewed contractor costs in California. The above man-hour calculations assume the following: use of a four-man mulching machine, seed and fertilizer applied at a rate of 0.75 ton per acre, and an application rate of 2 tons per hour.

## Sprigs, Rootstocks or Plugs, Rhizomes, and Tubers

Costs for digging grasses and other herbaceous plants in their native habitat and transplanting propagules of these will vary depending on the harvesting system used, the placement of the plants, and the site. For digging, storing and handling, and planting 1,000 plants of sprigged wetland grasses and sedges, Knutson and Inskeep (1982) reported a rate of about 10 man-hours. Sprigs of this type were placed on 0.5-m centers, which would cover 250 sq m. For the same kinds of plants, Allen, Webb, and Shirley (1984) reported a rate equivalent to 400 plants per 10 man-hours for digging, handling, and planting single sprigs. According to Knutson and Inskeep (1982), using plugs of any species (grass or forb) is at least three times more time-consuming than using sprigs (30 man-hours per 1,000 plugs).

## Bare-root Tree or Shrub Seedlings

Depending on type of plant and local conditions, the reported costs of planting vary considerably. On good sites with deep soils and gentle slopes, the authors have experienced planting up to between 100 and 125 plants per man-hour. Logan et al. (1979), however, estimated that only 200 to 400 plants per day per person could be achieved on sites like the banks of the upper Missouri River.

## Ball and Burlap Trees or Shrubs

Planting costs for this type of transplant will range from 10 to 25 plants per man-hour (Schiechtl, 1980).

## Containerized Plantings

The cost of plantings varies depending on plant species, pot type, and site conditions. By using pots other than paper, 20 to 40 plants per man-hour can be planted. With paper pots, up to 100 plants per man-hour can be planted (Schiechtl, 1980). Logan et al. (1979) stated that the cost for hand-planting containerized stock ranges from one-half the cost for bare-root seedlings to a cost equal to or exceeding that of the container seedling.

# 6 SUMMARY AND RECOMMENDATIONS

Bioengineering can be a useful tool in controlling bank erosion, but should not be considered a panacea. It needs to be performed in a prudent manner and in consonance with good planform and channel bed stability design. It must be done with the landscape and watershed in mind, particularly with respect to erosion that has occurred as a result of both broad basinwide activities and local, site-specific causes. Nevertheless, bioengineering must be done at the reach level. This must be done in a systematic way with thought given to its effects both upstream and downstream and it may have to be done incrementally to overcome seasonal time constraints. For instance, woody plants must be planted in the dormant season. There are numerous questions that must be answered prior to bioengineering implementation. Answering these questions and designing a project must be an integrated process that starts with the planning phase and continues through the construction phase. There are obviously feedback loops from the design and construction phases back to the planning phase. Additional information may have to be retrieved that calls for more planning actions.

Bioengineering must be accomplished with enough hardness to prevent both undercutting of the streambank toe and erosion of the upper and lower ends (flanking) of the treated reach. This can be done with one or both of: (1) hard toe and flanking protection, e.g., rock riprap, refusals, and (2) deflection of water away from the target reach to be protected through deflection structures, e.g., groins, hard points, vanes, and dikes. With both of these methods, only appropriate plant species should be used in a manner consistent with their natural habitats. This is often done by using streambank zones that correspond with micro-habitats of native plant species in local stream environments. Where possible, both herbaceous and woody species are used with grass or grass-like plants, e.g., sedges, rushes, reed grasses, in the lower-most zone, then shrubby, woody vegetation in the middle zone, and for the most part, larger shrubs and trees in the upper-most zone. These zones are respectively called the "splash, bank, and terrace zones."

Careful planning must be done to acquire the kinds of plants in the amounts needed. This may take up to one year before installation of the various treatments because plants either have to be grown in sufficient quantities in nurseries or they have to be located in the wild and either collected or grown from wild plant stock.

Bioengineering treatments have been noted, depending on the type of treatment, to resist up to 12 fps local flow velocities. It is recognized, however, that local flow velocities during peak discharges are difficult to obtain during those events because of safety considerations.

Log revetments with geotextile rolls in Colorado sustained velocities up to 10 fps, but undermining the lower logs occurred in the lower part of the treated reach. A general rule of thumb is that for velocities exceeding 8 fps, some combination of inert material be used with plants that are well secured and have adequate toe and flank protection. The inert material may be deflection structures made from root wads or rock hard points or dikes, etc., or the inert material may be wire and stakes that hold down plant material long enough for that material to take hold. Even then, those materials, both inert and living plants, must have enough toe and flank protection to allow sustainment through flood events. This sustainment is especially critical during the early phases of the project.

Early monitoring and aftercare of a bioengineering project is essential. Each project should have incorporated into it from the beginning enough time and funds to provide some remedial work within the first year or so after treatment installation. It would be better to provide this contingency for up to and immediately after the first one or two flood events. Once weak spots in treatments are repaired, the bioengineered system continues to gain strength over time.

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