STREAMBANK STABILIZATION MEASURES FOR HIGHWAY ENGINEERS

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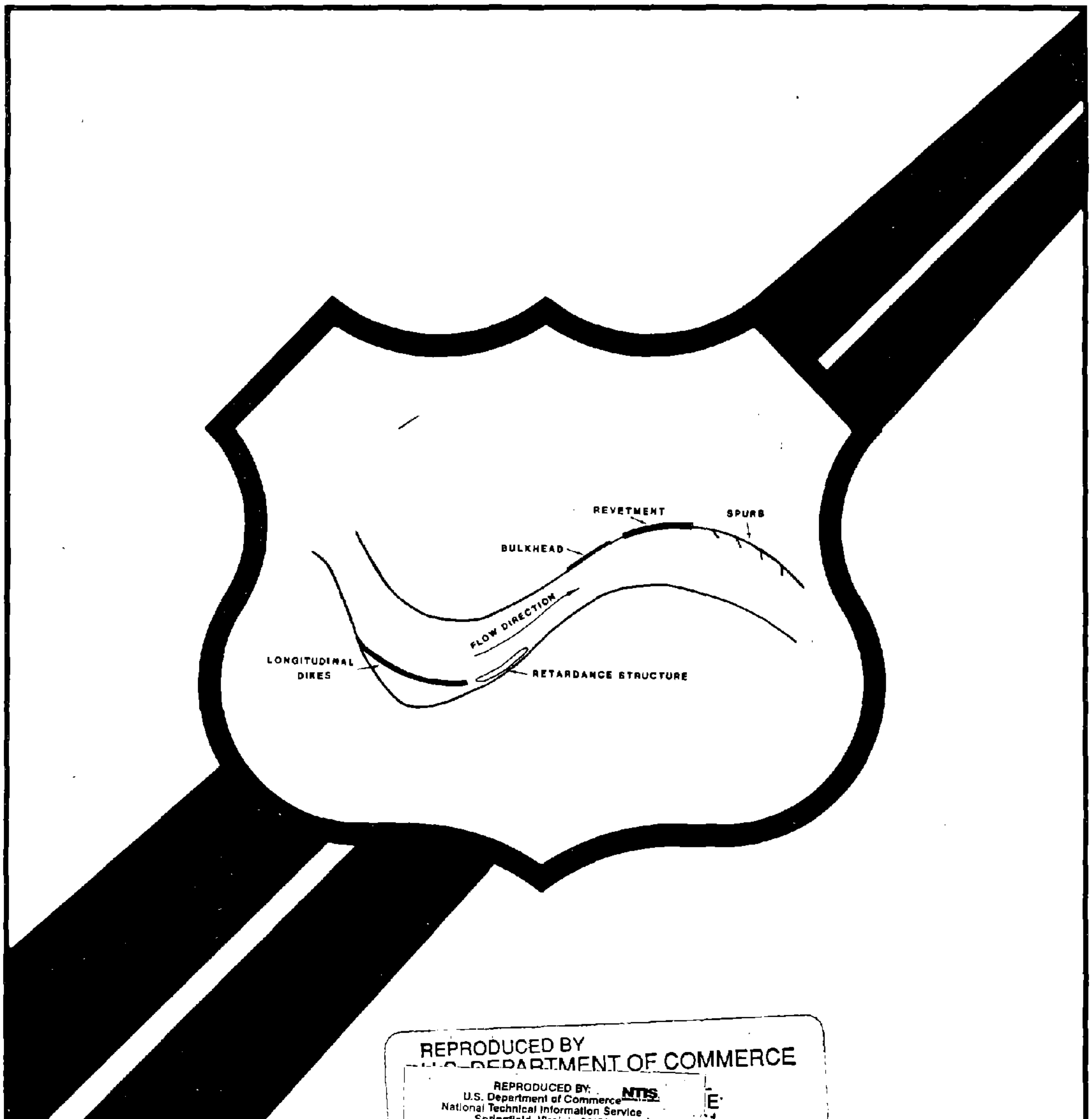

FOREWORD

Based on a thorough literature search, extensive field evaluations, and numerous personal contacts with design engineers, this report presents a study of erosion processes in channel bends and methods of controlling erosion in bends. Guidelines for the selection of a countermeasure type for specific site conditions are also presented.

Research and development in streambank stabilization is included in the Federally Coordinated Program of Highway Research, Development, and Technology Project 5H "Highway Drainage and Flood Protection." Dr. Roy E. Trent is the Project Manager and the Contracting Officer's Technical Representative for this study.

Sufficient copies of this report are being distributed to provide a minimum of two copies to each FHWA regional office, one copy to each division office, and two copies to each State highway agency. Direct distribution is being made to the division offices.

Richard E. Hay

Richard E. Hay, Director
Office of Engineering
and Highway Operations
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NOTICE

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16. Abstract

A study of erosion processes in channel bends and methods of controlling channelbank erosion in bends is reported on. Flow and erosion processes in channel bends are categorized and discussed to aid in identifying the erosion mechanisms and processes active at a particular site. Knowledge of the active erosion processes is important to the selection of an appropriate streambank stabilization measure, as well as to considerations in the final design of the structure. Flow control and streambank stabilization structures are also identified and classified. The categories of streambank stabilization structures reported on include spurs, revetments, retardance structures, longitudinal dikes, and bulkheads. Numerous individual countermeasure types are identified within each of these countermeasure groups. The applicability of countermeasure groups and individual countermeasure types are reviewed in terms of the countermeasures function or purpose, erosion mechanisms countered, river characteristics, geomorphic and other impacts of the erosion control system, vandalism and maintenance considerations, construction related considerations, and costs. This review provides guidelines for the selection of an individual countermeasure for a particular site. Some design information for individual countermeasure types is also included. This report is based on a thorough literature search, extensive review and evaluation of field installations, and numerous personal contacts with design engineers.

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METRIC CONVERSION FACTORS

Multiple	By	To Obtain
inches	2.5	centimeters
feet	30	centimeters
yards	0.9	meters
miles	1.6	kilometers
square inches	6.5	square centimeters
square feet	0.09	square meters
square yards	0.8	square meters
square miles	2.6	square kilometers
acres	0.4	hectares
ounces	28	grams
pounds	0.45	kilograms
short tons (2000 lbs)	0.9	tonnes

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Chapter 1

INTRODUCTION

This report provides guidelines for the selection and design of flow control and streambank-stabilization structures. It is intended to alert engineers to the advantages, disadvantages, effectiveness, and limitations of the more common types of flow-control and streambank-stabilization structures.

The first consideration in this report is a discussion of flow and erosion processes in channelbends. This presentation is intended to aid the designer in identifying the erosion mechanisms and processes active at a particular site. Knowledge of the active erosion processes will aid in the selection of an appropriate countermeasure for a particular site. Also included are discussions of the morphologic effects of channelbank stabilization, and methods for controlling various types of bank erosion.

Next, types of flow control and streambank stabilization countermeasures are identified, and criteria for the evaluation and selection of a specific countermeasure type are presented. Countermeasure types identified include revetments, retardance structures, longitudinal dikes, spurs, and bulkheads. The application of each of these countermeasure types is then considered. This discussion is intended to provide a basis for comparing the attributes of the most common flow- and erosion- control countermeasures to aid in the selection of appropriate countermeasures for a specific site. Numerous individual countermeasure types are identified within each of the countermeasure groups identified above, and advantages and disadvantages to their use under various environmental conditions are discussed.

This report is based on a thorough literature review, extensive review and evaluation of field installation, and numerous personal contacts with design engineers actively involved in designing flow-control structures.

Chapter 2

FLOW AND EROSION PROCESSES IN CHANNELBENDS

The selection and design of flow control and/or streambank stabilization structures requires a thorough understanding of the flow and erosion processes that cause streambank instabilities and general bank erosion. Streambank erosion can result from a variety of processes that can act individually or in combination to cause bank failure. Processes responsible for streambank erosion can be interpreted either in geomorphic terms or in terms of the mechanisms and forces involved. The proper interpretation of a channel instability or erosion problem requires an evaluation from both perspectives to provide an understanding of the mechanisms and contributing factors involved. In the following sections, both approaches are considered.

GEOMORHPIC EROSION PROCESSES

A river system's hydraulic geometry (i.e., its width, depth,and planview form) is a function of the external constraints applied to the particular system. These external constraints include water discharge, sediment discharge, valley slope, and those constraints imposed by the region. During the design life of a typical engineering project, the valley slope and geologic constraints can be assumed to be constant; the water discharge and the sediment discharge cannot. In fact, the water and sediment discharges will vary with every flow event. Since the hydraulic geometry of a channel is a function of these dynamic elements, a river system will attempt to adjust its geometry in response to these changing conditions to maintain or create a condition of dynamic equilibrium with respect to its own water and sediment load and channel makeup. The geomorphic approach then, looks at channelbank erosion as a system's natural mechanism of maintaining its own balance or equilibrium. The following sections will consider how the flow of water and sediments in alluvial channels affect channel width, depth, and sinuosity.

Functional Relationships

Geomorphic proportionalities that describe functional relationships between a channel's water and sediment load and the resulting channel size, shape, and sinuosity have been presented by numerous authors. Notable among these are Leopold et al. (1964), Lane (1955), Schumm (1977), and Simons and Senturk (1976). Most recently, a review of these relationships was presented by DeCoursey (1981). To demonstrate the effect of changes in flow and sediment load on channel morphology, the geomorphic relationships can be summarized as follows:

$$W \sim Q\,Q_s \tag{1}$$

$$w/d \sim Q_s \tag{2}$$

$$d \sim Q \tag{3}$$

$$S_c/D_{50} \sim Q_s/Q \tag{4}$$

$$P \sim S_v/Q_s \tag{5}$$

where

W	=	stream width
Q	=	water discharge
Q_s	=	sediment discharge
d	=	stream depth
S_c	=	channel slope
D_{50}	=	mean sediment size
P	=	sinuosity
S_v	=	valley slope

The above equations are simplified approximations of complete power relationships. However, in their simplified form, they can be used to look qualitatively at changes that can be expected to develop in response to fluctuations in water and sediment load.

Since water and sediment discharges are rarely constant, Equations 1 through 5 indicate that channels are constantly trying to adjust their width, depth, and planview form. This is true from a morphologic point of view. From a practical engineering standpoint, however, a quasi-equilibrium channel geometry can be defined based on dominant sediment and water discharge conditions. The dominant channel form is that which is evident from aerial photography and maps. The stability of this quasi-equilibrium channel form is of primary concern to the engineer designing structures in the vicinity of a river channel.

As mentioned above, the quasi-equilibrium channel form (that is, its width, depth, and planview geometry) is a function of dominant sediment and water discharge conditions. The notation of flow frequency plays an important role in defining these dominant conditions. It has been suggested that these dominant conditions be defined as the discharge conditions equaled or exceeded an 0.6 percent of the days of record (or 1 day out of 170) (Henderson, 1966). Shifts in these dominant conditions (i.e., changes in the frequency distributions), then, will threaten the stability of a given channel reach in accordance with Equations 1 through 5 (using dominant values of Q_s and Q as the variable).

To provide a better understanding of the geomorphic proportionalities presented in Equations 1 through 5, the following section will look at the geomorphic processes described in the equations, consider some of the more common causes of morphologic imbalance, and explain typical system responses to these events.

Geomorphic Response

There are three geomorphic responses or processes that can result from changes in dominant channel flow and sediment conditions. They are channel widening, channel deepening, and changing planview form (a change in sinuosity or meander pattern). All of these responses will result in some level of streambank erosion.

Channel widening is evidenced through an increase in channel width, with or without an increase in channel depth. Consideration of Equation 1 indicates that an increase in flow or sediment discharge results in a tendency towards channel widening. However, when both sediment discharge and flow increase, the channel section can be expected to increase its depth as well as its width (see Equations 2 and 3). When only sediment load increases, width increases but the depth may decrease. In the case of sediment load increase only, the channel is said to be aggrading, implying that the channel has aggraded or filled in because of an excess of sediments.

Channel deepening is a process of channel degradation that increases the depth of the channel. Channel degradation can cause bank instability by producing a steeper bank angle. Whether or not instability actually occurs is a function of the properties of the bank materials and the original bank geometry. Channel deepening results from increased flow without an appreciable increase in sediment discharge (Equation 3). Increased flow rates can result from an overall increase in the volume of water moving through the channel or an increase in channel slope.

Changing planview form includes changes in channel shape and position as viewed from above. Changes in planview form are most often exhibited through the downstream migration of meandering bends and changes in the sinuosity of meander bends. Other examples include the shifting of channels and the cutting off of meander bends. Generally, these changes are manifested by an adjustment of channel slope to conform with changes in flow or sediment discharge. These changes can be illustrated through an evaluation of Equations 4 and 5.

Equation 4 indicates that either a reduction in sediment discharge or an increase in water discharge will result in a reduction of the channel slope. These slope reductions result in increased channel sinuosity and/or channelbed degradation; both of which lead to increased bank erosion tendencies. Also, Equation 5 indicates that a reduction in sediment discharge will result in an increase in channel sinuosity; again, leading to increased bank-erosion tendencies.

It is important to recognize that the three geomorphic processes just discussed (channel widening, channel deepening, and changing planview form) are often interrelated and can occur simultaneously or in sequence. For example, adjustments in channel slope through degradation often are accompanied by increases in channel sinuosity and bank caving or channel widening. Also, the initiation of a given process at a particular site may initiate another process either upstream or downstream. For example, an aggrading channel reach can cause an increase in sinuosity in a downstream reach.

The shifts in dominant flow conditions discussed previously can result from either natural or human-induced causes. The resulting erosion process can be defined as natural, or accelerated, erosion.

Natural Erosion

Natural erosion results from natural occurrences such as normal fluctuations in hydraulic conditions, extended drought, or rainy periods, as well as single, excessive storm events. All of these events can cause short-term shifts in the magnitude of the dominant flow conditions, resulting in the adjustments in channel form previously described. For example, extended periods of high flow will cause a temporary shift in dominant discharge levels and possibly a corresponding upward shift in dominant sediment load conditions as well. Previous discussions indicated that these changes result in tendencies towards increased channel widths and depths, as well as a reduction in channel sinuosity. The reduced sinuosity results in a trend to shift meander bends downstream. This tendency will be discussed in subsequent sections of this report. Each of these responses will increase bank-erosion tendencies. These responses also will be true of single storm events.

Conversely, we could consider extended drought periods and the corresponding reductions in flow and sediment transport rates. Equations 1 through 5 indicate that under these conditions, one could expect reductions in channel width and depth and an increase in sinuosity. Because of the reduced flow conditions, these responses occur within the confines of the dominant channelbanks, and thus, do not pose any significant erosion hazards.

Channel modifications resulting from natural erosion processes include the gradual downstream migration of channelbends and channel avulsions, such as the development of meander cutoffs. When meander cutoffs occur, they can result in extensive reshaping of upstream channel networks. The sudden increase in channel slope that results when a cutoff occurs will result in upstream channel degradation and a tendency towards increased meander activity; both of which will affect channelbank stability. Natural erosion processes often are difficult to anticipate since they are so dependent on hydrologic events. A seemingly stable river system could suddenly become unstable as a result of a prolonged period of high flow or a single excessive storm event. The uncertain nature of hydrologic events makes it almost impossible to anticipate such occurrences.

Accelerated Erosion

Accelerated erosion results from some human activity within the watershed that influences flow and sediment transport rates. Human activities that influence morphologic erosion processes include agricultural activities, urbanization, construction activities, streambed sand and gravel mining, interbasin water transfers, and reservoir development and operation. Human activities are the more common cause of channel instabilities, and in general, are more widespread and greater in magnitude than natural erosion. Because accelerated erosion is associated with human activities, it often is possible to anticipate any impact on bank stability and provide adequate bank protection in advance. The following discussions will look at each of the activities mentioned above and discuss the ways that they affect channel morphology.

Agriculture-related activities include cultivating and harvesting crops, and grazing cattle and other animals. Deforestation and related activities also are included as agricultural activities. The general tendency in agriculture is towards increased peak flows and increased sediment yield. The result will be towards an increase in channel width and a reduction in overall channel sinuosity. Additionally, the grazing of animals along streambanks reduces the vegetative cover, and the continual migration of animals up and down the streambanks can have a significant impact on bank stability.

Stream-channel straightening is another activity that has been associated with agricultural activity in the past. In the early 1900's, channel straightening was a common practice in the central and southern agricultural states to make available additional farmlands along the meandering channels of the region. These activities greatly increased the channel slopes of the modified channels. Currently, the geomorphic response in these regions is extensive channelbed degradation and accelerated meander activities. Both of these responses are a result of the channels' attempts to readjust to their previous slopes.

Urbanization normally causes significant increases in the magnitude of runoff events while reducing the duration of the runoff event. Fully developed urban areas also are low-sediment-producing areas because of the large percentage of land that is protected by impervious surfaces. As a result, urbanization reduces the sediment inflow to a river. The combination of the increased peak runoff rates and the reduced sediment loads will result in channel degradation, channel widening, and a reduction in channel sinuosity. Each of these activities will contribute to increased meander activity.

Construction activities are known to produce increased discharge and sediment load magnitudes. The increased runoff or discharge results from clearing and grubbing activities that strip away the vegetative cover that normally acts as a flow retardant. Removal of the vegetative cover (as well as grading and other construction activities) bares and disturbs the soil, accelerating the erosion process and increasing sediment yields to tributary streams. The system's response to the increased discharge is to increase its width and reduce its meander radius. The response to the increased sediment load is an aggrading or building of the channel base level, which, when combined with the increased discharge level, will result in accelerated bank-erosion tendencies. However, since construction activities usually are temporary, these system responses will be short-lived.

Streambed Mining is another activity that upsets the natural balance in a river environment. Sand and gravel mining activities affect the sediment movement and supply in a channel system. Excess mining produces a steeper energy slope in the vicinity of the operation, as well as a reduction in sediment load downstream from the operation. Both of these activities increase the energy available in the water discharge downstream from the mining operation, which increases the potential for bank erosion.

Interbasin transfers of flow are becoming more and more common as the demands on our water resources increase. Diverting flow from one basin to another will increase both the magnitude and duration of flows in the receiving channel. Here again, the channel will respond by attempting to increase its dominant width and depth and reducing its sinuosity. These responses will result in a period of channel instability and bank erosion until the new channel regime is established.

Reservoir development and operation for storage and flood control also has an impact on downstream streambank stability. Reservoirs trap the incoming sediment load and release clear-water discharges. The clear water released has a higher energy level, since it is not carrying sediment. In an attempt to reduce the energy level, the flow stream will attack the channelbed and banks, producing both degradation and lateral instability. Besides trapping the sediment load, reservoir regulation also changes the downstream flow characteristics. To satisfy power generation, irrigation, or navigation requirements, reservoir regulation policies produce higher sustained downstream discharges than were characteristic prior to regulation. The increased duration of these higher discharges will again produce bank erosion tendencies. Reservoir operation, particularly for hydropower generation, produces sudden stage fluctuations, which result in saturation and draining of downstream channelbanks. Bank saturation and drainage is an important factor influencing both the magnitude and rate of bank erosion; this will be discussed in a later section.

DYNAMICS OF BANK EROSION

Streambank erosion is a consequence of the interaction of a channel boundary and the flowing water. For bank erosion to occur, both a soil displacement mechanism and a transporting mechanism must be present. Soil displacement mechanisms include streamflow, surface weathering, abrasion, subsurface flow, wave erosion, and chemical action. The transporting mechanism is provided by the flowing water. As long as the sediment-carrying capacity of the flowing water has not been exceeded, any material dislodged from the bank will be carried downstream by the streamflow. The sediment-transporting capacity of a given flow volume is a function of the streamflow velocity as well as the sediment particle size. If the sediment-transporting capacity of a given streamflow has been exceeded, the dislodged material will accumulate at the base of the bank, thus providing a stabilizing influence. The following sections contain detailed descriptions of soil displacement mechanisms and streamflow dynamics, which influence bank erosion.

Soil Displacement Mechanisms

As mentioned above, soil particle displacement mechanisms include streamflow, surface weathering, abrasion, subsurface flow, wave erosion, and chemical action.

Streamflow is the most prevalent soil-particle displacement mechanism, particularly where channelbanks are composed of noncohesive materials. Particle displacement by streamflow is driven by sheer forces and flow turbulence at the interface between the flowing water and the bank surface. The magnitude of the forces acting on the channelbanks are proportional to the flow velocity, the velocity distribution in the vicinity of the bank, and the bank's surface roughness. It has been shown that these forces are most severe near the toe of the bank (Prasad and Alonson, 1976). Theoretical considerations of the force's acting on individual bank particles can be found in most sediment transport texts. For example, see Simons and Senturk (1976) or Graf (1971).

Surface Weathering processes are most often directly associated with soil-moisture conditions that act on the bank surface to loosen and detach particles or aggregates. Surface weathering can be caused by a number of factors. Frequently, the driving rain associated with high-intensity storms will loosen bank particles that will then be removed by runoff from the rain as it flows over and down the face of the bank. Wet and dry cycles also can loosen bank materials by reducing the granular interlocking and destroying interparticle cohesion. Freezing of water in the pores of the surface material can heave soil particles apart and loosen them in a similar fashion. This loose surface material is then easily eroded by flowing water as the stream rises; it also can be removed by the force of gravity. Also, the rapid rise of water in a channel can cause material to flake or slough off the bank surface if it is fine-textured or dry. Although these discussions indicate that surface weathering could be a frequent problem, it usually is much less significant than other forms of bank erosion. Also, the rate of surface erosion is greatly reduced by the presence of streambank vegetation.

Abrasion occurs when solid materials carried by the flowing water, such as debris and ice, collide with and dislodge surface soil particles. Bank erosion from abrasion can be a problem in northern climates where ice buildups along the bank are common and also in areas where channel degradation undermines trees along the channelbanks, creating significant debris loads. Abrasive forces along the banks also can damage existing bank vegetation, resulting in a weakening of the bank structure, making it more susceptible to other erosion mechanisms.

Subsurface Flows or seepage are produced by flow through the bank materials; either towards or away from the river. In poorly drained banks, positive pore water pressure can weaken the bank by reducing its effective strength. The most critical condition occurs during heavy or prolonged precipitation, snowmelt, or rapid drawdown following a high-flow stage. Even if no significant pore pressures are exerted, the stability of the bank will be reduced by saturation because of an increase in the unit weight of the material and a decrease in the internal strength. Cycles of wetting and drying also are extremely important because they cause swelling and shrinkage of the soil. The seepage forces resulting from subsurface flow can dislodge bank material on a particle-by-particle basis, such as in flow piping; or in mass, through shear failures. Subsurface seepage can also result in a general weakening of the bank structure, making it more susceptible to other erosion mechanisms. Additional information on bank-failure modes resulting from subsurface flow conditions will be presented in later sections. Other information on bank-failure modes and methods of predicting bank failure from subsurface moisture conditions can be found in Thorne et al. (1981).

Waves caused by wind or vessel traffic cause surface deterioration of the bank near the stream surface as a consequence of the energy dissipated as the waves break along the bankline. Wave erosion is a particular problem in wide channels that remain at one elevation for sustained periods of time; for example, during flood flows or when regulation causes periods of sustained flow. The wave erosion mechanism is similar to that along ocean or lake beaches, slowly eroding the bank until a bench develops that is wide enough to dissipate the wave action before it reaches the bank. Theoretical consideration of wave erosion is presented in California Department of Public Works (1970).

Chemical Action affects the stability of banks composed predominantly of cohesive materials. Water and acid in water affect cohesive and other types of particle-to-particle bonding. In these cases, bank material is removed by dissolution. The influence of chemical action in the erosion of cohesive materials makes this process extremely variable in space and time. Thus, the erosion of cohesive materials is the least understood and the most difficult to quantify. Erosion of cohesive materials is described by Partheniades (1971).

The above discussions have pointed out the most common causes of soil particle displacement. Although the displacement mechanisms have been looked at individually, soil displacement is most often the result of the combined effect of several of these mechanisms. One or two mechanisms, however, can be isolated as the controlling factors at a particular site.

The flowing water provides the transporting mechanism, as well as the primary dislodging mechanism. In fact, in almost all cases the flowing water will be at least partially responsible for bank-particle dislodging after one of the other mechanisms weakens the bank structure. Because of its importance to bank-erosion processes, streamflow dynamics relating to bank erosion will be discussed.

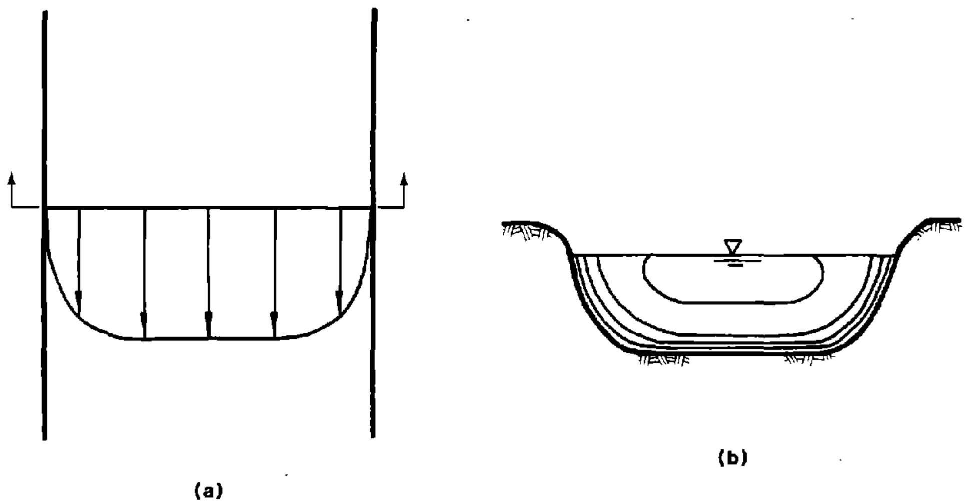


FIGURE 1. VELOCITY DISTRIBUTION IN A UNIFORM, STRAIGHT CHANNEL: (A) PLANVIEW, (B) CROSS SECTION

Streamflow Dynamics Influencing Bank Erosion

The fact that particle displacement and transport is driven by shear forces and turbulence at the channel boundary was pointed out previously. It also has been mentioned that the magnitude of these forces is proportional to the steepness of the velocity distribution alongside the channelbank, as well as the absolute magnitude of the velocity.

A channel boundary's resistance to flow sets up distinct velocity distribution patterns and resulting shear forces along the channel boundary. Figure 1 illustrates a typical velocity distribution in a straight reach. The steepness of the velocity distribution in Figure 1 is proportional to the shear force exerted by the water. This also is represented by the spacing of the equal velocity lines in Figure 1B; the closer the lines of equal velocity, the greater the shear force.

While bank erosion does occur in straight river reaches, most erosion and flow-control problems occur at channelbends. This is primarily because flow forces in bends are more severe than those in straight reaches. Another reason is that there are very few naturally straight river reaches. For these reasons, the following discussion centers on the dynamics of flow in channelbends.

Figure 2 illustrates typical flow patterns for a steady flow through a sinuous river reach. Isometric views of the changing flow distributions as they approach and pass through the channelbend are included. Notice that as the flow approaches the bend axis (at Cross Section 3) the velocity distribution shifts so that the major flow current is along the outside

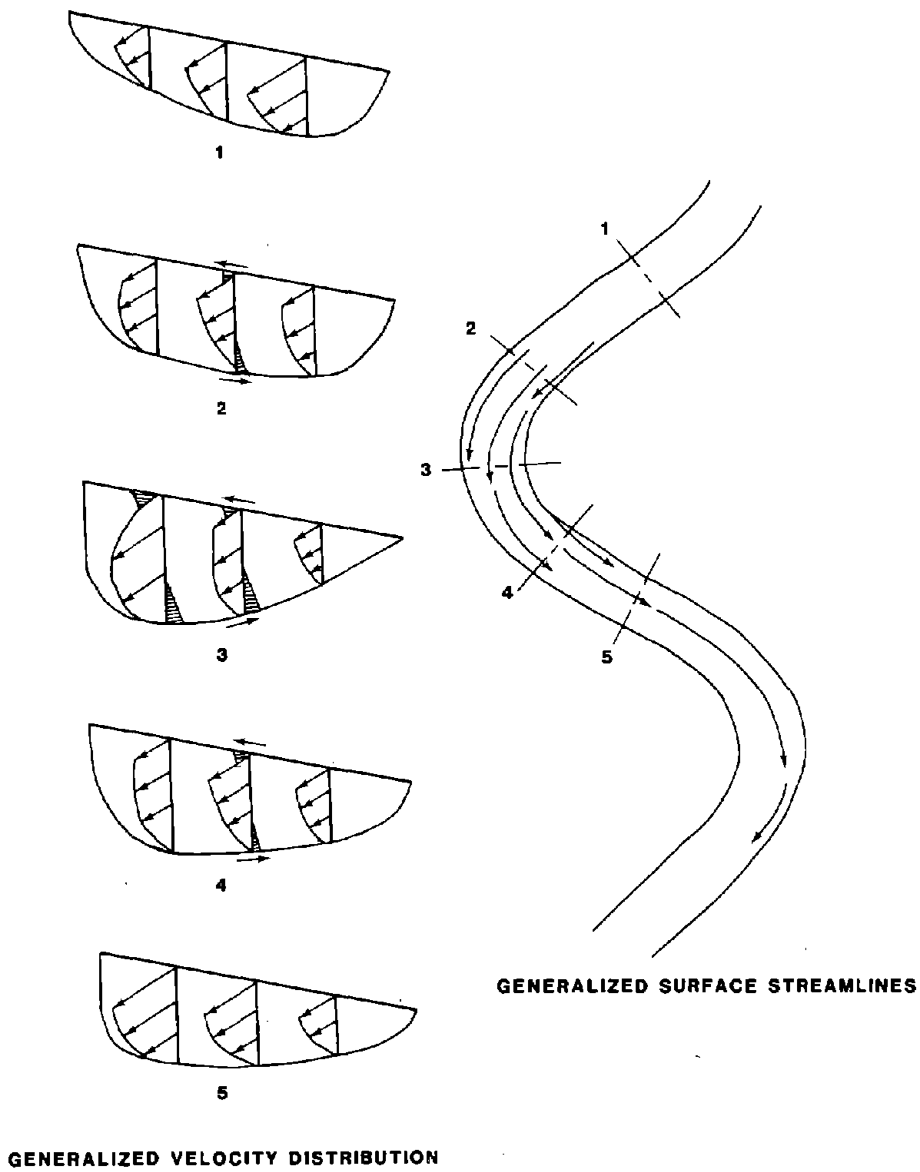


FIGURE 2. GENERALIZED DIAGRAM OF FLOW DISTRIBUTION IN A MEANDER (MODIFIED FROM LEOPOLD, WOLMAN, AND MILLER, 1964)

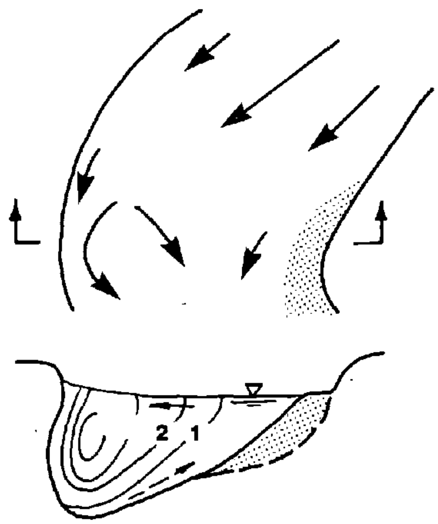


FIGURE 3. VELOCITY DISTRIBUTION IN CURVILINEAR FLOW
(MODIFIED FROM CALIFORNIA DEPT. OF PUBLIC WORKS, 1970)

channelbank. Also notice that the maximum velocity thread shifts from a point near the water surface at Cross Section 1 to a point located approximately at mid-depth at Cross Section 3.

These conditions have been verified by field observations indicting that the highest velocities in a bend occur just downstream of the bend axis along the concave bank at about mid-depth. Figure 3 shows a typical velocity distribution for a channelbend section. This figure also indicates that the primary current thread is located along the bank below the water surface. Comparison of Figure 3 with Figure 1 also documents the difference between typical velocity distributions in a bend and in a straight channel reach. Notice again that the high velocity core has shifted to a point below the surface on the outside of the bend. Since the lines of equal velocity are closest (adjacent to the outside bank and in the vicinity of the bank toe), this is the area most susceptible to erosion.

Secondary currents, another component of flow in channelbends, also must be considered. Primarily, secondary currents are thought to be a result of flows along the outside of a typical bend (see Figure 3).The resulting cross-channel slope of the water-surface causes a transverse component of flow near the bed from the outer bank to the inner bank and near the surface from the inner bank to the outer bank (see Figure 3). These transverse currents, superimposed on the longitudinal flow, form a screw-like, helicoil secondary circulation in river bends. The transverse component of this flow accelerates the erosion along the outer bank.

The magnitude of these secondary currents or cross-channel velocity components depends on longitudinal velocity, bend radius, cross-sectional shape, and channel width. For additional information, see Simons (1977), Bathurst et al. (1979), and Einstein (1971).

Change in flow pattern with increasing or decreasing stage is another important consideration in meanders related to flow control and bank stabilization. There is considerable change in the main direction of flow around bends, with fluctuations in flow stage. Under low-flow conditions, the flow streamlines are generally to a sinuous low-water channel (refer to Figure 4). If bank erosion occurs under these conditions, and it usually will not, it is confined to the concave bank just upstream of the bend axis. Flow streamlines at high river stages, however, are no longer confined to this low-water path; instead, they are free to cut across and erode the point bar sediments along the convex bank. At these high stages erosion also will occur along the concave bank; however, the point of maximum erosion shifts downstream to a point almost opposite the crossing. During lower but still high stages, erosion will continue along the concave bank with the critical erosion zone shifting upstream as the stage is lowered. This discussion indicates that as river-stage increases and then decreases during a flow event, the point at which the maximum current approaches the concave bank first shifts progressively downstream, shifting from a point just downstream of the bend axis to a joint just upstream of the crossing, and then shifts progressively back upstream as the stage lowers.

The location of the thread of maximum current also depends on whether the runoff is occurring on the rising or falling limb of the runoff hydrograph. The thread of maximum current approaches the concave streambank more closely during falling river stages than it does during rising stages for the same discharge magnitude. This translates to a tendency for more significant lateral erosion on the falling limb than on the rising limb. This occurs because the river attempts to adjust to a higher energy slope during the peak and rising limb of the flood hydrograph than on the falling limb. The increased slope takes the form of a reduction in the amplitude of swing of the main current; this increases the meander length, pushing the critical point of attack downstream. On the falling limb however, the energy slope undergoes a reduction in the form or an increase in the main current's amplitude of swing. This action brings the thread of maximum current closer to the concave bank through the bend, and shifts the most critical point of attack upstream.

The above discussions indicate that the flow patterns that must be considered when designing streambank stabilization schemes are quite complex. While this information is useful, it also is important to be able to estimate the magnitude of the velocities and shear forces that can be expected in a channelbend for determining the size and/or strength of a protection technique as well as the extent required for protection. An analysis of the distribution of these forces for large projects can be made through the use of physical and mathematical model studies. In most cases, however, time and economics will not allow for model studies, and simpler, less expensive methods must be employed.

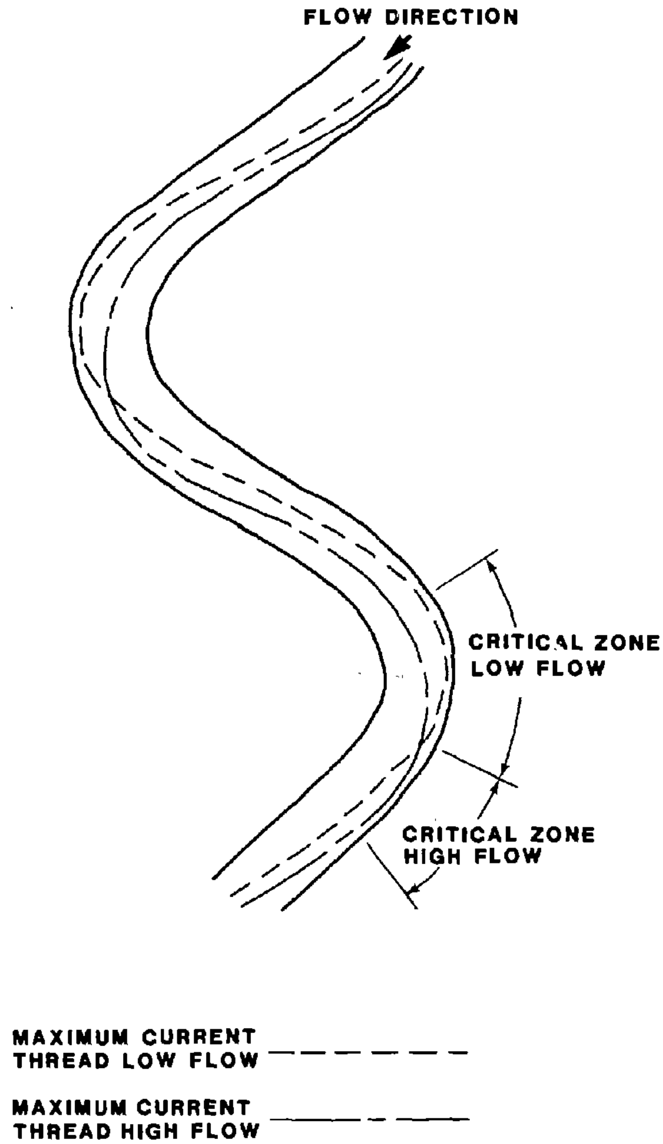


FIGURE 4. SHIFT IN MAXIMUM CURRENT THREAD WITH CHANGING STAGE

Investigations into the variations of velocities and shear forces in channel bends have been conducted by Rozovski (1957), Castle (1956), Al-Shaik (1964), and most recently, the U.S. Army Corps of Engineers (1981). The results of these studies indicate that the maximum velocity occurs in the downstream tangent to the bend and not in the upstream reaches of the bend as sometimes supposed. This fact was discovered by Rozovski (1957) during movable-bed model studies;

> "The results of the experiments disprove the assertion made by various authors that the cause of channel erosion in bends is the 'impact' of the stream on the concave bank and that at the entry into a bend, considerable erosion must take place. In actual fact, at the entry part of the bend...a certain rise of the bottom near the concave bank is observed. The most intensive erosion of the channel takes place near the exit of the bend, which...is explained by the shifting of the maximum velocity toward the concave bank and its continuation."

The maximum velocity in a channelbend has been found to range from 1.2 to 1.8 times the average channel velocities. The lower values are typical of channels having small width-to-radius-of-curvature (w/R_c) ratios, and the higher values are typical of larger w/R_c ratios. From measurements in several rivers in California, Castle (1952) has related the maximum attack velocities in channelbends to the mean channel velocity. His results are shown in Figure 5.

Studies of shear distributions that occur in channelbends have been conducted by Ippen et al. (1960), Yen (1965), and Apmann (1972). The results of these studies are valuable in the design of streambank stabilization measures. Apmann (1972) presents available data relating the maximum shear/mean shear ratio as a function of the channel width/center-line radius ratio. This data is plotted in Figure 6. From his analysis, Apmann drew the following conclusions:

- The maximum shear increases with curvature ratio.
- Surface roughness increases maximum shear by about 15 percent.
- Upstream conditions play a significant role in amplifying maximum shear if in successive curves there is a reversal of direction; this increase was on the order of 30 percent.
- Combining these influences indicates that in a bend, maximum shears might be 50 percent above the smooth trend line drawn in Figure 6.

Apmann's trend line, as well as a comparison curve taken from Soil Conservation Service (1977) are shown. Note that while the Soil Conservation Service curve appears acceptable over its application range, extending it to higher values of w/R_c will result in poor estimates of maximum shear/mean shear as attested to by the Buffalo Creek data.

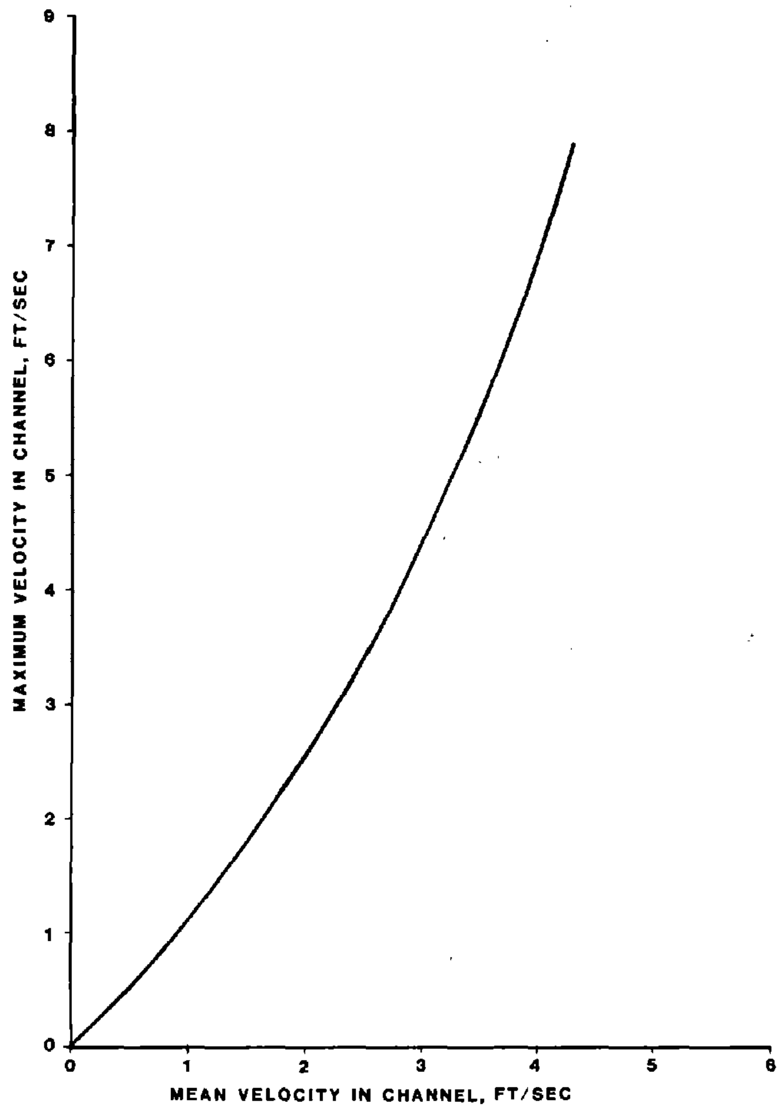


FIGURE 5. RELATIONSHIP BETWEEN MEAN AND MAXIMUM VELOCITIES IN RIVER BENDS (AFTER CASTLE, 1956)

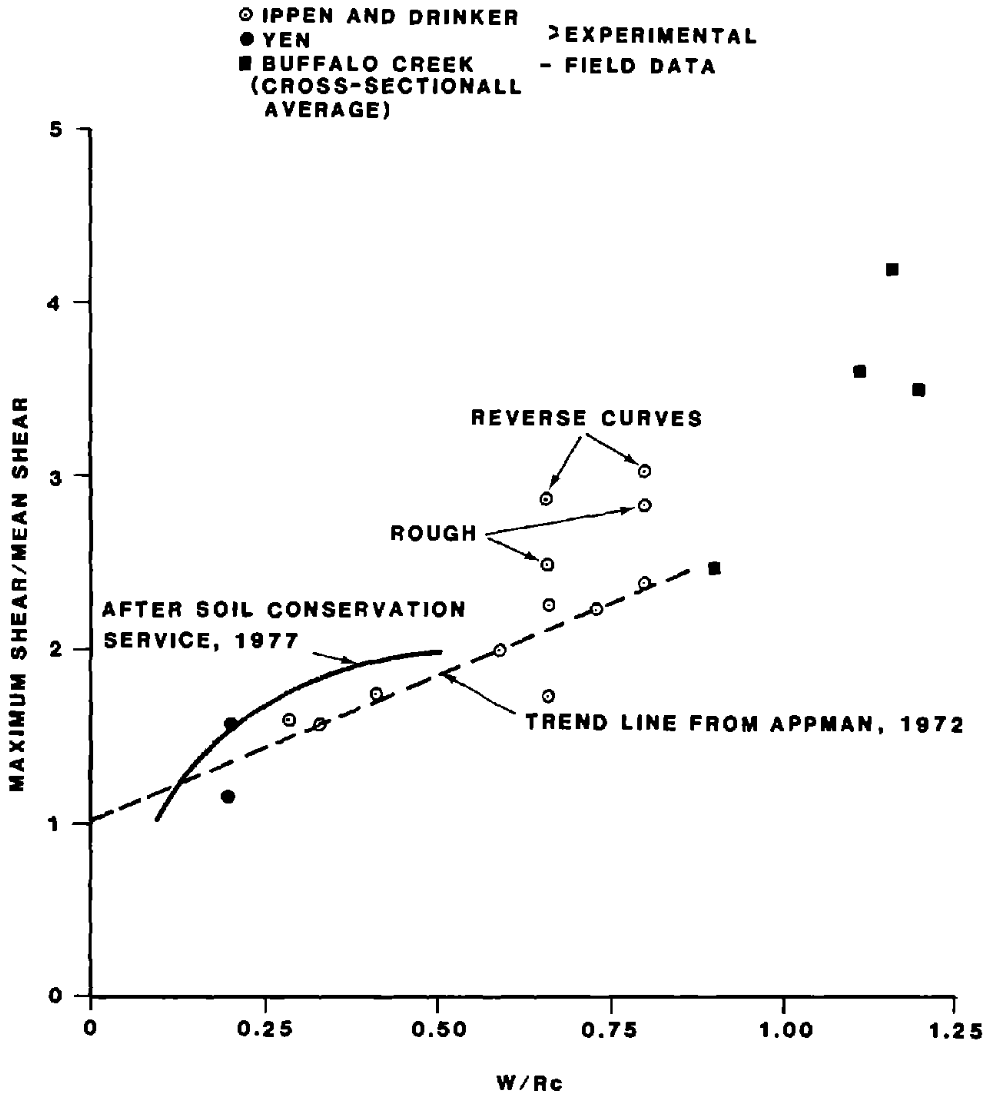


FIGURE 6. RELATION OF MAXIMUM SHEAR TO CURVATURE RATIO (MODIFIED FROM APMANN, 1972)

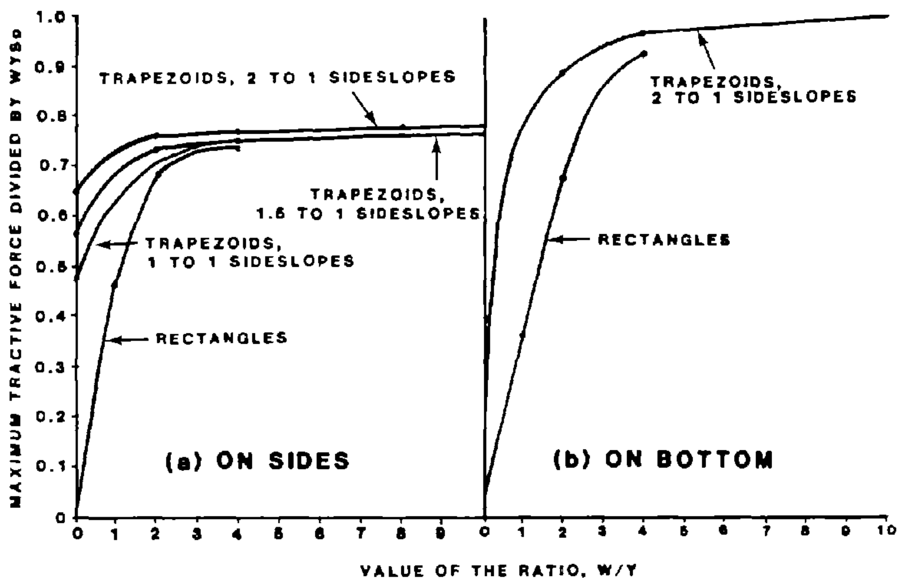


FIGURE 7. MAXIMUM TRACTIVE FORCES IN A STRAIGHT CHANNEL (AFTER LANE, 1955)

Lane (1955) has presented information that can be used to convert the maximum bed-shear stresses to bank-shear stresses. His plot of maximum tractive stresses for straight channels is shown in Figure 7; plots are shown for both channel bottoms and channel sides for various channel shapes and width to depth ratios (W/Y). The maximum tractive force multiplier from Figure 7(b) can be used to adjust the mean shear calculated as

$$\tau = \delta RS \qquad (6)$$

where,

τ = mean shear stress,
δ = specific weight of fluid,
R = hydraulic radius of channel section, and
S = energy slope (often assumed to be the channel bottom slope).

based on channel shape and W/Y. Figure 6 can then be used to calculate the maximum shear on the channelbed in a curved reach. Then, referring to Figure 7(a) an adjustment factor can be found to adjust the maximum shear on the channelbed to a maximum shear on the channelbank in a curved reach.

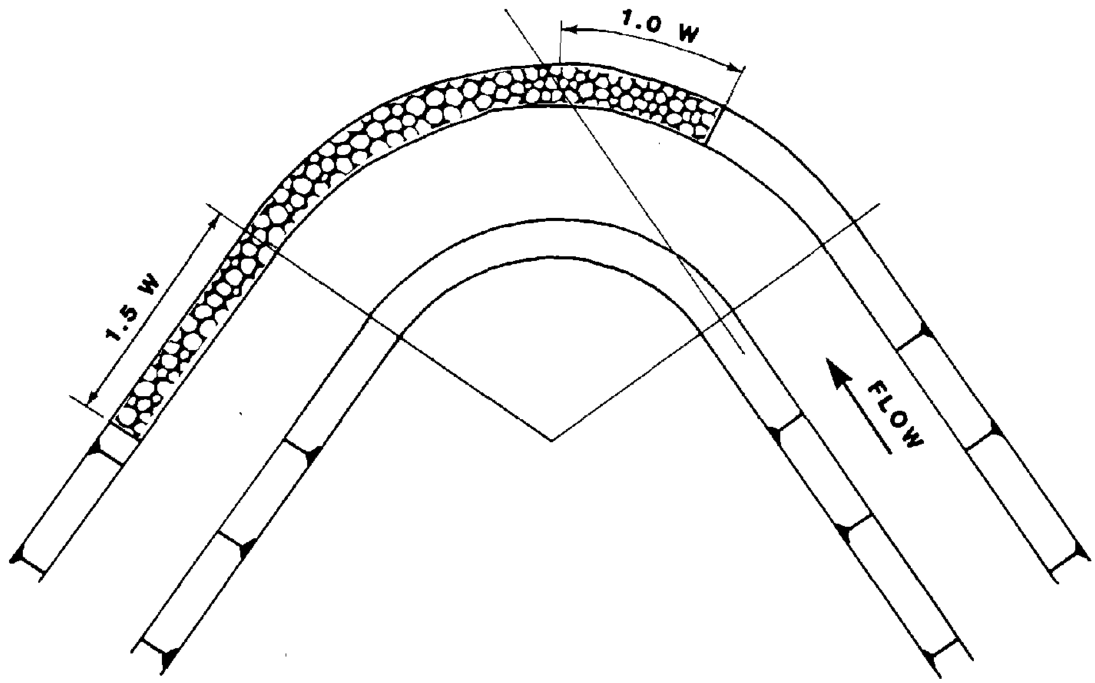


FIGURE 8. EXTENT OF PROTECTION REQUIRED AROUND A CHANNELBEND (AFTER U.S. ARMY CORPS OF ENGINEERS, 1981)

The extent to which the length of the bank is subject to erosion also must be considered. As indicated previously, the most intensive erosion occurs near the exit of the bend. Parsons (1960) has conducted field studies of the complete or partial failure of established protection measures. His results concur with the findings of Apmann; i.e., a common misjudgement in streambank-protection works is to provide protection too far upstream and not far enough downstream.

The U. S. Army Corps of Engineers (1981) conducted a series of model studies to define more completely the limits of bank protection as suggested by Parsons. From these studies it was concluded that the minimum distances for extension of protection are an upstream distance of 1.0 channel widths and a downstream distance of 1.5 channel widths from corresponding reference lines as shown in Figure 8. These findings agree with those presented in Figure 4. The protection limits presented should be used as minimum criteria; the many site-specific factors affecting field sites also should be considered in establishing the appropriate limits of protection required.

Factors Influencing the Magnitude and Rate of Streambank Erosion

The magnitude and rate of streambank erosion is governed by channel and environmental conditions unique to each river reach and situation. These characteristics include channel-flow conditions, channelbank composition, channelbank vegetation, and channelbed stability.

Flow Characteristics

Channel flow is the dominant factor in the bank-erosion process. Besides being a significant contributor to the erosion process itself, channel flow also provides the transport mechanism required to carry material away from the bank. The channel flow dynamics responsible for bank erosion were discussed above. Discharge magnitude and duration also are important flow characteristics.

Flow magnitude is directly proportional to the magnitude of bank erosion. As discussed previously, the plan view form and geometry of a channel reach is determined by its dominant flow and sediment conditions. The resulting channel is relatively stable with respect to bank erosion during low and moderate flow conditions. However, during major flow or flood events, the velocities and shear stresses driving the bank-erosion process become large enough to produce a significant erosion potential. Based primarily on field experience, it has been estimated that 90 to 99 percent of all bank erosion occurs during major flood events (Simons et al., 1979).

The duration of a particular discharge can have an impact even greater than discharge magnitude on bank stability. The initiation of channelbank erosion is similar to the initiation of channelbed erosion; it requires more energy to overcome the initial bank resistance to erosion than it does to maintain the erosion process once it has started. For channelbanks, the resistance created by bank vegetation and other cohesive forces, as well as the soil particles' structural resistance to erosion, must be overcome first. However, once the bank is exposed, the erosion process proceeds much more rapidly. Therefore, the longer the bank is exposed to a high discharge, the faster the potential rate of bank erosion.

Characteristics of Channelbank Materials

A channelbank's resistance to erosion is closely related to the characteristics of the bank material. Channelbank materials can be classified as noncohesive, cohesive, or stratified.

Noncohesive bank materials include some silts, sands, gravels, cobbles, and boulders. Channelbanks composed of these materials are usually heterogeneous deposits of silts, sands, and gravels. Pure noncohesive banks rarely exist in nature; there usually is a degree of cohesiveness provided by the silts or a small fraction of clay in the mixture. Bank vegetation also provides cohesion through its root structure. When the primary bank structure is provided by the particle to particle structure, however, the banks are considered to be cohesionless.

The erosion of noncohesive bank materials can occur on a given grain-by-grain basis or as a result of flow slides. The removal of bank materials on a grain-by-grain basis is affected by particle size, bank angle, and hydraulic factors such as velocity magnitude, and the intensity of turbulent velocity fluctuations. The bank's resistance to erosion is provided by the particle structure within the soil mass. The removal of material in this fashion causes an erosion of the lower portions of the bank, resulting in bank steepening and sloughing of upper bank material to maintain a bank angle consistent with the bank material's natural angle of repose. Flow slides occur when a buildup of water in the pores of the soil reaches a point at which the pore water pressures balance the normal pressures between particles. At this point, the material loses its shearing strength and flows down the bank. Both of these failure mechanisms are illustrated in Figure 9(A).

Because of the electrochemical forces that create the cohesive bond, cohesive channelbank materials are characterized by a very low level of permeability and a high resistance to surface erosion. These electrochemical forces can be many times stronger than gravity and may develop either directly between adjacent soil particles or between absorbed water films and thin layers of particles that are dependent upon the soil-solution chemistry of the water. The rates of erosion in cohesive materials are functions of the temperature, antecedent water content, rate of wetting, pore pressure, and chemical quality of the eroding water (Grissinger et al., 1980).

Bank saturation and drainage are primary factors in the erosion of cohesive channelbanks. In poorly drained cohesive materials, hydraulic pore pressures weaken the bank by reducing its effective strength. Even if no significant pore pressures are exerted, the stability of the bank will be reduced by saturation because of an increase in the unit weight of the material and a decrease in its internal strength. Failure of cohesive channelbanks usually occurs through loss of a block or mass of soil as a result of a shear failure within the bank. Bank height also plays an important role in the erosion of cohesive channelbanks; the larger mass of high banks increases the driving force behind the mass wasting process. Typical failure modes for cohesive banks are illustrated in Figure 9(B).

Composite or stratified banks are the most common in nature and the most complex. Any combination of cohesive and noncohesive material can exist. Stratified banks are the product of variations in past transport and deposition of sediments by the river. Specifically, these types of channelbanks consist of layers of materials of various sizes, permeability, and cohesion. Layers of noncohesive material are subject to surface erosion but may be partially protected by adjacent layers of cohesive material that makes them more stable. The noncohesive lenses also can help relieve the increased hydraulic pressures in cohesive banks during periods of bank drainage. However, these lenses can be a source of instability. Flow piping through these lenses can weaken the structure of the bank and cause failure through mass wasting or some other mechanism. Typical bank-failure surfaces are illustrated in Figure 9(C).

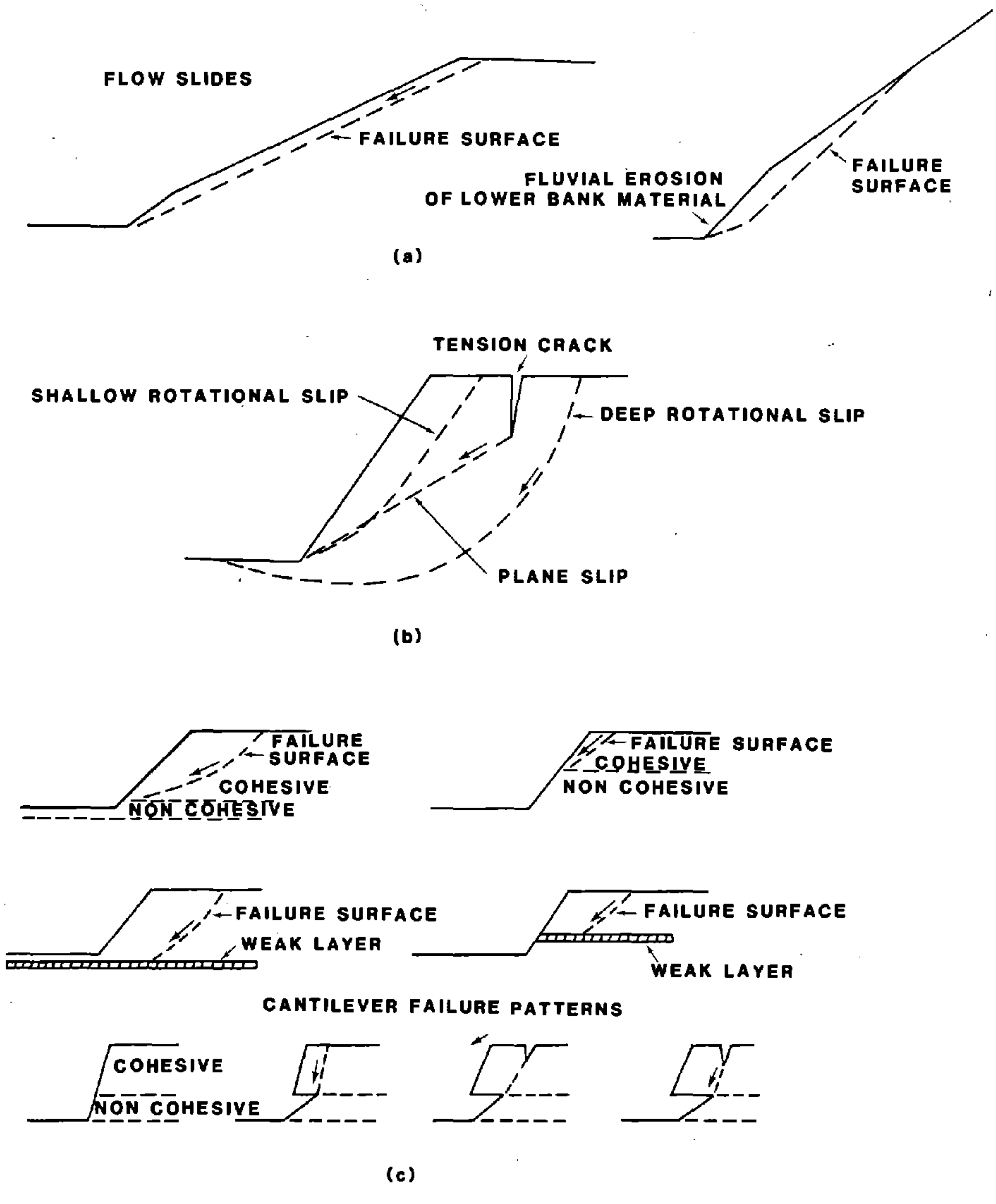


FIGURE 9. TYPICAL BANK FAILURE SURFACES (A) NONCOHESIVE, (B) COHESIVE, (C) COMPOSITE.

Channelbank Vegetation

The stability of all channelbanks, regardless of their soil composition, can be greatly influenced by the existence of natural vegetation. The primary stabilizing influence provided by streambank vegetation is its root system. The root systems of herbaceous and woody vegetation physically bind and restrain soil particles, making the banks more resistant to erosion. Root systems also influence the balance of forces in a channelbank through the transfer of soil-shear stresses to tensile resistance in the roots. Bank vegetation has been used successfully as a means of stabilizing channelbanks against some erosion mechanisms (U.S. Army Corps of Engineers, 1981), and therefore, it can be expected that existing bank vegetation will have a stabilizing influence on channelbanks.

Channelbed Stability

Channelbed instabilities are manifested through the processes of aggradation and degradation. Aggradation is the general raising of the channelbed elevation throughout a river reach, thereby increasing its rate of energy expenditure. Degradation is the general lowering of the channelbed elevation over a river reach thereby reducing its rate of energy expenditure. Degradation is of particular concern in streambank stability. Channelbed degradation usually results from some human activity that has unnaturally increased the rate of energy expended in a stream. Some of these activities will be discussed in the next section.

Channelbed degradation indirectly affects bank erosion. If the channelbed is eroded, a higher unsupported bank results that is increasingly susceptible to undercutting and failure. Also, since increased meander activity through bank erosion is another mechanism that will produce a reduction in a stream's rate of energy expenditure, it often accompanies channelbed degradation. If the channelbed material is more resistant to erosion than the bank material (which often is the case in channels that develop armor layers on their beds) the stream's erosive energies will be expanded directly on the channelbanks in the form of bank erosion.

EFFECTS OF CHANNELBANK STABILIZATION

The above discussions indicate that streambank erosion is a river system's natural mechanism for adjusting to changing hydraulic and environmental conditions, and that there are many factors influencing the rate and magnitude of bank erosion. To control bank erosion adequately at a specific site, it is important to understand the geomorphic processes and erosion mechanisms at work. It also is important to be aware of the consequences of stabilizing a channelbend.

Geomorphic impacts from channelbank stabilization affect both the cross-sectional and planview channel geometries. As discussed in previous sections, freely meandering rivers shift their bends by erosion of the outer bank and sedimentation in the remaining part of the cross section. This

FIGURE 10. COMPARISON OF CHANNELBEND CROSS SECTIONS
(A) FOR NATURAL CONDITIONS, AND (B) FOR STABILIZED BEND

results in a rather gentle slope in the cross section as illustrated in Figure 10(A). In comparison, a typical cross section profile for a stabilized channelbend is illustrated in Figure 10(B). Note that stabilizing a channelbank that has been eroding induces a steeper slope in the cross section and hence a greater thalweg depth at the toe of the stabilization structure. This channel deepening can cause the failure of the bank- stabilization structure if the additional channel depth is not anticipated during foundation design. It also can have detrimental effects on bridge piers and other structures constructed in or just downstream of stabilized bends. Excessive channel deepening, especially along the channel thalweg, can undermine pier foundations, either weakening them or causing their failure.

Another morphologic impact is the effect the stabilization of a meander bend has on the normal morphologic development and shifting of meander bends. Figure 11(A) illustrates the typical downstream progression of an uncontrolled alluvial channel. In comparison, Figure 11(B) illustrates the effect the stabilization of a single meander bend has on upstream and downstream meanders. Note that the meanders in the uncontrolled reach maintain uniform meander amplitudes and lengths as they move downstream (ideally). However, as is illustrated in Figure 11(B) as the meanders approach the stable bend, upstream meander sinuosity increases. This reduces the meander length and can increase the meander amplitude. The controlling meander length and amplitude will be determined by the characteristics of the bed and bank material in the given river reach. The upstream meander radius will gradually become smaller until the resistance to flow (caused by the reduced meander radius and increased channel length) lowers the flow energy available to erode the channelbanks below that required. In environments where the channelbanks are highly erodable, the meander bend will continue to increase its length until a cutoff or chute forms. This will occur when the resistance to flow along the increased channel length exceeds the resistance to flow across the neck of the meander.

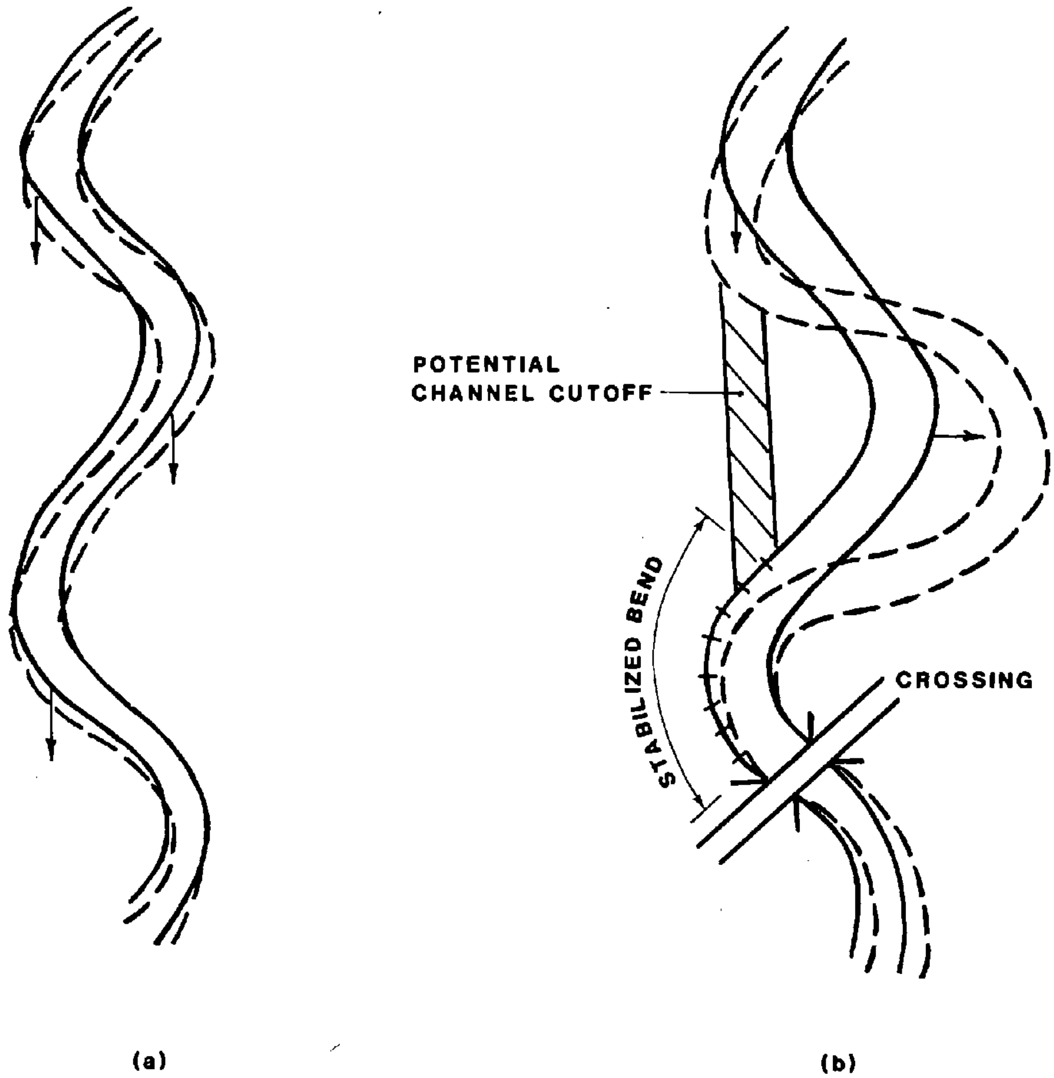


FIGURE 11. MEANDER MIGRATION IN (A) A NATURAL CHANNEL, AND (B) A CHANNEL WITH STABILIZED BEND.

The above discussion described conditions as they would occur in a highly dynamic river system cut through homogeneous material. Of course, these conditions rarely exist in nature. However, it does give an indication of the trends that will occur. The U.S. Army Corps of Engineers (1981) documented the migration of meander bends on North Fork Tillatoba Creek in Northern Mississippi. Their findings are illustrated in Figure 12. Note the general downstream movement of the meander bends in the upstream section of the reach illustrated. The downstream limit of the reach illustrated is controlled by a bridge crossing. This channel control has caused the upstream meander radius to become smaller as discussed above.

CONTROLLING STREAMBANK EROSION

There are four general ways of dealing with streambank erosion problems in the vicinity of bridges or highway embankments. The most common approach is to armor the bank with a protective revetment of sufficient integrity to resist the erosive forces. The second method is to reduce the force of the attacking water with a flow-retardance structure. The third method is to shift the attacking water away from the embankment with a flow-control device such as a spur or longitudinal dike. The fourth method, which is the best in terms of success and performance, is to move the roadway. This last method, however, is rarely feasible.

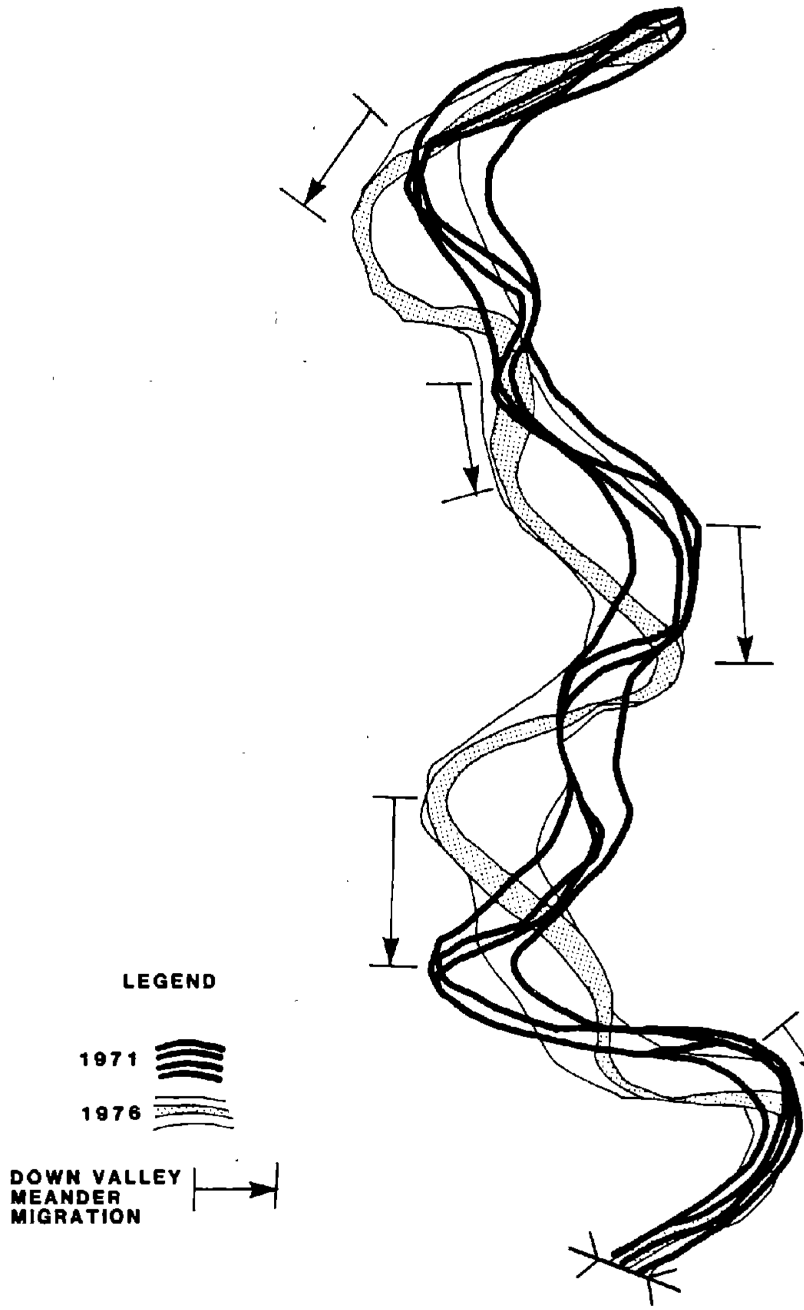


FIGURE 12. MEANDER MIGRATION, NORTH FORK, TILLATOBA CREEK
(AFTER U.S. ARMY CORPS OF ENGINEERS, 1981)

Chapter 3

CLASSIFICATION OF FLOW CONTROL AND BANK STABILIZATION COUNTERMEASURES

Countermeasures for flow control and bank stabilization can be defined as structures that protect channelbanks by providing an erosion-resistant barrier between the flowing water and the bankline or by controlling the direction and/or velocity of the flowing water. The usage of terms for different types of flow-control structures is inconsistent from one highway agency to another. In this report, types of flow-control structures are distinguished based on their mechanisms of flow control and positions relative to the bank. The following four classifications of countermeasures have been identified:

- Spurs
- Bank revetments
- Retardance structures
- Longitudinal dikes
- Bulkheads

In Figure 13, each of these countermeasures is shown in relation to the others. The following section briefly defines and describes the countermeasure groups listed above. Detailed descriptions of individual countermeasure types within these groupings, including critiques and design information, will be given in later chapters.

SPURS

A spur is a permeable or impermeable linear structure that projects from the bank into a channel to alter flow direction, induce deposition, and/or reduce flow velocities along the bank. Other common names for spur-type structures include jetties, groins, dikes, deflectors, and wing dams. A wide variety of spur designs has been documented (Acheson, 1968; Brice et al., 1978; U.S. Army Corps of Engineers, 1981). Spurs can be broadly classified as permeable or impermeable. They can be classified further by functional type as retardance spurs, retardance/diverter spurs, and diverter spurs. Retardance and retardance/diverter structures fall into the permeable spur category, while diverter structures are impermeable. Retardance spurs are designed to reduce the flow velocity in the vicinity of the bank as a means of protecting the channelbank.

Retardance/diverter structures also produce a flow retardance along the channelbank, but they are angled to produce a flow deflection away from the

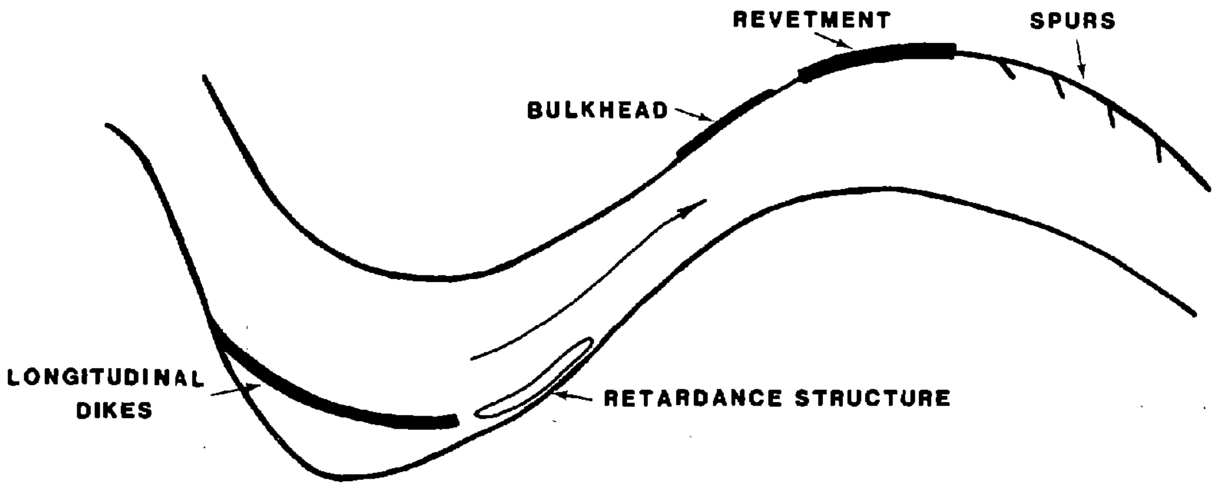


FIGURE 13. PLACEMENT OF FLOW-CONTROL STRUCTURES, RELATIVE TO CHANNELBANKS, CROSSINGS, AND FLOODPLAIN.

eroding bankline. Impermeable flow diverters function by deflecting the main flow currents away from the eroding bankline. Impermeable flow diverters function by deflecting the main flow currents away from the bank. Spurs within each of these categories can be further identified by material and construction type as follows:

Retardance Spurs

- fence type (wood or wire)
- Henson spur jetty
- jack/tetrahedron

Retardance/Diverter Spurs

- light fence (wood or wire)
- heavy diverters

Diverter Spurs

- hardpoints
- transverse dike spurs

Further subclassifications of the various spur types can be made based on other design and construction details. For example, riprap includes graded rock, random rock, rubble, furnace slag, etc. In addition, riprap spurs can be constructed using hand placement, plating, or random-dump techniques. Similar variations exist for other spur types. Common spur types are illustrated in Figures 14 through 26. The most widely used spur types are riprap, wood fence, and wire fence structures.

FIGURE 14. HENSON TYPE SPUR JETTY. BARZOS RIVER NEAR ROSHARON, TEXAS.

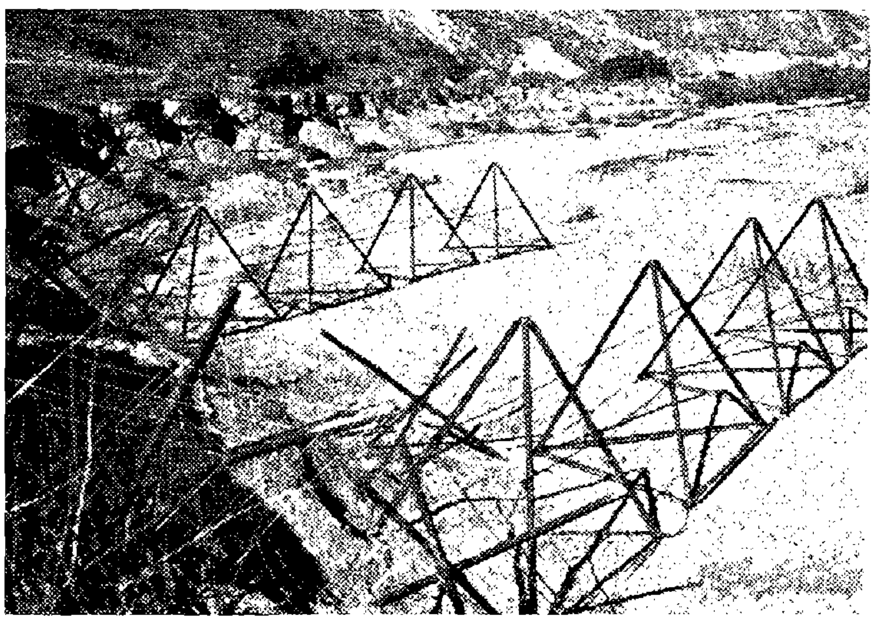


FIGURE 15. TETRAHEDRON SPURS SAN BENITO R., CALIFORNIA (SOURCE: CALIFORNIA DEPARTMENT OF PUBLIC WORKS, 1970)

FIGURE 16. WOOD-FENCE SPUR BATUPAN BOGUE, GRENADA, MISSISSIPPI

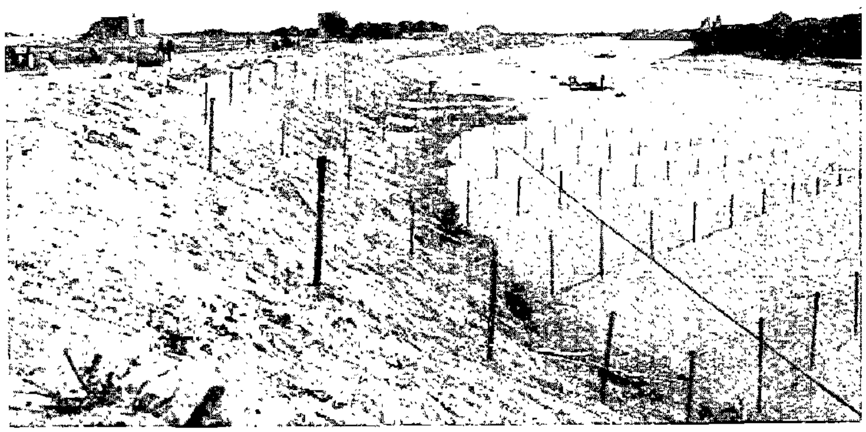


FIGURE 17. WIRE FENCE SPURS
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

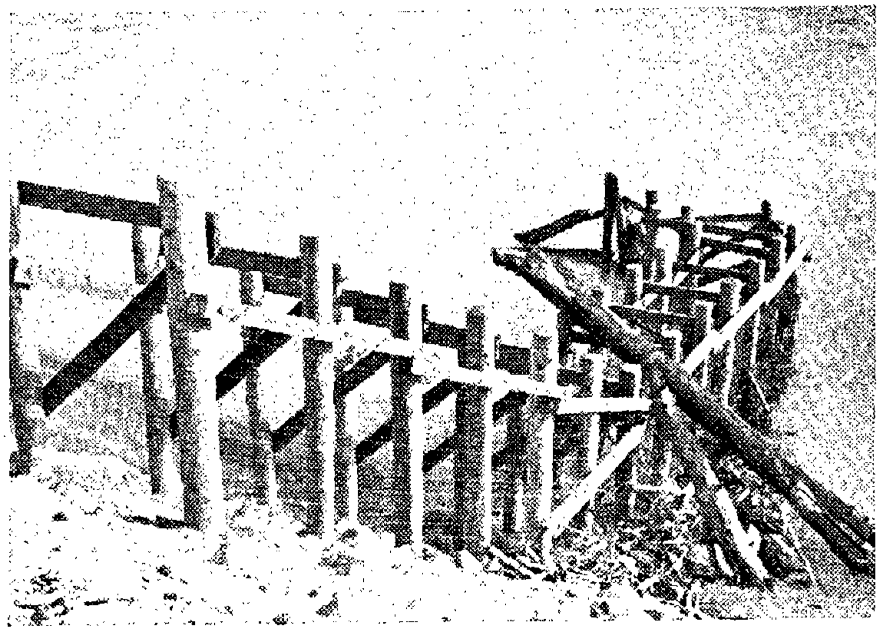


FIGURE 18. DOUBLE-ROW TIMBER PILE AND WIRE-FENCE SPUR (SOURCE: CALIFORNIA DEPARTMENT OF PUBLIC WORKS, 1970)

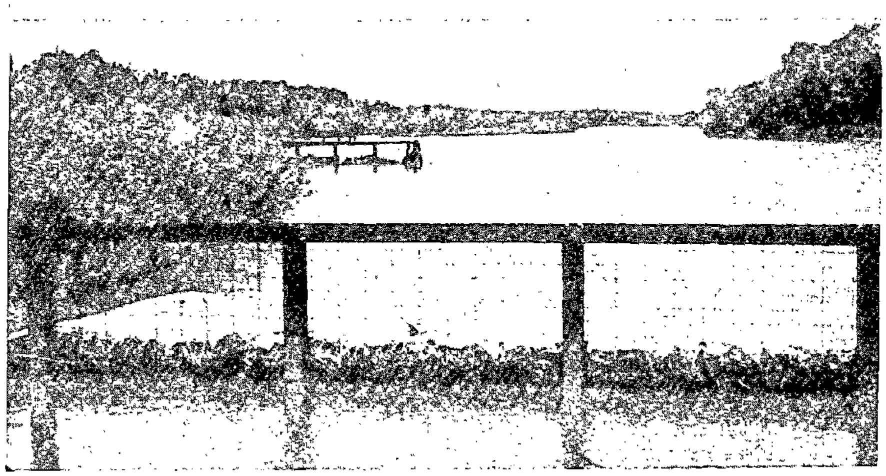


FIGURE 19. WELDED-WIRE AND STEEL H-PILE PERMEABLE SPUR. ELKHORN RIVER AT SR-32 AT WEST POINT, NEBRASKA. (SOURCE: BRICE ET AL., 1978)

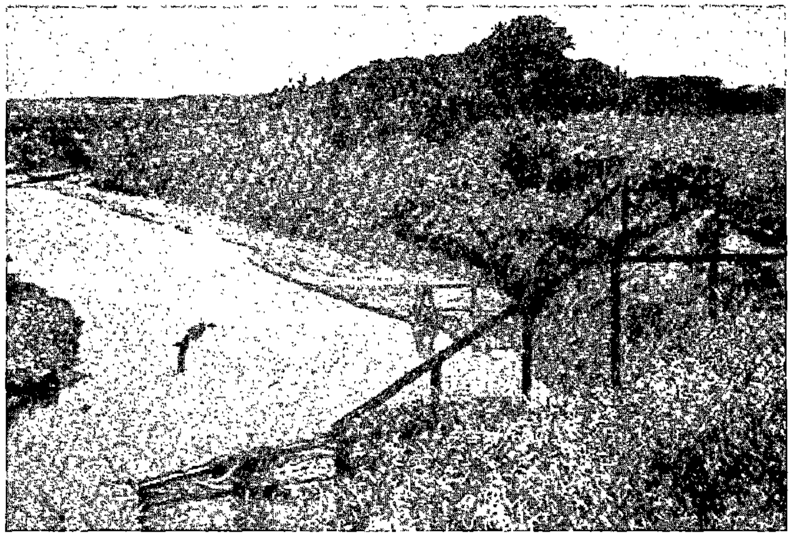


FIGURE 20. STEEL PILE/WELDED WIRE MESH SPUR;
LOGAN CREEK NEAR PENDER, NEBRASKA
(SOURCE: BRICE ET AL., 1978)

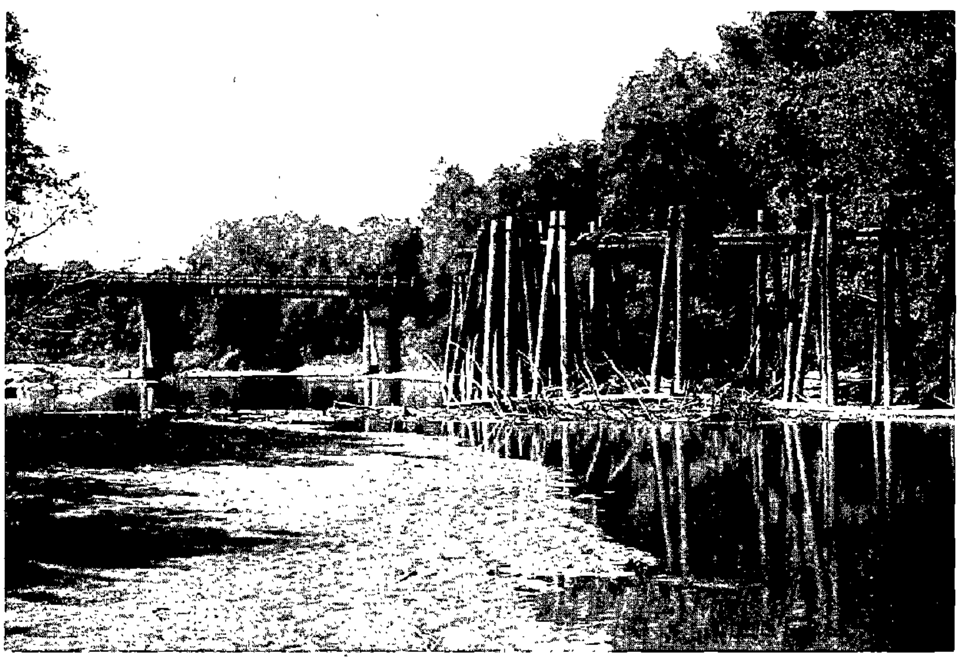


FIGURE 21. TIMBER PILE SPURS. BIG BLACK RIVER AT DURANT, MISSISSIPPI.

FIGURE 22. TIMBER PILE/SUSPENDED LOG SPURS. ELKHORN RIVER WEST OF ARLINGTON, NEBRASKA.

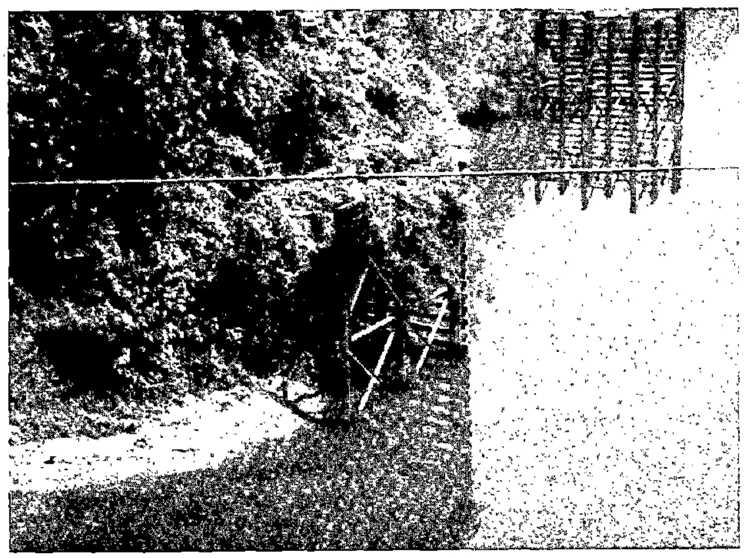


FIGURE 23. TIMBER PILE AND HORIZONTAL WOOD PLANK DIVERTER STRUCTURE. (SOURCE: BRICE ET AL., 1978)

FIGURE 24. ROCK RIPRAP SPUR, LOYALSOCK CREEK NEAR MONTOURSVILLE, PA.
(COURTESY PENNSYLVANIA DEPT. OF TRANSPORTATION, DISTRICT 3-0)

FIGURE 25. GABION SPURS, LOYALSOCK CREEK NEAR
LOYALSOCKVILLE, PA.

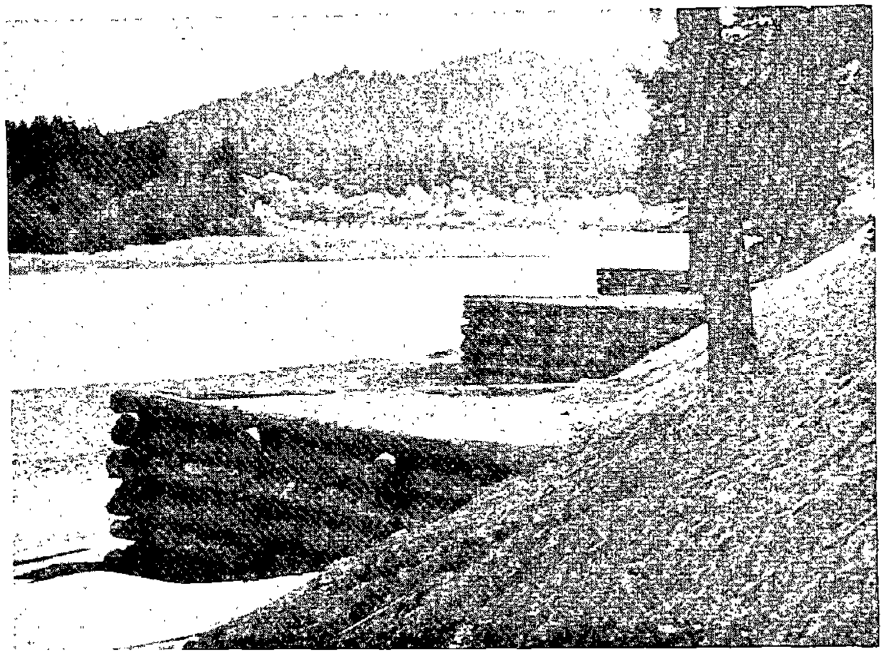


FIGURE 26. CRIB SPURS (SOURCE: CALIFORNIA DEPARTMENT OF PUBLIC WORKS, 1970)

REVETMENTS

Channelbank revetments provide channel streambank protection by armor; that is, by facing a bank or embankment with erosion-resistant material. Revetments are distinguished from other countermeasures in that they are totally supported by the bank itself. Revetments can be classified as rigid or flexible. Flexible revetments can conform to changes in the underlying surface (caused by subsidence or erosion) without being seriously damaged. Conversely, rigid revetments do not conform to such changes, and thus, may fail because of lack of support. Revetments can be subdivided further by means of the armor layer material. The following is a listing of common revetment materials:

Rigid

- Pavement (concrete and asphalt)
- Concrete-filled mats
- Sand/cement bags
- Grouted riprap

Flexible

- Riprap
- Rock windrow
- Rock and wire mattress
- Tire mattress
- Precast concrete blocks
- Vegetation

As with spur-type structures, further subdivisions can be made based on specific materials and construction methods. The most common revetment material is rock riprap.

Revetments also can be designed as composite structures, which incorporate two or more revetment materials into a single design. Typically, a material of high erosion resistance is used on the lower portion of the channelbank, and a lesser material is used on the upper bank. Upper-bank protection usually is provided by vegetation treatments.

Typical revetment schemes are illustrated in Figures 27 through 35.

RETARDANCE STRUCTURES

Retardance structures are permeable bank-protection structures designed to check riparian velocity and induce silting or accretion. They are customarily constructed at and parallel to the toe of the slope either in a linear or area design. The primary function of a retardance structure is to offer protection to the toe of the bank by reduction of in-channel velocities. The resulting deposition reverses the trend of erosion and replaces lost material. This causes a shifting of the strength of the stream away from the bank. The following is a list of common types of retardance structures:

Linear

- jacks/tetrahedrons
- wood fence
- wire fence
- timber pile

Area

- jacks/tetrahedrons
- fence

As with other countermeasures, further subclassifications can be made based on specific materials used and design configurations. Typical retardance-structure designs are illustrated in Figures 36 through 41.

FIGURE 27. CONCRETE PAVEMENT REVETMENT
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

FIGURE 28. SAND/CEMENT BAG REVETMENT
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

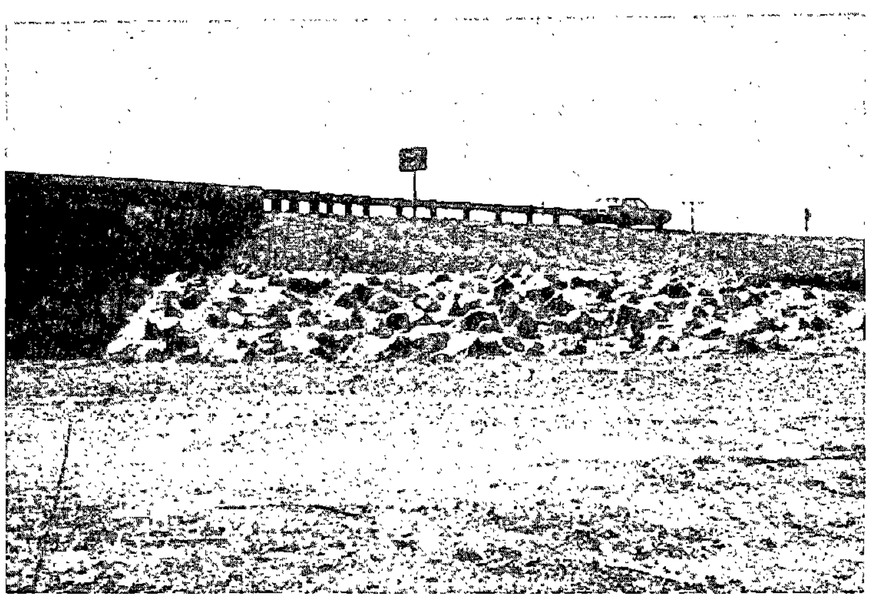


FIGURE 29. GROUTED RIPRAP
(SOURCE: BRICE ET AL., 1978)

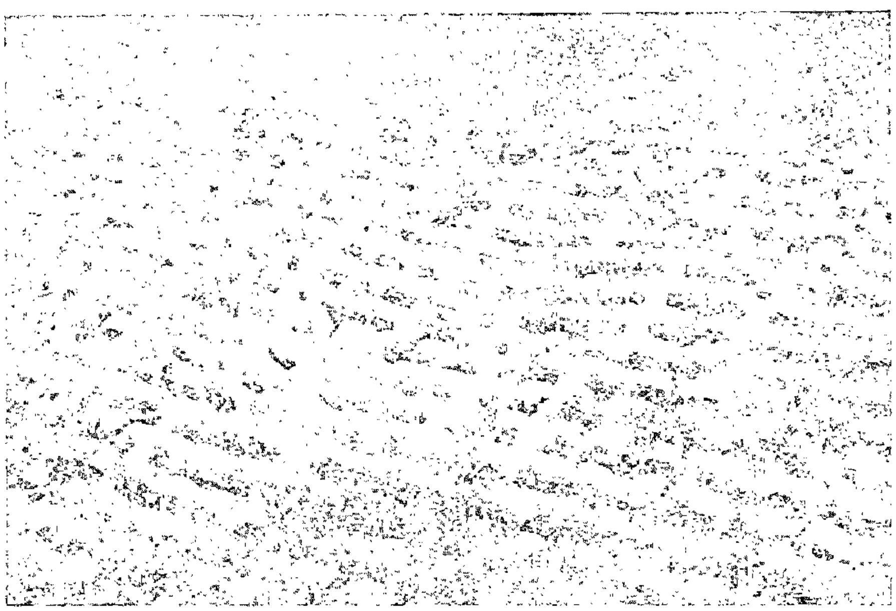


FIGURE 30. ARTICULATED CONCRETE BLOCKS
(U.S. ARMY CORPS OF ENGINEERS, 1981)

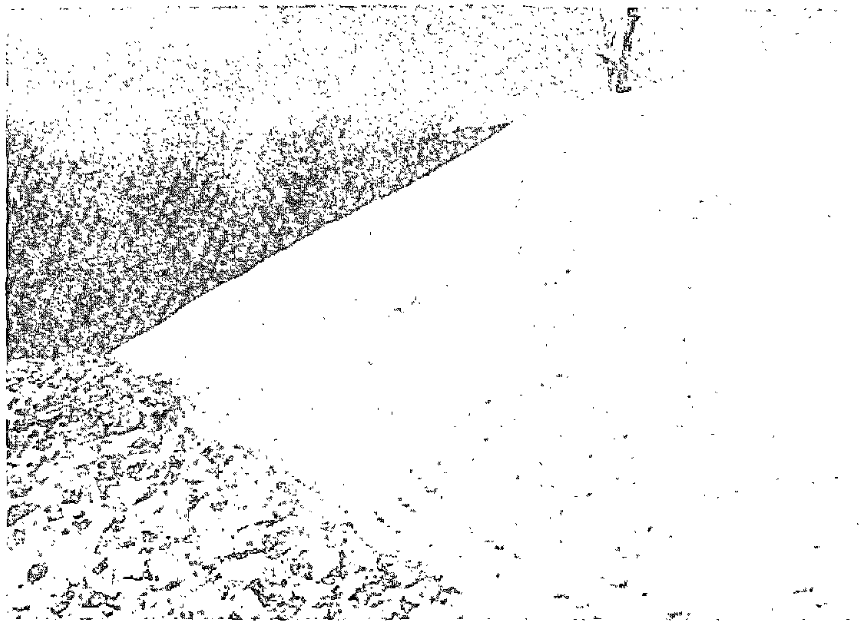


FIGURE 31. CONCRETE-FILLED MATS
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

FIGURE 32. RIPRAP LINING.
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

FIGURE 33. ROCK WINDROW REVETMENT
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

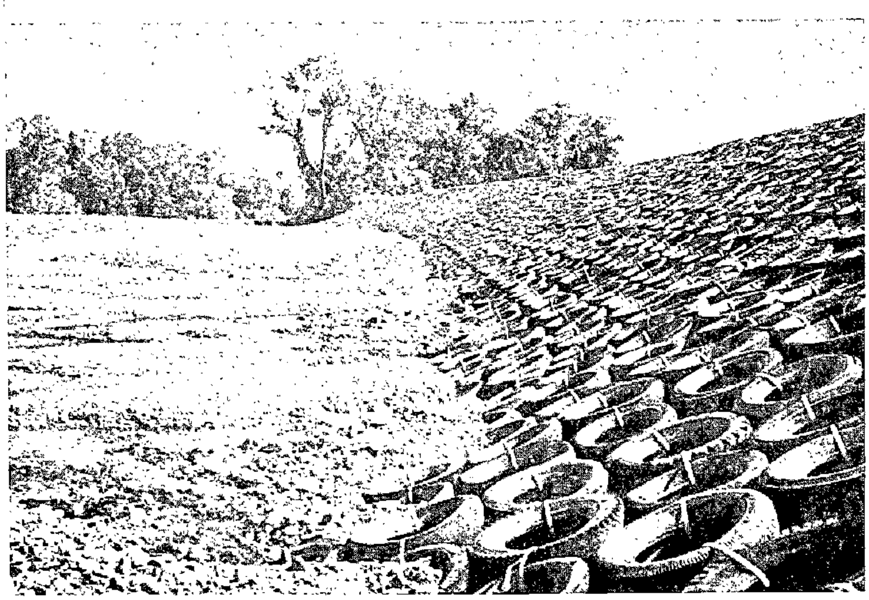


Figure 34. TIRE MATTRESS
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

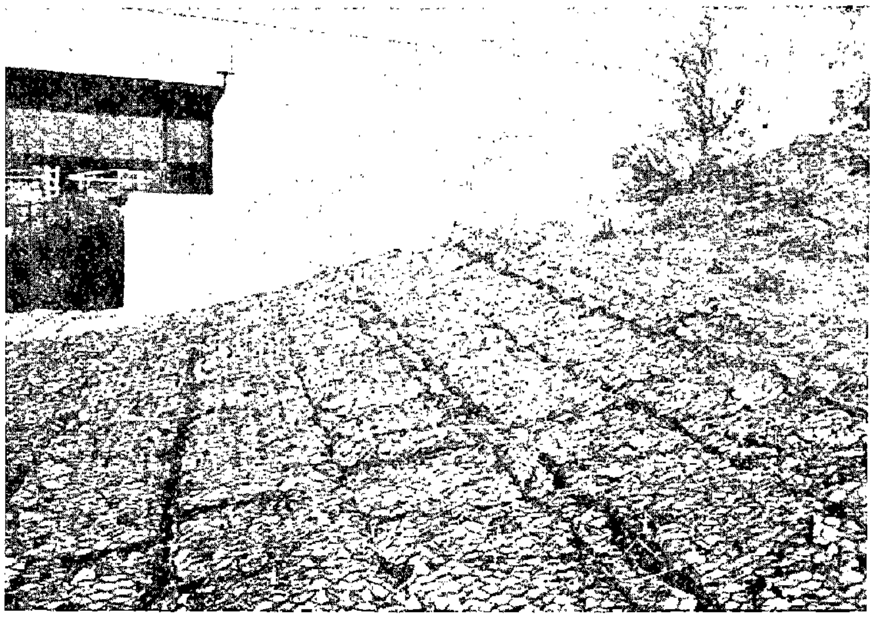


FIGURE 35. ROCK AND WIRE GABION MATTRESS
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

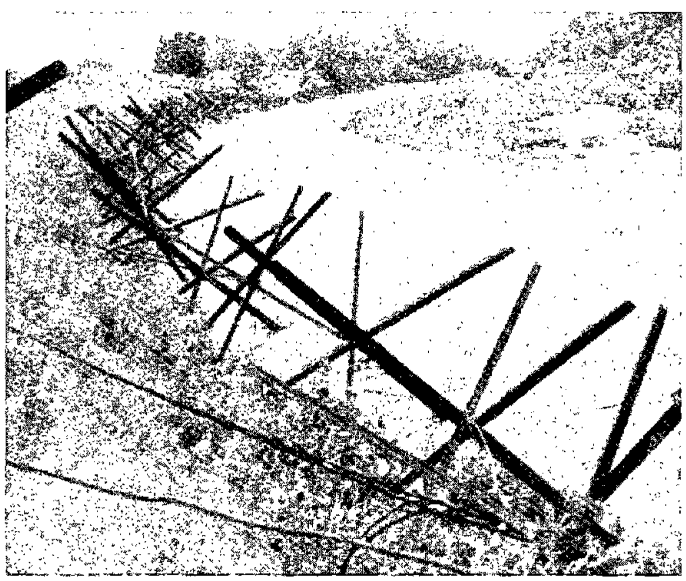


FIGURE 36. JACK-TYPE LINEAR RETARDANCE SYSTEM
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

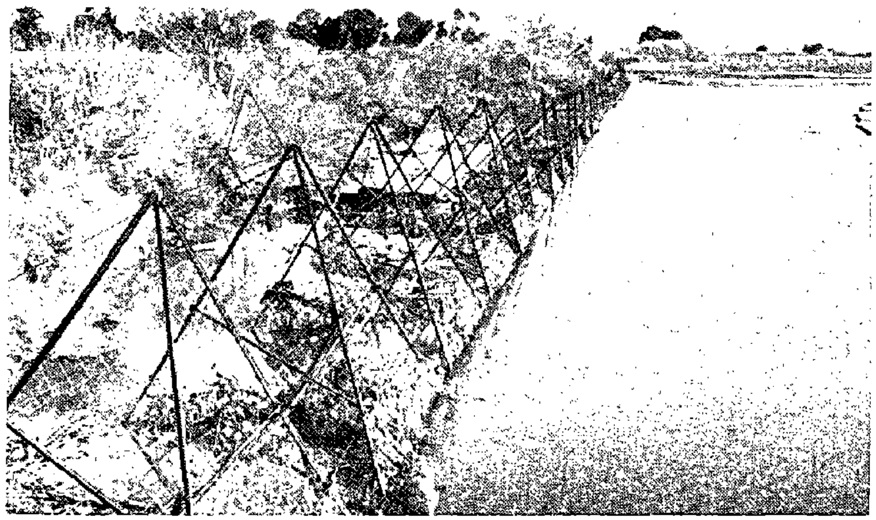


FIGURE 37. TETRAHEDRON TYPE LINEAR RETARDANCE STRUCTURES
(SOURCE: CALIFORNIA DEPARTMENT OF PUBLIC WORKS, 1970)

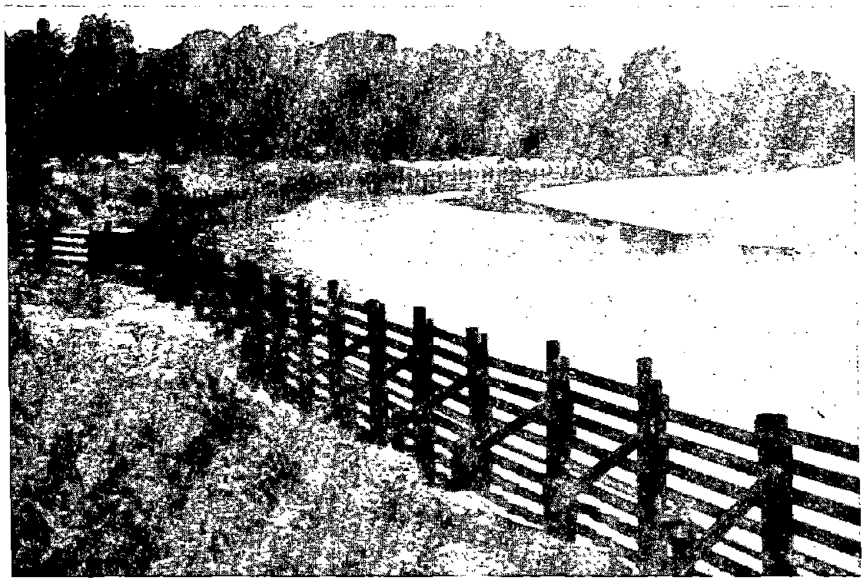


FIGURE 38. LINEAR WOOD FENCE RETARDANCE STRUCTURES
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

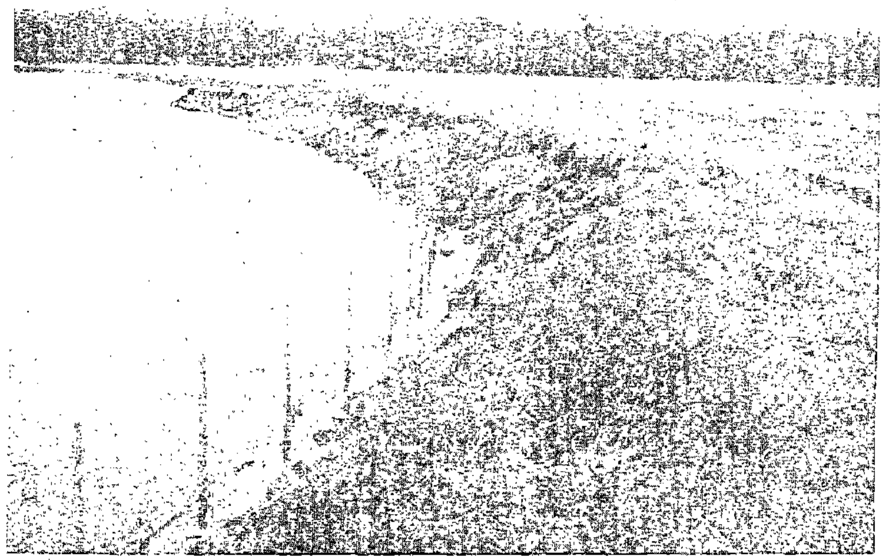


FIGURE 39. LINEAR WIRE-FENCE RETARDANCE STRUCTURES
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

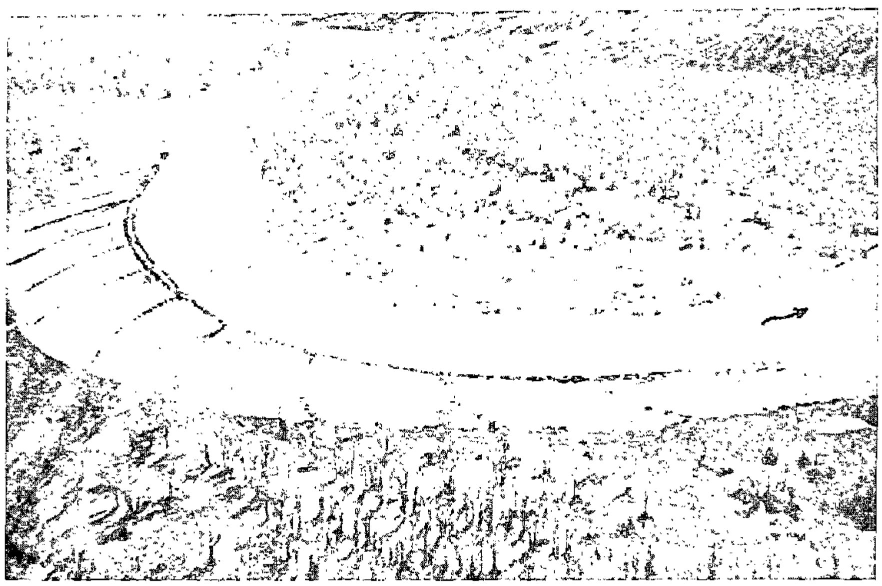


FIGURE 40. AREA JACK-TYPE RETARDANCE STRUCTURES
(SOURCE: BRICE, ET AL., 1978)

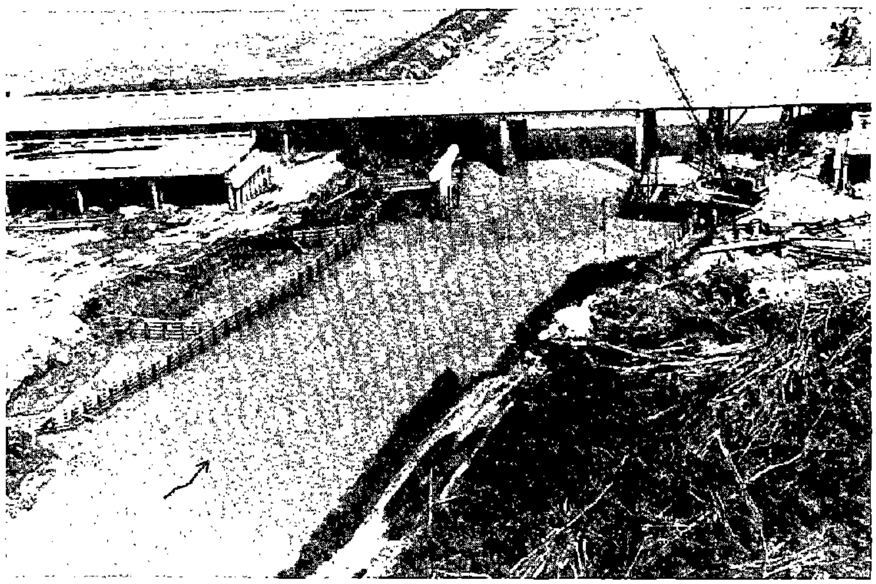


FIGURE 41. AREA FENCE RETARDANCE STRUCTURE
(SOURCE: BRICE ET AL., 1978)

LONGITUDINAL DIKES

Longitudinal dikes are barriers constructed parallel to the bankline or the desired flow alignment. They differ from linear retardance structures in that they are essentially impermeable to flow conveyance. Longitudinal dikes are used primarily as toe and lower-bank protection. There are three main types of longitudinal dikes. They are:

Rock and/or earth embankments,
Rock toe dikes, and
Crib dikes

Rock and earth embankment dikes are used primarily to provide flow alignment by restoring or increasing the height of an existing bank or by creating a new bank line and flow alignment. As the name implies, they usually are constructed of rock or earth in materials faced with rock riprap or other revetment. Typical rock-toe dikes are illustrated in Figure 42. These structures consist of a dike or rock riprap material placed parallel to and at the toe of the channelbank. Toe dikes are designed to protect the toe of the bank from undermining caused by dynamic scour and general channel degradation. Crib dikes can be constructed of a variety of structural crib member- and fill materials. The most common crib dikes are constructed of a double row of wire fence with rock riprap fill. A typical wire-crib dike is shown in Figure 43.

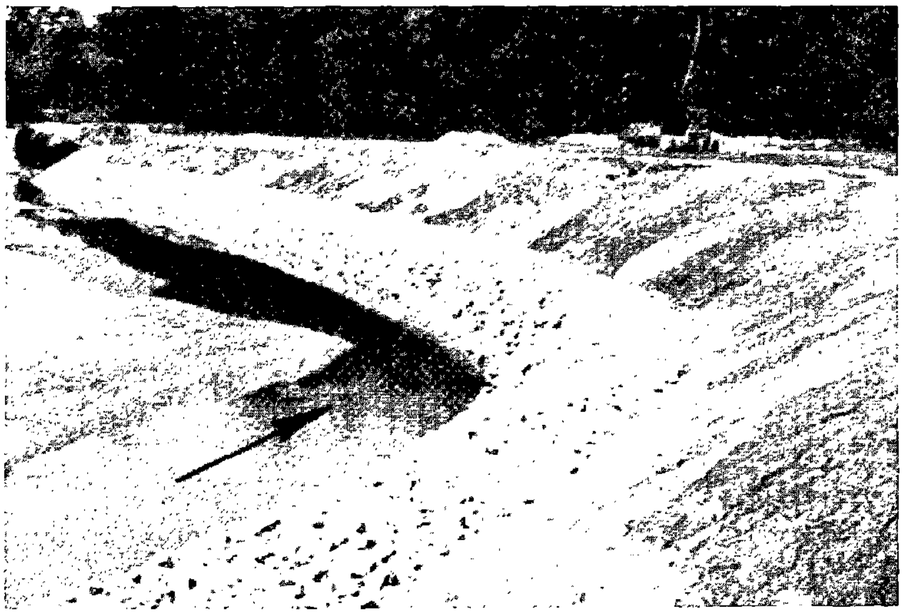


FIGURE 42. ROCK-TOE DIKE
(SOURCE: U.S. ARMY CORPS OF ENGINEERS)

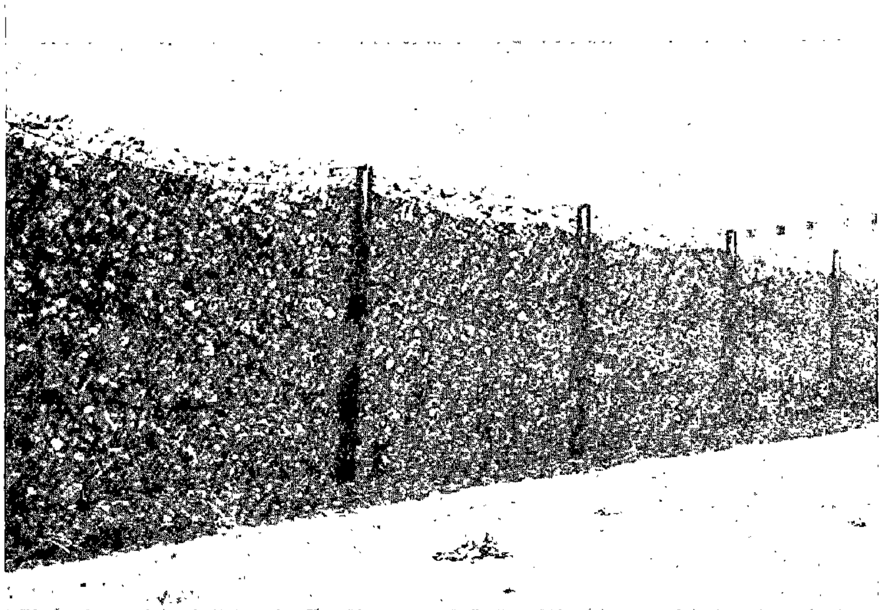


FIGURE 43. WIRE AND ROCK CRIB LONGITUDINAL DIKE
(SOURCE: BRICE, ET AL., 1978)

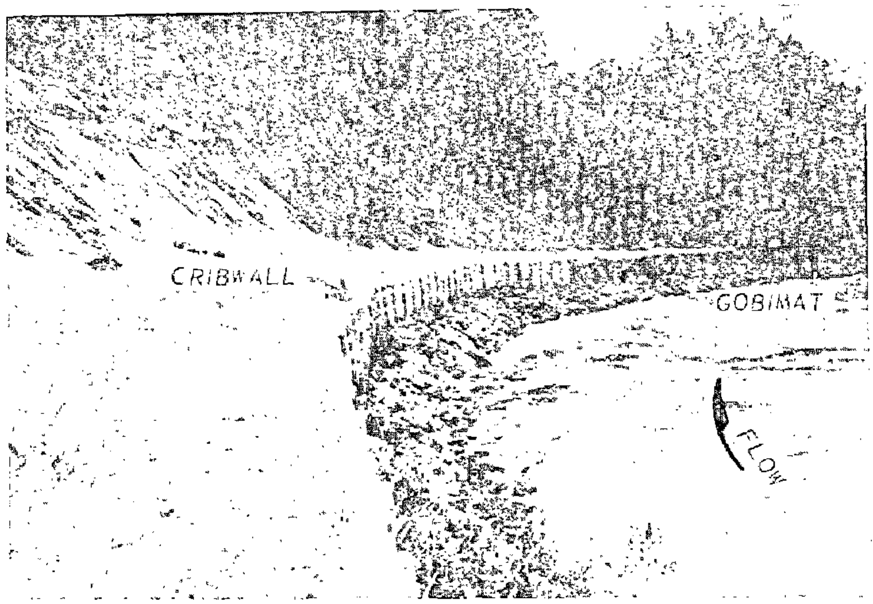


FIGURE 44. CRIB-TYPE BULKHEADS
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

BULKHEADS

A bulkhead is a vertical or near-vertical structure supporting natural or artificial embankments. Bulkheads usually are very expensive and may be economically justified only in special cases where valuable property or improvements are involved. Bulkheads usually are used as lower-toe protection in combination with an upper-bank revetment. Bulkhead designs include:

cribs,
concrete walls,
sheet piles,
timber piles,
gabions, and
stacked tires.

Several typical bulkhead designs are illustrated in Figures 44 through 48.

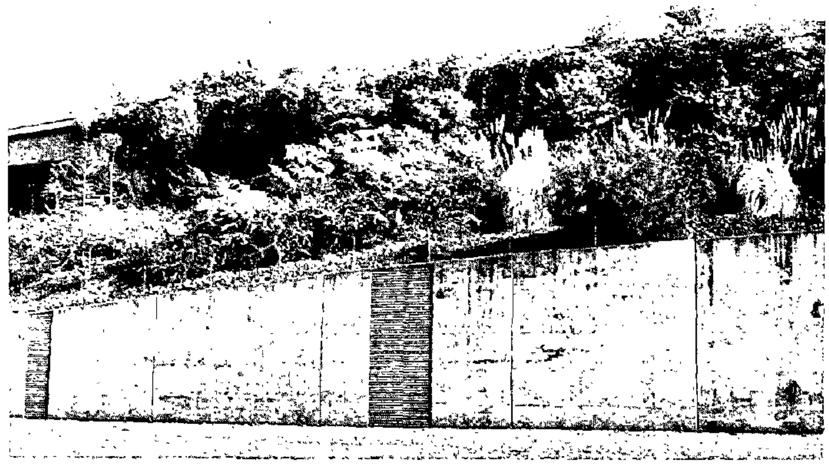


FIGURE 45. CONCRETE WALLS
(SOURCE: GRAY AND LEISER, 1982)

FIGURE 46. GABION BULKHEADS
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

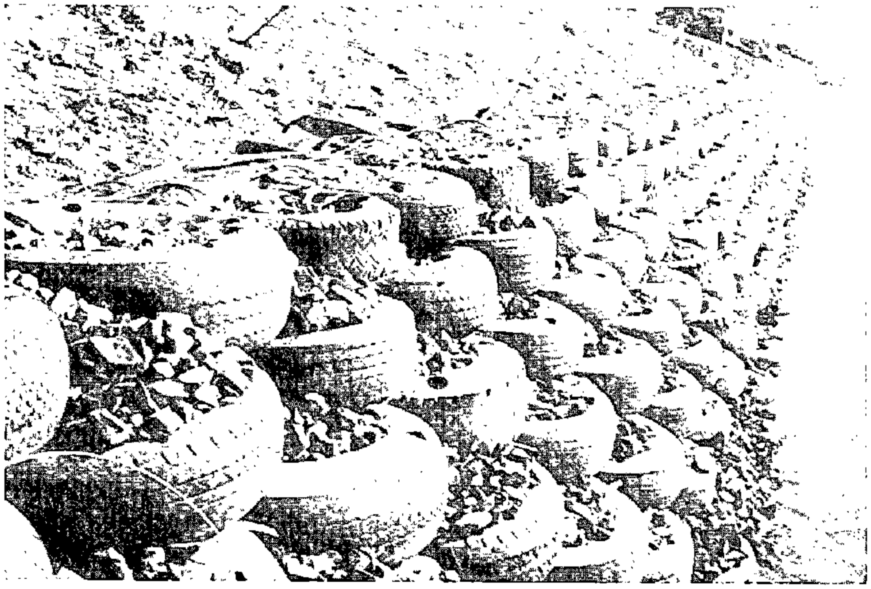


FIGURE 47. STACKED-TIRE BULKHEADS
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

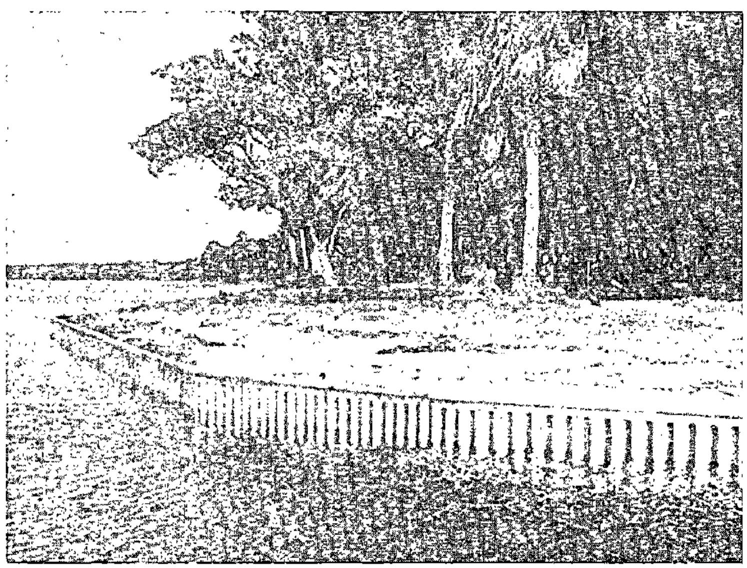


FIGURE 48. SHEETPILE BULKHEADS
(SOURCE: U.S. ARMY CORPS OF ENGINEERS, 1981)

Chapter 4

CRITERIA FOR THE EVALUATION AND SELECTION OF A COUNTERMEASURE TYPE

The selection of an appropriate countermeasure type for a specific bank erosion/channel instability problem is dependent on many factors or selection criteria, including structure, function, or purpose, erosion mechanism, river characteristics, system impacts, vandalism, maintenance, construction-related factors, legal considerations, and costs. Of these, the primary criteria are structure, function, erosion mechanism countered, and river environment. These factors define the set of specific countermeasures that are best suited to the specific site conditions. From this point, consideration of potential environmental impacts, maintenance, construction-related activities, and legal aspects can be used to refine the selection a bit more. When all these factors are of equal importance, the final determining factor is cost; the structure that provides the desired level of protection at the lowest cost will be the "best" for a particular site. Of all the factors mentioned, structure costs always will play a major role in the final selection of a countermeasure system.

The selection of a countermeasure type is site-specific and depends on the combined effect of all the selection criteria listed above. Because of their interdependence, it is impossible to isolate each of the above criterion and evaluate the effectiveness of each countermeasure type solely as a function of that criterion. Therefore, the following sections will consider each of the criterion and in some cases will offer guidelines regarding the applicability of various countermeasure types. It must be remembered, however, that it is the collective evaluation of the selection criteria that will result in the choice of an appropriate countermeasure.

COUNTERMEASURE FUNCTION OR PURPOSE

Three bank stabilization/flow-control functions have been identified. They are as follows:

- Protect an existing bankline,
- Reestablish some previous flowpath or alignment, and
- Control and constrict channel flow.

The **protection of an existing bankline** is the primary function of most bank-stabilization countermeasures. The only exceptions to this are structures designed specifically to reestablish some previous flow alignment away from the existing bank.

The **reestablishment of some previous or new flowpath or alignment** is best facilitated through the use of dikes, spurs, and/or area retardance structures. Since revetments by definition need to be supported by the bank, they are not applicable here; the use of bulkheads would require an excessive amount of backfill and therefore would be prohibitively expensive; linear retardance structures allow the development of flow channels between the structure and the eroded bank, allowing continued bank erosion.

To **control and/or constrict channel flows**, spurs and longitudinal dikes are the best countermeasures. Again, revetments do not apply; retardance structures have been found to be ineffective as flow-control structures in most cases; bulkheads would be excessively expensive if used in this capacity.

EROSION MECHANISM

In Chapter 2, it was pointed out that bank-erosion processes are primarily dependent on a transport mechanism and a bank- weakening or particle-displacement mechanism. The flowing water provides the transport mechanism, and bank weakening/particle displacement mechanisms can be provided by any of the following:

- streamflow-toe attack,
- streamflow-bank surface attack,
- surface weathering,
- abrasion subsurface flow,
- wave erosion, and
- chemical action.

Table 1 is presented as a guide to assist in selecting a countermeasure type that will best counter the erosion mechanism active at a specific site. Please note that this table assumes sufficient structural integrity of the countermeasure indicated. The effectiveness of the countermeasure group, as listed, implies that countermeasures within this grouping have been effective against the specific mechanism. This does not, however, guarantee that all countermeasures within that group will be adequate; each specific countermeasure should be evaluated on its own merit.

RIVER CHARACTERISTICS

There are many river characteristics that influence the selection of a countermeasure. These include the following:

- channel size (width),
- channelbank characteristics,
- channelbed environment,
- channelbend radius,
- channel hydraulics, and
- ice and debris loadings.

TABLE 1. COUNTERMEASURE EFFECTIVENESS AS A FUNCTION OF EROSION MECHANISM.

COUNTERMEASURE	EROSION MECHANISM							
	Transport	Bank Weakening/Particle Displacement						
	Streamflow	Streamflow		Abrasion	Subsurface Flow	Waves	Surface Weakening	Chemical Action
		Toe	Bank					
SPURS								
Permeable	$X^{1,2}$	X	$X^{3,2}$	X^2				
Impermeable	X	X	$X^{3,2}$	X^2				
REVETMENTS								
Rigid		X^2	X^2	X^2	X^4	X	X	X
Flexible		X^2	X^2	X^2	X^4	X	X	X
RETARDANCE STR.								
Area	$X^{1,2}$	X	$X^{2,3}$	X^2		X		
Linear	$X^{1,2}$	X	$X^{2,3}$	X^2		X		
DIKES								
Rock or Earthen Embankment	X^2	X^2	X					
Rock Toe	X^2	X						
CRIB	X^2	X	X^2	X^2		X		
BULKHEADS	X	X	X^2	X^2	X^4	X	X	X

NOTE: In all cases effectiveness assumes adequate structure design.

[1] Reduces Velocity: Specific design and river conditions determine actual effectiveness.

[2] If structure is construction to an elevation lower than bank height, transport and/or erosion of upper bank materials may continue during periods of high flow.

[3] Primary use had been for lower bank erosion; can also be effective as upper bank protection in some cases

[4] May require addition of special drainage structure.

These characteristics primarily influence the applicability of individual countermeasure designs and not the selection of a specific countermeasure group.

With respect to **channel size**, rivers can be classified as small (< 150 feet wide), medium (150 to 500 feet wide), and large (>500 feet wide). Channel size was found only to influence the use of spur-type structures. On small and some of the smaller medium-sized channels, some spur designs can create too much flow constriction, and as a result, can cause current deflections towards the opposite bank. In addition, the excess channel constriction can cause bed degradation greater than that caused by other countermeasure types. This results in a need for deeper, more expensive foundations. However, spurs can be used effectively on small channels where their function is to shift the location of the channel. In these cases, there usually is sufficient area available so that excessive flow constriction is not a problem. Rock-hard points having a minimal projected area also have been used successfully on small channels.

Channelbank characteristics important to the selection of a specific countermeasure type include bank height, bank configuration, bank-material composition, and type and extent of bank vegetation. Bank heights can be classified as low (< 10 feet), medium (from 10 feet to 20 feet), or high (>20 feet). If bank-surface erosion is a mechanism of erosion to be countered, bank height will play a role in the countermeasure selection process. In general, bulkheads are only cost effective for the protection of low banks, and retardance structures, spurs, and longitudinal dikes are only cost effective for the protection of low to medium height banks. High bank protection requires the use of bank revetments: either alone or as a composite design with upper bank revetments in combination with one of the other countermeasure types used for lower-bank protection.

Bank material composition relates to both countermeasure construction and the type, rate, and extent of erosion occurring. Whether it is clay, silt, or sand, the bank material will influence the construction techniques used for some countermeasures and thus affect their costs. For example, the costs associated with constructing countermeasures requiring pile driving will be higher when clay is used instead of sand or silt. The type, rate, and extent of erosion occurring is related to the erosion mechanism criteria given above.

Bank configuration refers to the geometry of the bank. High, steep-cut banks will pose design requirements different from gently sloping or low banks. Each specific countermeasure is designed with this in mind, and therefore, no general recommendations can be given here.

Bank vegetation also influences the selection of a specific countermeasure type. In areas where significant bank vegetation exists, this vegetation usually will volunteer to the bank, helping to stabilize both the upper and lower sections of the bank. The existence of significant amounts of bank vegetation and the possibilities of having this vegetation volunteer to exposed areas of the bank can, in some cases, reduce the level of protection required by enhancing the stabilizing features of a particular countermeasure type.

The channelbed environment can be described either as regime, threshold, or rigid. Regime channelbeds are defined as those whose beds are in motion under most river flow conditions. The large rivers of the Midwest (the Mississippi and the Missouri) are prime examples. Many smaller rivers and streams, however, can be regime channels. In general, channels cut through sand- and silt- size noncohesive materials are regime. Well-defined ripple and dune formations on a channelbed signal regime conditions. Threshold channelbeds are defined as channelbeds that are stable under most normal flow conditions but become mobile at some higher than normal river flows. Regime channels develop an armor layer near the bed surface that is stable at normal flow conditions, but is broken and becomes mobile at higher than normal flow. Regime channels can cut through cohesive or noncohesive materials. In cohesive materials the armor layer is created by the bonding forces of the clay minerals; in noncohesive materials an armor layer of coarse-grained material develops through the sorting action of sediment transport. Noncohesive, threshold channels typically are cut through channels having a wide range of bed-material sizes (such as sand and gravels; gravels and cobbles; or sands, gravels, and cobbles) and can readily be identified by the armor layer of coarse-grained materials on the bed under low-flow conditions. Rigid channelbeds are those cut through rock and cobbles and whose beds rarely or never become mobile.

A particular channelbed environment will influence the design of a specific countermeasure type more than it will the selection of one of the broader groupings of countermeasure types. Specific countermeasures within each of the group classifications will function better under one channelbed environment than another. In general, the more permeable structures are better suited for regime and some threshold channel conditions, where they will cause the deposition of the bed material in transit. Conversely, impermeable structures often are better suited for use in channels having rigid or near rigid threshold channelbeds. Please note that these statements are not to imply that impermeable structures should not be used on regime channels, and permeable structures are not suited for use on rigid channels. Permeable structures have been used effectively on channels having immobile beds, and impermeable structures have been used on regime or mobile-bed channels. Each site must be evaluated on a case-by-case basis, and the relative importance of other criteria.

Channelbend radii are defined as small, medium, or large. Small bends have radii shorter than 350 feet; medium bends range from 350 feet to 1000 feet; large radius bends have radii greater than 1000 feet. The use of spur-type flow-control structures on short radius bends is usually not cost effective when compared with other countermeasures; this is due primarily to the short interspur spacing that is required. Also, short radius bends are typically found on small (in terms of width) channels; the consequences of using spurs on small channels already have been discussed.

Channel hydraulics encompass consideration of flow velocities and depths. Some specific countermeasure types are better suited for specific flow velocities and flow depths than others. As with other countermeasure criteria, there is little or no distinction in this area among the general countermeasure groups; specific countermeasure types within each group will provide protection for a specified range of hydraulic conditions.

Ice and debris conditions also affect the selection of a specific countermeasure type. Again, these criteria do not separate countermeasure types along major category lines. In general, the larger, more structurally sound countermeasure types are best suited to resist damage from floating ice and debris. The degree of flow obstruction created by the countermeasure also must be considered. For example, spurs or retardance structures constructed perpendicular or at sharp angles to the primary flow direction are more susceptible than other countermeasure types to damage resulting from impingement of floating ice and debris. This is particularly true of permeable structures that will act as skimmers and become partially or wholly blocked by the debris, resulting in increased hydraulic pressures on the structure. Floating debris also has been seen to cause severe damage when it becomes lodged on top of countermeasures as the water recedes after an event that had topped the structure.

SYSTEM IMPACTS

An additional criterion that should be considered during preliminary stages of selecting a countermeasure type is the river system impacts that can result from a particular design. System impacts can be environmental, esthetic, or related to the safe access and use of the river.

Environmental considerations include impacts on channel geometry, water quality, and aquatic and terrestrial plant and wildlife. As was discussed in Chapter 2, channelbank stabilization results in impacts on the channel's planview form and cross-sectional geometry. In Chapter 2, the natural downstream migration of meander bends was discussed. It is apparent from this discussion, it is the rate of meander migration that is important to the selection of a bank-stabilization scheme. The rate of meander migration depends on several factors, including the bed and bank material characteristics, amount of bank vegetation, and characteristic flow conditions. The true rate of meander migration can be estimated by an analysis of sequential aerial photography. The rate of bank recession is important for providing the appropriate level of protection for a crossing of given return period design. For example, if it is anticipated that the rate of downstream meander movement is sufficiently slow, the required design life of the bank-protection scheme would not dictate the same level of protection that would be required for a system experiencing rapid meander movement.

The general deepening of the channel cross-section within a stabilized bend also was discussed in Chapter 2 (see Figure 10). The magnitude of this channel deepening is influenced by the type and design of the particular countermeasure used. For example, streambank revetments and retardance structures generally produce less severe channel deepening than spur-type structures. Channel deepening also is a function of the relative amount of flow constriction produced by the stabilization scheme; the greater the flow constriction, the deeper any associated general bed scour. Another channel geometry impact occurs when the countermeasure design causes flow deflection towards an opposite bank. This has been found to be a primary problem where spurs, dikes, and retardance structures are misaligned.

Water-quality impacts result from changes in river turbidity, as well as other alteration of the local riverine habitat. Water-quality problems usually are associated with construction activities. Impacts on aquatic and terrestrial plant and wildlife also are primarily related to construction activities; these impacts result from the alteration, disturbance and elimination of riparian-zone ecosystems and biosystems. The construction methods for some countermeasures might be unacceptable if these are sensitive issues at a particular site.

Esthetic impacts relate to the overall appearance and general acceptance of a particular scheme. In urban areas, the public pressure for an esthetically pleasing design can be a controlling factor in the selection process.

Consideration also must be given to **access to and use of the river.** Structures such as longitudinal fence retards, some dikes, strings of concrete, or steel jacks and/or tetrahedrons limit access to the river from the bank. Jacks, tetrahedrons, and some spur-retardance structures also can pose safety hazards to boaters and sportsmen using the river. On the other hand, some structures can be designed to enhance river access and the recreational use of the river reach. Specifically, rock spurs, toe dikes, and some revetments can be designed to incorporate boat ramps and pedestrian access to waterways.

VANDALISM AND MAINTENANCE

Vandalism, particularly in urban areas, is a problem that must be dealt with when designing all bank-protection schemes. Both the U.S. Army Corps of Engineers (1981) and Keeley (1971) document cases of vandalism. Vandalism can render ineffective a technically effective bank-protection scheme. Vandals' efforts include dismantling; burning; cutting with knives, hatchets, and axes; etc. If vandalism is determined to be an important consideration, steps can be taken to reduce the vandals' chances of succeeding. For example, steel structural members could be used instead of wood, or the wood could be treated to eliminate or minimize the possibility of burning. Also, other structural types that are less susceptible to vandalism could be used, such as rock riprap structures.

Maintenance requirements also must be considered. All types of streambank protection will usually require some degree of maintenance. The need to repair a bank stabilization structure can result from vandalism or damage from excessive hydraulic conditions and/or ice and debris conditions. In general, the greater the structural integrity of the spur, the less susceptible it is to adverse flow and debris conditions. However, the dynamic nature of rivers makes it virtually impossible to predict all possible combinations of forces to which a bank-stabilization scheme will be subject. Also, it is not usually economically justifiable to build countermeasures that will resist all possible combinations of flow and debris impingement forces. Therefore, a regular program of inspection and maintenance is important to ensure economical, efficient, and reliable streambank protection. Of course, there will be an associated cost, which must be considered when evaluating alternative bank-stabilization schemes.

CONSTRUCTION-RELATED CONSIDERATIONS

Several considerations relating to the construction of bank-stabilization schemes have already been mentioned, including water-quality and channelbank conditions. Other construction-related factors influencing the choice of a countermeasure type include the following:

- required access and right-of-way,
- extent of bank disturbance,
- required construction methods,
- local availability of construction materials, and
- construction time and delays.

The impact these factors have on selection of a countermeasure depends on specific site conditions and the design details of each countermeasure being considered.

Constructing a bank-stabilization scheme will, in some cases, require the acquisition of a right-of-way on which to build the scheme and/or from which to build it. There also must be access to the site for the initial construction as well as to meet maintenance requirements. Some design schemes might require more extensive rights-of-way than others.

The construction requirements for different schemes will produce varying levels of bank disturbance. Schemes whose construction requirements necessitate excessive bank grading and clearing might not be desirable due to the high costs associated with these activities, the loss of the bank-stabilizing characteristics of natural vegetation, and increased environmental impacts (water-quality degradation, and disturbance and riverine biosystems and ecosystems.

Countermeasure designs that require special construction methods or techniques also may not be desirable. If local contractors are not familiar with or accustomed to the required construction methods, shortcuts that reduce the effectiveness of a particular scheme might be taken inadvertently, or the learning process might result in excessive project cost overruns.

Another construction-related consideration is the local availability of the required construction material. If a locally unavailable material is specified, the cost of the countermeasure might not justify its use.

Although not directly affecting the selection of a countermeasure type, construction delays are another important consideration when designing flow-control and bank-stabilization structures. Construction delays can reduce or nullify the effectiveness of stabilization measures even under the best design conditions. During the U.S. Army Corps of Engineers study of streambank stabilization (1981), several stream pattern changes occurred from the time of the design survey until construction. In one case, the low-water thalweg pattern reversed itself in a sine-cosine pattern between the preliminary surveys and actual construction. Even minor changes in channel pattern can alter the effectiveness of some stabilization schemes. Therefore, all possible measures should be taken to reduce the time between design and construction.

LEGAL CONSIDERATIONS

Legal considerations relating to streambank stabilization can be classified into two areas. First, there are the legal liabilities associated with stabilizing a bend. The effects of stabilizing a channelbend are discussed in Chapter 2. The potential upstream and downstream impacts discussed must be weighed against the potential for a lawsuit from a downstream landowner. Construction activities for the "general good of the community," however, are usually exempt from any legal action. However, a bank-stabilization scheme that will minimize any system impacts should be selected. This will reduce the possibility of any legal action.

The second legal consideration involves the use of a patented protection device, method of arrangement, or method of construction. California Department of Public Works (1970) provides a listing of "Landmark Patents in Bank and Shore Protection" that includes patents taken through 1956. Identifying active patents on streambank- stabilization devices was not a part of this study. However, streambank stabilization countermeasures that are known to be patented will be indicated as such when discussed in this report.

COSTS

As was mentioned previously, the bottom line in the countermeasure selection process is cost. The structure that provides the desired level of protection at the lowest cost will be the "best" for a particular site. The final cost of a streambank-stabilization scheme will depend on many features, including the following:

- countermeasure type and specific design,
- channel size and bank height,
- hydraulic conditions,
- right-of-way costs,
- site preparation requirements,
- local labor and material costs, and
- maintenance costs, etc.

Additionally, the economic importance of the crossing must be weighed. The importance of a highway crossing depends on the population of the area, the location of critical services near the structure (such as hospitals, fire companies, etc.) and the number and accessibility of other highway crossings. The technique used to evaluate these factors is known as risk analysis. This subject is covered in detail in Tseng, Kanpp, and Schnalz (1975), Snyder and Wilson (1980), and FHWA (1981).

A risk analysis will help to determine the level of protection justified at a particular site and in this way aid in the selection of an appropriate countermeasure.

Because of the limited number of sites for which cost data are available, any attempt to evaluate all of the factors influencing countermeasure cost would be speculative at best. Therefore, a more general approach will be presented. First, a cost comparison subdivided along common countermeasure type lines is presented. The cost of countermeasures built on the same river are then compared to give some guidance as to the relative costs of several countermeasures built under the same or similar conditions.

The primary source of data for this analysis came from sites documented by the U.S. Army Corps of Engineers during their Section 32 Program. The costs data used were compiled from Appendices A through F of the Corps' final report (1981) and the Soil Conservation Service's Inventory and Evaluation Reports (Michael Baker, Jr., 1980). Additionally, some site data were compiled from Brice et. al (1978), and personal contacts were made by the authors. All cost data then were adjusted to a 1982 average base cost by using the Engineering News Records (ENR) average yearly costs indices.

The following comparison of countermeasure costs is intended only to provide a relative comparison between the major countermeasure types for preliminary selection purposes. Because of price variances from one area of the country to another, no unit material prices will be given here; only prices reflecting a final construction cost. Also, it is recognized that the costs data used are not what one might call "statistically pure." One reason for this is the difference from one site to another in the method of reporting reporting cost data. Where they are given, countermeasure costs, are typically reported as a lump sum. At some sites, this lump sum includes only construction costs; at other designs, construction and maintenance costs are reported, and at others a lump sum cost that includes some construction activity not directly a part of the countermeasure construction is included. In all cases, every effort was made to adjust the reported costs to reflect construction of the countermeasure only. Also, the use of the ENR construction costs index to adjust all costs to a uniform base has undoubtedly introduced some error. The ENR index is known to vary regionally, and the average value used may not accurately reflect the true adjustment in a specific region. Another point to remember is that the analysis is based on available data and might be skewed to channel conditions characteristic to the sites where most of the data was available. Regardless of these factors, the analysis still provides a useful comparison of the relative magnitude of countermeasure costs.

Figure 49 indicates the results of the cost analysis for individual sites. A total of 515 sites was used in the analysis; of these, 48 were spurs, 201 were revetments, 149 were retardance structures, 105 were longitudinal dikes, and 12 were bulkheads. The number of individual countermeasure types is included in the figure next to the name. The bar following each countermeasure type represents the cost range found. The darkened portion of the bar represents the dominant data range. The dominant data range was computed by first computing the average cost and then two standard deviations; the standard deviation of the data falling above the mean and the standard deviation of the data falling below the mean. Adding and subtracting these values (respectively) from the mean yields the dominant data range. When a countermeasure type did not have more than five sites for analysis of the dominant data range, no dominant range was computed, and only the total range is shown.

A quick scan of Figure 49 reveals those countermeasures that are least expensive. Arbitrarily setting $100.00 per foot of bank protected as a cutoff point, Figure 49 indicates that rock riprap spurs, horizontal wood-slat spurs, rock windrow revetments, vegetation, jack retards, wood-fence retards, and rock toe dikes will usually be the least expensive. Figure 49 also indicates that Henson-type spurs, large permeable diverter spurs, cellular block revetments, and concrete-filled mats typically will be the most expensive schemes. Also, tire mattresses show the largest variance in cost.

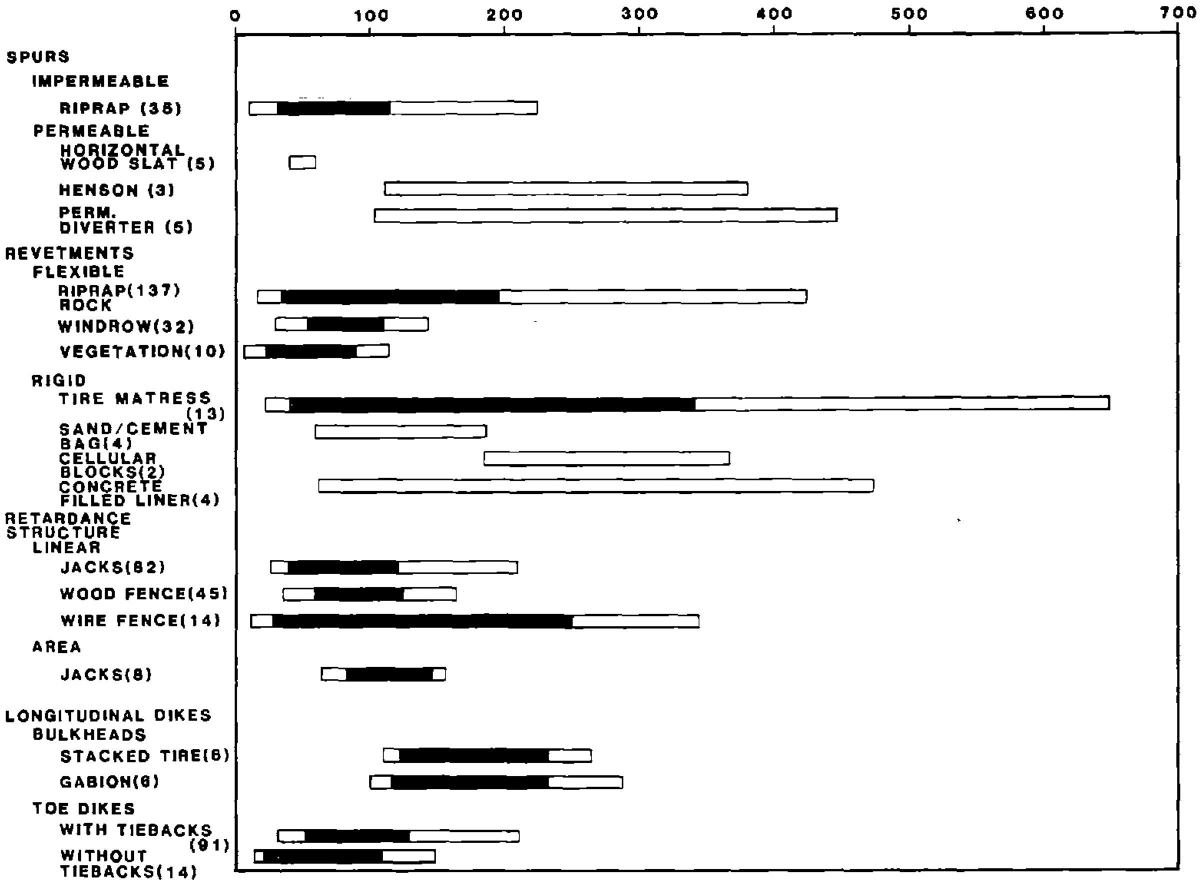


FIGURE 49. COSTS PER FOOT OF BANK PROTECTED

Because of the wide variation of site conditions included in the above analysis, a second analysis, comparing different countermeasures constructed under the same or similar environments, was conducted. Twenty-seven rivers were identified where cost data on more than one countermeasure type was available. This information is listed in Table 2. Review of this data provides a comparison of countermeasures that were designed to provide the same function, resist the same erosion mechanism, and function under the same hydraulic conditions. Note that in all instances of spur use except one, a spur scheme was the most economical per foot of bank protected. Of the spur types for which data was compiled, dumped rock riprap spurs were generally the least expensive. This is due to the almost universal availability of this material and the low labor costs associated with rock riprap designs. This comparison also reveals that rigid revetments and bulkheads are usually the most expensive of the schemes compared; of the rigid revetment types covered, cellular blocks were the most expensive; of bulkheads, cribwalls were the most expensive. Flexible revetments and retardance structures usually fell in the middle of the cost range at most sites with no real distinction between the two in terms of cost.

TABLE 2. COST COMPARISON BETWEEN COUNTERMEASURES ON THE SAME RIVER

RIVER / COUNTERMEASURE	Arkabutla Creek Mississipi	Big Sandy Creek Mississipi	Chilli Creek Mississipi	Chiwapa Creek Mississipi	Cypress Creek Mississipi	Hickahala Creek Mississipi	Hotophia Creek Mississipi	Hunter Creek Mississipi	Kings Creek Mississipi	Long Creek Mississipi	Oaklimeter Creek Mississipi	Red Banks Creek Mississipi	Missouri River: Cavins Pt. to Ponca	Missouri River: Dam to Lake Carson	Missouri River: Ft. Randall to Niobrara	Roanoke River Leesville, VA	Rio Chama Abiquiu, N.M.	Lower Chippewa River Eau. Clarp, WI	Little Miami River Milford, Ohio	Conn. River @ Haverhill & Northfield, Conn.	Allegheny River Pennsylvania	Eel River California	Perry Creek Mississipi	Batupan Bogue Mississipi	Tillatoba Creek Mississipi	Senatobia Creek Mississipi	Tallahatchie River Mississipi
SPURS																											
IMP:																											
Riprap													46		68			19			74	130		49[2]	40		
Gabion																	126										
Perm																											
Board Fence																								49[3]	53[3]		
REVET																											
FLEX:																											
Riprap Lining			164	269					68	135				61[1]		177	190		46		127[3]		32		29		78
Windrow													91	107	161	124					48						
Composite													87	119	121												
RIGID:																											
Sand/Cement Bags																				127[2]				117	139		
Cellular Blocks																		103		254[2]							
Gabion																				121[2]							
Tire Mat.																210				69[2]					46		
RET. STR.																											
Linear																											
Jack	90	57	78	120	27	69	136	44		62	67	56													59	54	73
Board Fence	130	79	130			72	60	42		155	66	100											79	217[3]	73	75	
Wire Fence			31		39			23	27	13								42				235					
AREA:																											
Jack										133																	
DIKES																											
Bulkheads:																											
Cribwalls																	240		306								
Tirewall																				184[2]							
Gabion																			179								
Toe Dikes:																											
w Tiebacks													100	135	144							147		206	74		
w/o Tiebacks																		51	215[2]		115		38	98	77		

[1]Lower bank protection.

[2]Lower bank protection with upper bank vegetative plantings.

[3]Includes a rock toe.

The above cost analysis provides some general evidence with respect to countermeasure costs. This information should be used along with the other countermeasure selection criteria as a preliminary evaluation tool. It should not be used to establish estimates of final construction costs for flow control and bank stabilization projects.

Chapter 5

APPLICATION OF FLOW CONTROL AND STREAMBANK STABILIZATION STRUCTURES

This chapter discusses the attributes and disadvantages of the most common flow-control and streambank-stabilization structures as presented in Chapter 3. The criteria for the evaluation and selection of a countermeasure type introduced in Chapter 4 are used as the basis of comparison in this chapter. Some design information is also included. In most cases however, the reader is referred to reports and other documents containing more detailed design information than that included here.

Much of the information contained in this chapter is based on a thorough review and analysis of reports and other documentation resulting from the U.S. Army Corps of Engineers' Streambank Erosion Prevention and Control Demonstration Program (U.S. Army Corps of Engineers, 1981). This Program (often referred to as the Section 32 Program) consisted of an evaluation of the nationwide extent of streambank erosion; an evaluation of existing bank protection methods; hydraulic research on the effectiveness of bank protection methods; research on soil stability; identification of the causes of streambank erosion; and the design, construction, monitoring, and evaluation of more than 115 bank-stabilization schemes throughout the United States. The intent here is to make the wealth of information documented during the Section 32 Program available to highway engineers and planners in a concise and more usable fashion.

SPURS

Previously, spurs were defined as permeable or impermeable linear structures that project into the channel for the purpose of altering flow direction, inducing deposition, and/or reducing flow velocities along a channelbank. A detailed coverage of the applicability of spur-type structures is presented in FHWA (1983b). The following is a summary of the application criteria for spurs as reported in FHWA (1983b).

Spur-type structures can be used for any of the functions or purposes outlined in Chapter 4. They provide a particular advantage over other countermeasure types, however, in providing flow control and constriction as well as the reestablishment of a previous or new flowpath. The erosion mechanism countered best by spurs is bank-particle displacement caused by abrasion and streamflow-induced shear stresses. By diverting flows away from the channelbank, spurs are also effective at removing the transporting mechanism that drives the erosion process.

Several comments can be made regarding the application of spurs in various river environments. With respect to channel size, spur-type structures are not well-suited for use on small-width (less than 150 feet) channels. On these narrow-width channels, spur designs often will create excessive flow constriction at high streamflows and cause current deflections towards the opposite bank. Also, the use of spur-type structures for flow control and bank stabilization on short-radius bends (less than 350 feet) is usually not cost effective when compared to other countermeasure types. With respect to channelbank height, spurs are best suited for the protection of low- (less than 10 feet) to medium- height (from 10 to 20 feet) banks. Protecting high banks with spurs often requires special design considerations.

Additional advantages to the use of spur systems have been identified as follows:

- Spurs often do not require extensive bank reshaping or grading prior to construction, therefore making them well-suited for use along steep-cut banks where significant site preparation might be required of some other countermeasure types.

- The use of spurs is not adversely effected by irregular banklines; spurs can be used to create a smooth bankline without excessive site preparation.

- The use of spur-type structures has been found to provide an enhancing influence on bank vegetation since the structures shift flow currents away from the immediate vicinity of the channelbank.

- The most important single advantage provided by spurs is that spurs often will provide a significant economic advantage over other countermeasure types for flow- control and bank-stabilization purposes (see Figure 49 and Table 2).

Several potential drawbacks to the use of spur-type structures are:

- Design errors in the geometric layout of spur systems can have severe impacts on channel geometry. For example, misalignment of spurs can cause severe flow deflection and initiate an erosion problem on the opposite bank. Also, if the spurs produce too much flow constriction, excessive channel deepening may occur, which can undermine downstream structures in the channel (such as bridge piers), and cause the eventual failure of the structure itself.

- Some spur-type structures can cause potential hazards to recreational users of the river. They pose particular dangers to boaters and can also be a potential hazard to children who might find spurs attractive structures to play on or around.

As discussed previously, there are three main categories of spurs. They are retardance spurs, retardance/diverter spurs, and diverter spurs. The merits of each of these designs will be discussed briefly in the following sections. Specific advantages of and design guidelines for individual spur types are presented in FHWA (1984); therefore, they will not be discussed here. However, the applicability of each of the three major spur types will be covered below.

Retardance Spurs

As discussed previously, retardance spurs are designed to reduce the flow velocity in the vicinity of the channelbank as a means of protecting the bank from erosion. As mentioned, there are two primary types of retardance spurs: fence type and jack/tetrahedron type. As illustrated in Figures 14 through 17, these spurs are usually light structures; as such, they are not well- suited for extremes in environmental conditions.

Retardance-type spurs function best at protecting existing bank-lines as opposed to diverting flows to create some new flow alignment. However, wire fence, and jack/tetrahedron spurs have been used to reestablish previous flow alignments where only a minor shift in flow orientation is necessary. Unless special allowances are made, retardance-type structures will usually only provide protection to the toe of the streambank, and therefore, are not effective for upper-bank protection.

Permeable retardance spurs have been found to be particularly effective in regime/low threshold environments. In fact, they generally provide an advantage over other spur types in these environments. The flow retardance created by retardance spur schemes creates a depositional environment within the retarded flow zone along the channelbank for the suspended- and bed-sediment loads carried by these channels. This produces a sediment berm or bench that will stabilize the base of the channelbank. Also, by lowering flow velocities in this zone, permeable retardance spur schemes will reduce or eliminate the transporting ability of channel flows adjacent to the bank. This is important in cases where erosion resulting from bank-weakening mechanisms (wave erosion, subsurface flow and drainage, etc.) is occurring.

As discussed above, retardance spurs are best suited for regime and low-threshold sediment environments. Within these environments, however, retardance spurs have not been successful in high-velocity environments (> 8 fps), or some of the higher medium-velocity environments (> 6 fps). In these environments, retardance spurs do not provide sufficient flow retardance and are often undermined or outflanked by the unstable nature of the channel boundary. This is particularly true for jack and tetrahedron structures, which should not be used in the higher medium- or high-velocity environments. Retardance spurs are also smaller and less structurally rigid than other spur types; therefore, they are more susceptible to structural damage in high-velocity environments than other types of spurs.

Retardance-type spurs are best suited for the protection of the lower portions of the channelbank (often referred to as the bank toe). This makes them best suited for the protection of low- to medium- height channelbanks. When used to protect some medium and high channelbanks, retardance spurs have had a tendency to be outflanked at the bank end. This disadvantage can be overcome in some cases by increasing the structure height and ensuring that the retardance-spur structures are adequately tied to the channelbank to prevent or minimize the potential for outflanking. Although spur-type structures are generally not well-suited to protecting high banks.

With respect to channelbend radius, the more passive, permeable retardance structures perform as well as other spur types on large-radius channelbends (> 1000 feet). This statement can be extended to include some of the larger medium-radius bends as well (> 600 feet). However, smaller radius bends (< 350 feet) require a more positive flow control, and retardance-type spurs become less acceptable.

Retardance spurs function best when there is light debris present to reduce the permeability of the structures and enhance their flow-retardance qualities. However, large debris and ice will damage these light structures and render them ineffective. This is particularly true of the wire-fence and jack/tetrahedron designs. The wire-fence and jack/ tetrahedron designs have also been found to be less effective than other spur types in minimal debris environments. Without light debris to clog or block partially the structural frames of some of these structures, they do not provide sufficient flow retardance to protect the channelbank adequately.

Cost data was generally unavailable for retardance spur installations with the exception of the Henson-type wood-fence spurs (see Figure 14). The costs reported (as indicated in Figure 49 and Table 2) ranged from $110/foot to $380/foot. All sites where costs were reported were on medium- width channels with medium to high banks. They all also had moderate channelbend radii. However, all Henson spur installations consist of the same components and protect only lower portions of the bank. Therefore, bank height is not a significant consideration. The component primarily responsible for the cost variance reported was spur spacing. Spacings reported ranged from 40 to 100 feet. Costs reported for sites having spur spacings from 40 to 50 feet ranged from $300/foot to $380/foot; at the other end of the scale, schemes having 100- foot spacings had reported costs in the neighborhood of $110/foot to $150/foot. Although less expensive, the schemes designed with 100-foot spacings have not been as effective at stabilizing channelbanks as the 40- to 50- foot spacings.

Retardance/Diverter Spurs

Retardance/diverter spurs are permeable structures that are designed to function by retarding flow currents along the channelbank and providing flow deflection. This combination of functions makes them the most versatile of all spur types. Retardance/diverter spurs have been further classified as light fence structures and heavy diverter structures. These classifications generally separate the retardance/diverter structures by size and degree of permeability. In general, the light fence structures are smaller and more permeable than the heavy diverter structures. Retardance/diverter spurs are generally oriented with a downstream angle to enhance their flow-diversion qualities. Typical retardance/diverter spurs were illustrated in Figures 18 through 23.

Retardance/diverter spurs have been used effectively to protect an existing bankline, to reestablish some previous flow alignment, and to provide flow constriction. As is the case with retardance structures, retardance/diverter structures function by producing a flow retardance along the channelbank. They are also designed to produce a diversion of flows. The heavier diverter-type retardance/deflector spurs have been found to provide an advantage over other types of permeable structures where flow constriction and/or the reestablishment of some previous flow alignment are primary concerns.

With respect to channelbed environment, retardance/deflector spurs have functioned well in both regime/low-threshold and medium-threshold environments. However, because of their flow deflection characteristics, they are best suited for medium-threshold environments. This is particularly true of the larger heavy diverter structures. Local scour problems associated with these larger structures have resulted in structural undermining in some cases when they are used in regime/low-threshold environments.

Like retardance spurs, retardance/deflector spurs are subject to undermining and outflanking in high- velocity environments. However, because they divert channel flows and provide flow retardance, they have been effective in higher velocity environments than retardance spurs. Retardance/deflector spurs are also more structurally rigid than retardance spurs; therefore, they can withstand higher flow forces. However, the extremely permeable retardance/ diverter spurs (such as the welded wire mesh structures illustrated in Figures 19 and 20) should not be used in the higher medium- and high-velocity environments because they will not provide sufficient flow retardance.

Although spur-type structures are generally not well-suited for protecting high channelbanks, some large retardance/deflector spurs have been found to be adaptable to these conditions. This is because their structural design extends up and into the channelbank. See Figure 20 for an example.

Because of their flow-deflection qualities, permeable retardance/deflector spurs have been used effectively on both large- and medium-radius channelbends. Because of their permeability, however, they have not been as effective as impermeable deflector spurs on small-radius bends.

Retardance/deflector spurs have been used successfully in most debris and ice environments. Like retardance spurs, the presence of light debris enhances the effectiveness of retardance/deflector spurs and makes them particularly adaptable to environments where light debris is present. Because of their flow-deflection qualities, these structures have also been moderately effective in minimal debris environments. The large structural size of heavy diverter spurs makes this type of retardance/diverter acceptable in large debris and ice environments as well. However, some of the lighter fence-type retardance/diverters are susceptible to extensive damage in environments characterized by large debris and ice.

Cost data were found for four of the retardance/diverter spur types (see Figure 49 and Table 2). Data for the board-fence structures (similar to the spur illustrated in Figure 16) were reported by the U.S. Corps of Engineers (1981). Five installations were reported having an average cost of $51/foot. These structures were on small- to medium- width channels with medium-height banks and mild channelbends. They were constructed at 100-foot spacings and had lengths of approximately 25 feet.

The other retardance/diverter structures for which cost data were available all were heavy diverter structures. Two steel-pile and welded-wire fence structures were documented on the Soldier River by Brice, et. al (1978) (similar to those illustrated in Figures 19 and 20). The average reported cost for these structures was $230/foot. The Soldier River is a medium-width channel with medium to high channelbanks. The structures were placed on meandering channelbends. Structure length was about 110 feet with a interspur spacing of 110 feet. These structures are designed to protect the entire bank height.

Cost data also were available for several timber-pile retardance/deflector spurs. The costs ranged from $295/foot to $445/foot. These structures were all on medium-width channels with medium to high channelbanks and moderate channelbends. Spur spacing ranged from 130 feet to 450 feet; spur lengths ranged from 55 feet to 150 feet. The two designs for which cost data were available were pile structures with timber piles as horizontal members (see Figure 21), and timber-pile structures with wood-plank sheathing as horizontal members (see Figure 23). The cost of the timber- pile structure with horizontal-pile stringers was $445/ft.; the average cost of the timber-pile structure with wood-plank sheathing as horizontal members was $332.50/foot.

Diverter Spurs

Diverter spurs (alternately referred to here as deflector spurs) are impermeable structures that are designed to function by diverting the primary flow currents away from the channelbank. Several diverter spurs were illustrated in Figures 24 through 26. The two primary subclassifications of diverter structures are hardpoints and transverse-dike spurs. The primary difference between these two types of diverter spurs is the structure's length.

Impermeable deflector spurs function by deflecting the main flow current away from the bank. Like retardance/deflector spurs, they have been found to provide an advantage where flow constriction and/or the reestablishment of some new or previous flow alignment is desired. They are also as effective as other spur types when the primary function is to protect an existing bank-line. Impermeable deflector spurs have also been quite effective at countering erosion caused by abrasion. Impermeable diverter spurs have two advantages over other spur types in this area. First, impermeable diverter spurs function by deflecting currents and any floating debris away from the channelbank. Second, impermeable structures also have more structural mass than most permeable structures and therefore, are subject to less damage from floating debris.

With regard to channelbed composition, impermeable deflector spurs are best suited for use on high threshold/rigid channels. They have been used effectively, however, in some regime and low-threshold environments.

When using impermeable deflector structures in alluvial environments it is important to recognize their potentially detrimental impacts. Flow concentration, which is inherent in impermeable spur design, and local scour are the most common of these impacts. A consequence of the flow-constricting effect produced by spurs is a concentration of flow lines along the riverward tip of each spur. The flow concentration in this area results in a magnified potential for erosion of the channelbed in the vicinity and just downstream of the tip of the impermeable structures. This condition is much more pronounced in high-velocity environments and around sharp bends than low-velocity environments and around mild bends. The occurrence of significant erosion at and downstream from the spur tip has been observed by the authors at numerous field sites and is well documented in reported laboratory studies (FHWA, 1983a; Ahmad, 1951a and 1951b). Local scour is a primary concern in alluvial environments because of the highly erosive nature of the gravel-, sand-, and silt-size material comprising the channelbed. The potential for excessive erosion at the spur tip, combined with the high cost of providing protection against the erosion is a drawback to the use of impermeable diverter spurs in alluvial environments.

The flow concentration and local scour conditions just described are characteristic of impermeable installations in all river environments. In high threshold/rigid channels (those cut through large gravel- and cobble-size materials), however, these conditions pose less of a threat to the stability of impermeable spur schemes. Flow concentration at the spur tip will still cause erosion in these environments. Because of the low transportability of the coarse materials making up the channelbed and the natural channelbed armoring that occurs in these environments, however, it will be of a much smaller magnitude. In most cases, only a limited amount of erosion (in comparison with truly alluvial environments) will occur. This erosion can usually be anticipated and control structures can be adequately designed at little additional cost.

With respect to the channel's flow-velocity environment, deflector spurs have been found to be effective over a wider range of flow conditions than other spur types. Because of their structural rigidity, impermeable deflector spurs are the least susceptible to damage in high-velocity environments than other spur types. For this reason they are considered to be applicable for low-, medium-, and high- velocity environments (velocities < 4 fps, < 8 fps, and > 8 fps respectively). It must be remembered, however, that they are subject to limitations in regime and low-threshold sediment environments.

With respect to channelbend radius, impermeable deflector spurs provide an advantage over other spur types on both medium and small channelbends. This is primarily due to their capacity as positive flow-control structures. On extremely small radius bends (bend radii less than 350 feet) the larger transverse dike impermeable structures will cause excessive flow constriction and scour problems that will make them unacceptable. Impermeable hardpoint spurs have, however, been used effectively on some channelbends less than 350 feet in radius because they do not cause a significant flow obstruction.

Impermeable deflector spurs have been used effectively in all categories of debris and ice environments. They provide a significant advantage over other spur types in large debris and ice environments. Impermeable deflector spurs divert much of the floating debris instead of skimming it from the surface as do permeable structures. Also, their structural mass makes them less susceptible to damage than the lighter permeable structures. This does not, however, imply that they will not be damaged by floating debris, only that the damage will be less severe.

Cost data were also available for diverter spurs. Costs for riprap hardpoints (see Figure 12) ranged from \$13/foot to \$110/foot. The primary factor affecting the reported costs is hardpoint spacing, which is dependent on channelbend radius. Other factors influencing the cost of these structures are site preparation and bank height. The low end of the reported range was for hardpoints spaced at 100 feet and having lengths of 68 feet. The \$110/foot hardpoints were designed with 100-foot lengths, spaced at 40 feet on mild channelbends in channels having large widths and medium bank. A comparison of these costs indicates that hardpoint spacing is one of the important design parameters that must be defined.

Costs for both gabion and riprap diverter structures were reported. The costs reported for gabion spur installations ranged from \$32/foot to \$126/foot. The low end of the scale was for 10-foot long spurs in a small channel with low channelbanks. The higher cost was reported for 25-foot long spurs on a medium-width channel with low channelbanks. Both ends of the cost range reported were documented on channels having sharp bend radii. No cost data were reported on channels having mild bends or medium to high channelbank heights. Also, cost data were not reported for larger structures. Cost data for large riprap diverter structures ranged from \$50/foot to \$226/foot. Here again, a major factor reflected in the cost range is the spur length and spacing.

REVETMENTS

Channelbank revetments are defined as armor layers that are supported by the channelbank. The primary function of revetments is to protect an existing bankline. Revetments are the most commonly used bank-protection devices, primarily because if properly sized, they almost always will prevent erosion caused by abrasion and scour, regardless of river gradient or velocity. Revetments also are used, however, as parts of dikes designed to reestablish a previous flowpath and/or control or constrict channel flows. As outlined in Chapter 4, Table 1, revetments can be used to counter all bank weakening and particle-displacement erosion mechanisms. The only erosion mechanism they do not affect is the transport capability of the flowing water.

Additional advantages of revetment systems have been identified as follows:

- Because revetments are bank linings and do not project into the channel, they do not produce a constriction of channel flows, and they do not affect flow patterns significantly in the bend.

- Since revetments pose no obstruction to flow patterns, they are less susceptible to debris and ice damage than other countermeasures.

- Revetments pose no safety hazards to boaters or waterskiers and can be used to create boat-launching facilities.

- Revetments of natural material are esthetically pleasing in appearance and therefore are acceptable in recreation areas.

- Revetments can be easily constructed with land-based or floating plants.

- Economy is served because a wide range of revetment types can be easily adapted to most sites.

Several potential drawbacks to revetment systems also have been identified. They are as follows:

- Protection of steep, high banks with revetments can require extensive bank preparation (grading and excavation), which often will require acquisition of extensive right-of-way.

- The protection of high banks with revetments also will require large volumes of revetment material.

- Most revetment schemes require extensive bank clearing, which leaves bank soils bare and susceptible to erosion during construction. Stripping natural vegetation also can weaken the existing bank structure.

- General channel deepening and bank-toe scour that normally accompany bank-stabilization activities require special consideration in revetment toe design along the entire length of channelbank.

- Revetments rely on the channelbank for support, and therefore, must follow an existing bankline.

The most common causes of bank-revetment failures are undermining of toe material and loss of the supporting bank material resulting from excess seepage pressures. Toe protection can be provided by anticipating local scour depths along the bank toe and designing the base of the revetment accordingly. The loss of bank material resulting from excess seepage pressures can be countered through adequate filter design and/or subsurface drainage systems. In the following sections, both of these items will be discussed along with design information for specific revetment types.

Flexible Revetments

Revetment designs can be further classified as either flexible or rigid. Flexible revetments provide the following advantages over rigid designs:

- They will adjust to minor shifts in underlying bank material and therefore will not fail completely as a result of minor undermining.

- They can be more effectively repaired after damage.

- By virtue of form, flexible revetments are less susceptible to damage by distortion and actually are intended to function after moderate displacement.

There is only one disadvantage in the use of flexible revetments. When these revetments are supported by sandy or silty bank soils, they may require either a granular or fabric filter material; these filters will protect against failure caused by erosion of the supporting ground through the interstices of the revetment by subsurface flows.

Flexible revetments are pervious in design and as such, have two principal functions, i.e., their function as filters that allow for the passage of groundwater but prevent the passage of underlying particles, and their function as protective layers that can resist the impact of currents and waves. These functions make flexible revetments suitable for most erosion mechanisms.

Riprap Revetment

Dumped riprap is the most widely used type of revetment in the United States. Its effectiveness has been well established where it is properly installed, of adequate size and suitable size gradation. Riprap materials include quarry-run, rubble, or other locally available materials. Rubble consisting of concrete-waste and rock-spoils is available in areas undergoing widespread urban renewal projects involving the demolition of buildings. Although it is somewhat unsightly, rubble can provide inexpensive short-term protection where it is available. Steel-furnace slag is another riprap material available in the vicinity of steel smelting plants.

Where stones or other materials of sufficient size are available, riprap usually is a primary contender among the bank- protection methods for the following reasons:

- A riprap blanket is flexible and is not impaired or weakened by slight movement of the bank caused by settlement or other minor adjustments.

- Local damage or loss is easily repaired by the placement of more rock.

- Construction is not complicated, and no special equipment or construction practices are necessary.

- Appearance is natural, thus acceptable in recreational areas.

- If riprap is exposed to fresh water, vegetation will often grow through the rocks, adding structural value to the bank material and restoring natural roughness.

- Riprap is recoverable and may be stockpiled for future use.

One drawback to the use of riprap revetments is that they are more sensitive than some other bank-protection schemes to economic factors. For example, freight/haul costs can significantly affect the cost of these revetments. This fact was born out by the cost analysis reported on in Chapter 4. Costs of riprap installation ranged from $15.00 to $425.00 per foot of bank protected.

The design of riprap revetments is described in detail in Searey (1967), California Department of Public Works (1970), Norman (1975), Maynard (1978), and Simons et al. (1980). More recent studies by the U.S. Army Corps of Engineers (1981) have provided additional design information in the following areas:

- thickness of protection,
- face slope,
- elevation of top of protection,
- toe protection, and
- need for and design of filter material.

These items will be discussed briefly in the following sections.

THICKNESS OF PROTECTION: Riprap stability increases with riprap blanket thickness because additional protective material is available to armor areas that might otherwise be exposed. If the stone is thicker, however, the design will be more expensive. Field studies on a wide variety of river and channel types have been conducted by the C.O.E. (1981) to study determine riprap blanket thicknesses. Their findings indicated that the thickness of riprap should be 1 to 1.5 times the maximum diameter of the largest stone used in the blanket, or 2 times the average stone diameter; whichever is greater.

FACE SLOPE: Based on field experience, the face slope of any riprap design should never exceed 1.54H to 1V; it is recommended that a slope of 2.0H to 1V be used to provide a better factor of safety against slippage failures.

ELEVATION OF TOP OF PROTECTION: The height of structural protection on a bank slope should be at least equal to the river stage with a one-year recurrence interval. In areas characterized by large fluctuations in stage, a higher elevation should be used. In areas subjected to severe wave action, wave heights also must be considered.

TOE PROTECTION: Undermining of revetment-toe protection was identified earlier as one of the primary mechanisms of riprap revetment failure. To counter undermining, a rock toe should be placed along the entire length of the base or toe of the revetment. It has been found that the volume of material required to protect the revetment toe adequately is equal to 1 1/2 times the volume that would be required to extend the slope protection to the expected depth of degradation. The placement of the toe material also has been investigated. After field evaluation of several toe configurations (see Figure 50, a through d), it was found that the thick, narrow, horizontal blanket in Figure 50d was clearly superior to that of other designs. Figure 51 illustrates a typical rock-riprap revetment design. Note that the two optional toe configurations are included. The toe protection can be mounted in the form of a low dike if the required volume of toe material cannot be otherwise accommodated. Trenching is another way to accommodate large volumes of toe material. Toe trenches, however, which require three or more feet of excavation below the water surface, usually will be prohibitively expensive.

NEED FOR AND DESIGN OF FILTER MATERIAL: Filter materials have been found to be necessary to stabilize riprap protection over noncohesive bank material subject to significant subsurface drainage conditions. These conditions exist in streambanks of noncohesive silts and sands that are subjected to frequent fluctuations in water surface, or are in areas of high groundwater levels. Where groundwater levels are low and the duration of high stages is short, a filter may not be cost-effective. Therefore, designers should not automatically specify expensive filter material, particularly when the risks and consequences of minor loss of bank material through the riprap cover are small. Filter material can be composed of either cloth or granular bedding material. When filter fabric is used instead of granular material, care must be taken to ensure that the fabric is not punctured and that the sides and toe of the filter fabric are entrenched or otherwise sealed to the bank. These measures will prevent leaching of the bank material in these areas. Also, it is necessary to ensure that properly sewn, overlapped, or welded seams are used to prevent leaching. Tests indicate that noncohesive streambank material tends to migrate downslope beneath the filter fabric when exposed to wave and/or seepage flow conditions. The downslope movement of streambank material did not occur beneath the granular filters, thus indicating the superiority of granular filters over filter fabric. Granular filter design is described in Searcy (1967).

Windrow Revetments

Windrow revetment is an erosion-control technique that consists of burying or piling a sufficient supply of erosion-resistant material below or on the existing land surface along the bank, then permitting the area between the natural riverbank and the rock windrow to erode until the erosion reaches and undercuts the supply of rock. As the rock supply is undercut, it falls

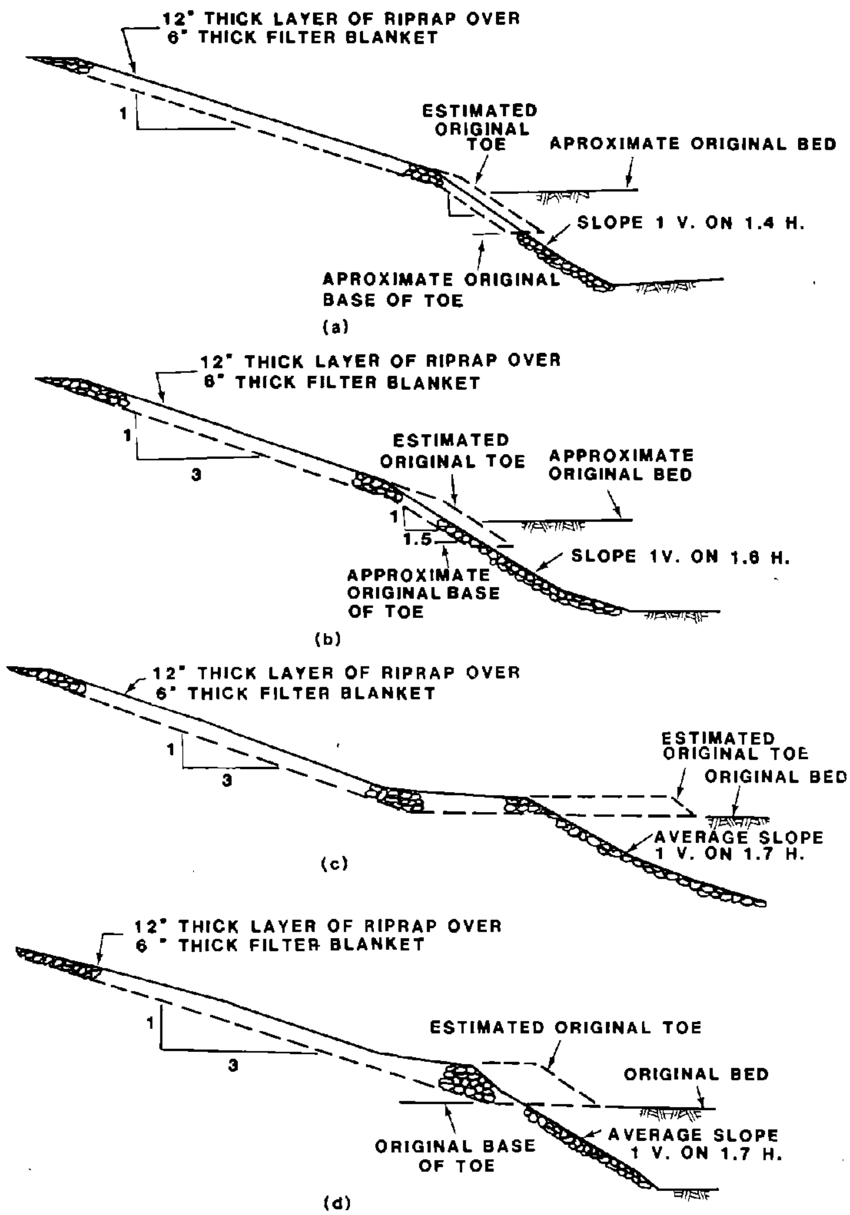


FIGURE 50. REPUBLICAN RIVER AT MILFORD DAM OUTLET CHANNEL, KANS. CROSS-SECTIONAL VIEW OF A) TEST SECTION 1, B) TEST SECTION 2, C) TEST SECTION 3, AND D) TEST SECTION 4 (SHOWN AS CONSTRUCTED IN 1969 AND AS SURVEYED IN 1974). (AFTER U.S. ARMY CORPS OF ENGINEERS, 1981)

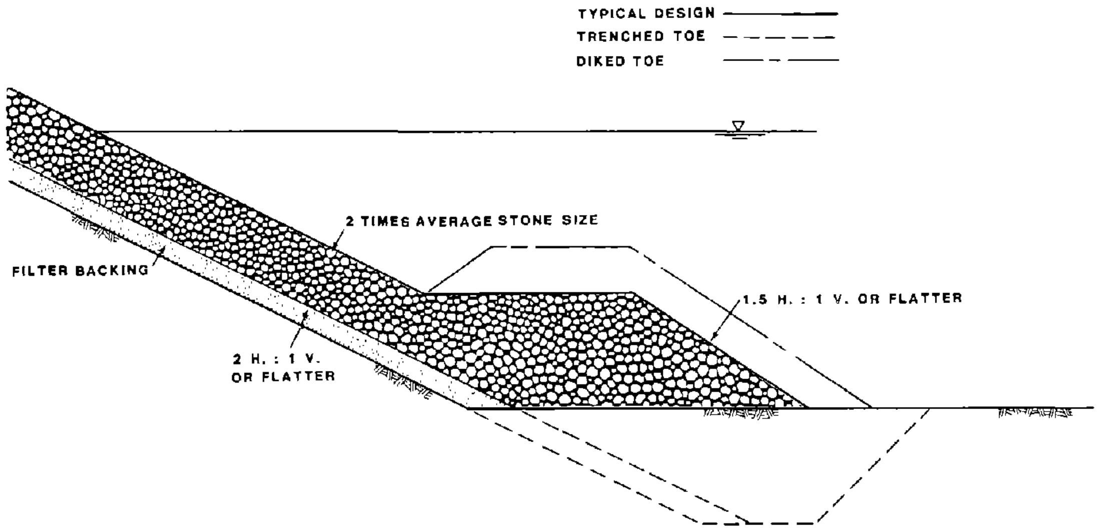


FIGURE 51. TYPICAL ROCK RIPRAP REVETMENT.

onto the eroding area, thus giving protection against further undercutting, and eventually halting further landward movement. Figure 52 illustrates the design concept of windrow revetment. Figure 52(A) shows a windrow trench-placement, and Figure 52(B) illustrates the launching of the stone material from a mound on the bank surface. In reality, the formation of this type of revetment is complex. Initially, the lateral erosive force of the stream undermines the windrow stone, causing some of the stone to drop into the stream. This stone slows the lateral erosion of the bank, but causes an increase in the vertical erosion along the leading edge or toe of the newly formed revetment. The initial quantity of stone that drops into the stream forms an unstable revetment that is constantly adjusting itself during the vertical-erosion process as the toe of the revetment advances into the scour area. As this process continues, revetment eventually will stabilize, halting the lateral erosion.

Windrow revetments can be used to protect an existing bank line against toe erosion and wave attack on lower portions of the bank. The object of the revetment is to protect the lower bank from the erosive force of the water; not to armor the entire bank. Windrow revetments will not adequately protect against surface or wave erosion on the upper portions of medium-to-high banks. They also are ineffective against seepage or other subsurface-flow erosion mechanisms. The windrow technique lends itself particularly well to the protection of adjacent wooded areas or placement along stretches of presently eroding irregular bank lines. Windrow revetments have been more successful on mild bends than on sharp ones. The long-term effectiveness of windrow revetments has not been adequately demonstrated as of this writing because of their relatively short history of use. They have proven

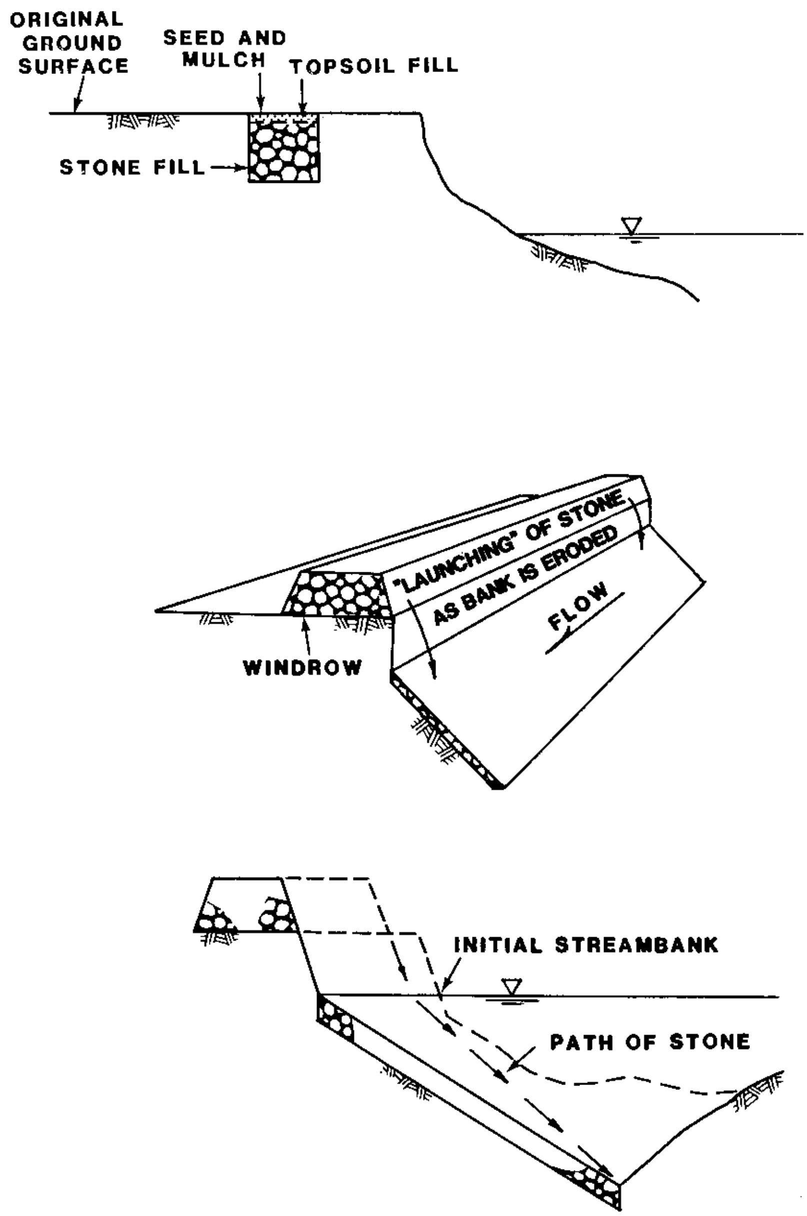


FIGURE 52. WINDROW REVETMENT DESIGN SKETCH; a) TRENCH-FILL DESIGN, b) LAUNCHING OF MOUNDED SCHEME. (MODIFIED FROM U.S. ARMY CORPS OF ENGINEERS, 1981)

themselves, however, as an effective temporary or emergency protection. Windrow revetments also are relatively inexpensive. On the Roanoke River at Leesville, Virginia, rock windrow revetments were about 32 percent cheaper than conventional rock revetments and 41 percent cheaper than tire-mat revetments. In Chapter 4 it is illustrated that windrow revetments are among the cheapest bank-stabilization designs available.

Windrow revetments have the following additional advantages over other more conventional methods:

- Construction procedures are simple, and specialized equipment is not required.
- Required construction time is short.
- Windrow revetments do not require bank grading.
- Hazardous bank-line erosion sites can be protected without risking the safety of personnel during construction.
- Construction can take place during high river stages (as long as they do not top the bank).
- Windrow revetments can effectively stabilize irregular bank lines.
- Manipulation of the stone is reduced.
- If the stone supply in the original windrow is not adequate, additional stone can be efficiently added.

Disadvantages associated with windrow designs include:

- Additional minor bank line erosion loss must occur to allow stone material to displace to the underwater bank area and function as desired.
- Construction may require more top-of-bank clearing than other structures. Land construction equipment requires a minimum clearing of 50 feet or more on the overbank to permit adequate structure placement.

There are three principal components of windrow-revetment design. These are the stone volume or application rate, the stone size, and the windrow cross section. The stone volume required to achieve a stable condition is dependent on the bank height, anticipated erosion depths along the streambank, and the magnitude of stream velocities. The flow velocity and characteristics of the stream dictate the size of stone that must be used to form a windrow revetment. Research has indicated that as long as the stone size used in the windrow is large enough to resist erosion on its own, it will perform satisfactorily (U.S. Army Corps of Engineers, 1981). It also was found that larger stones require more tonnage than smaller stone sizes to produce the same revetment thickness. The use of all well-graded stone is important to ensure that the revetment does not fail from leaching of the underlying bank material. Various windrow slopes have also been investigated and a rectangular cross section was found to be the best windrow configuration. This type of windrow is most easily placed in an excavated trench of the desired width. The second best windrow shape was found to be trapezoidal. This shape provides a steady supply of stone to produce a uniform blanket on the eroding bank line. A triangular shape has been found to be the least desirable. Additional design information for windrow revetments is found in Appendix H of U.S. Army Corps of Engineers (1981).

Rock and Wire Mattress

Rock and wire mattress revetments consist of flat mats of wire mesh fencing that are filled with rock and fastened together. Provisions are made for adequate anchorage to the embankment (see Figure 35. By definition, the mattress must have a thickness no greater than one foot. This distinguishes rock and wire mattresses from gabions, which are more equidimensional. Rock and wire mattresses are only semiflexible. They will flex with bank- surface subsidence, but if excessive subsidence occurs under the center of the mattress, it will span the void and fail if the mat connections do not provide sufficient tensile strength.

The application of rock and wire mattresses is similar to that of other revetments. However, their economic use is limited to locations where the only rock available economically is too small for rock slope-protection, or where grouted protection is unsuitable because of the fineness of the stone or bedding or foundation insecurity. In addition, the performance of rock and wire mattresses is aided by an arid climate. Corrosion in arid regions is slow, and many of the streams are ephemeral; thus, the wire mesh is not subject to continuous abrasion by sediments. Variations such as wire strength, mat thickness, and compartmentalization make it adaptable to a wide range of hydraulic conditions. Rock and wire mattresses also are better suited for bank surface protection than for toe protection; this will be discussed in a later section.

The primary advantages of rock and wire mattresses are:

- flexibility,
- the ability to use smaller, more readily available rock material, and
- the mattresses allow heavy regrowth of vegetation through the stone and wire mesh. This vegetation can provide an additional stabilizing influence, even when the wire mats have been corroded or otherwise destroyed. Revegetation also enhances the appearance of the protective scheme.

Additionally, rock and wire mattresses have been found to provide a flexible toe protection for other types of embankment armor. The purpose is to provide a mat that will extend from the embankment slope into the streambed, ready to adjust itself by flexure and subsidence to block the progress of erosion and scour, which might threaten the toe of the embankment. For deep, soft, streambeds, the mattress can (in some cases) be an economical alternative to a deep foundation. The disadvantages of rock and wire mattresses are listed below:

- Corrosion and abrasion damage are common problems plaguing the wire baskets.
- Labor costs associated with fabricating and filling the wire baskets make them more expensive than standard stone protection.
- They are less flexible than standard stone protection.
- They are more difficult and expensive to repair than standard stone protection.

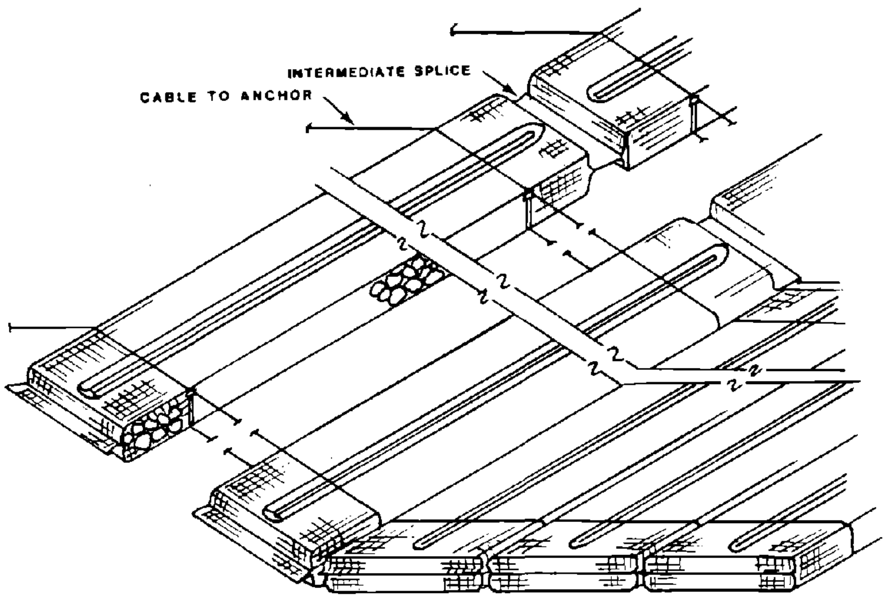
A primary failure mode for rock and wire mattresses is the undermining and subsequent failure of the rock toe-protection. This leads to the conclusion that the mattresses are not as flexible under field conditions as might be desired. Therefore, they are not a good choice where excessive toe scour is a primary cause of bank erosion, unless some other toe protection is provided as well. Also, their frequency of failure on channelbends has led to the recommendation that they be used only on tangent reaches.

Little information is available relating to the design of rock and wire mattress revetments. Figure 53 provides a typical design sketch including extension of the blanket for toe protection. In general, the same filter material requirements presented for standard rock revetments apply to rock and wire mattresses. Special wire baskets of manageable sizes are manufactured and sold throughout the United States. Since the service life of the installation depends mostly on the rate of deterioration of the wire mesh and ties, they should be galvanized.

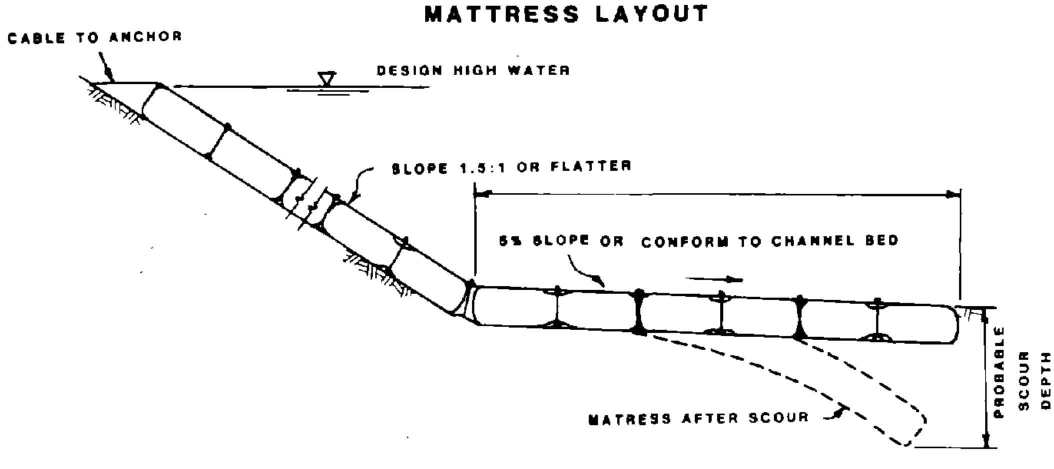
Used-Tire Revetments

The U.S. Army Corps of Engineers (1981) reports that revetments constructed of used automobile tires have been successfully employed as streambank protection by the U.S. Army Engineer District, Sacramento; U.S. Bureau of Indian Affairs, Oklahoma; Washington State Highway Department; West Virginia Department of Natural Resources; and the U.S. Forest Service, Mississippi. The used tires usually are placed over the surface to be protected and lashed together with wire, steel bands, or nonbiodegradable rope to form the mat structure. A typical tire mattress design is shown in Figure 54.

Used-tire revetments are best suited for bank-surface protection. As surface protection, tire mats have been effective against velocities up to 10 feet per second on mild bends. They are flexible to the same extent as rock and wire mesh revetments; that is, they will give with a limited amount of surface subsidence, but they usually will be damaged if excessive subsidence occurs. As a result, tire mattress designs are not well-suited for use where



MATTRESS LAYOUT



SECTION

FIGURE 53. ROCK AND WIRE MATTRESS DETAILS.
(MODIFIED FROM CALIFORNIA DEPT. OF PUBLIC WORKS, 1970)

excessive toe scour is anticipated. This limits their usefulness to tangent reaches or mild curvature bends, unless some other means of toe protection is provided. With respect to subsurface seepage and groundwater flow conditions, tire mattresses have experienced mixed success. When the tire mattress is underlaid with a granular filter and the tires are filled with gravel, they appear to provide a sufficient level of protection against subsurface flow-erosion mechanisms.

When tire mattresses are placed on granular filters, however, care must be taken to anchor the mattress adequately to the bank. Lack of sufficient anchoring has caused several instances of mattress slippage to occur. Anchoring recommendations are indicated in Figure 54. The addition of vegetative plantings to the mattress scheme is helpful. After vegetation becomes established, mattress stability becomes less of a problem.

There are several disadvantages associated with the use of tire revetments. First, the costs of the scheme must be considered. As indicated in Chapter 4, the cost of constructing tire mattresses is extremely variable and can be quite expensive; constructing a mattress is extremely labor intensive. If volunteer labor and free materials can be used, the scheme could become cost effective. If there is a significant cost associated with the used tires, however, and hired labor must be used, the schemes will be extremely expensive. While individual landowners might be able to find volunteer labor and free materials, this option usually is not available to Department of Transportation applications. Tire revetments also have been found to be more susceptible to vandalism than other schemes. The cutting and dismantling of the mattresses is a common problem, particularly in urban areas. They also are considered to be esthetically and environmentally unacceptable to large segments of the general public.

Precast Concrete Blocks

Two types of precast concrete blocks have been used as flexible revetment mattresses. These are cellular-block designs (Figure 30) and articulated concrete block designs. Both designs are somewhat permeable and provide a limited amount of flexibility. This permits free draining of the bank materials and allows the mattress to conform to minor changes in bank geometry. The primary function of precast concrete blocks is to prevent surface erosion from streamflow. These bank revetments have been effective in resisting surface erosion under a wide range of flow environments. Precast concrete blocks are particularly applicable to high-velocity environments. Their limited flexibility, however, makes them subject to undermining in environments characterized by large fluctuations in bed elevation. This has been a problem when they are used at sharp bends in channels having dynamic channel beds. As was shown in Figure 49, precast concrete block designs can be very expensive bank protection treatments. For this reason, their use has been limited to large rivers or areas where structures of significant value need to be protected. They also might prove to be cost effective in areas of the United States where riprap is not readily available.

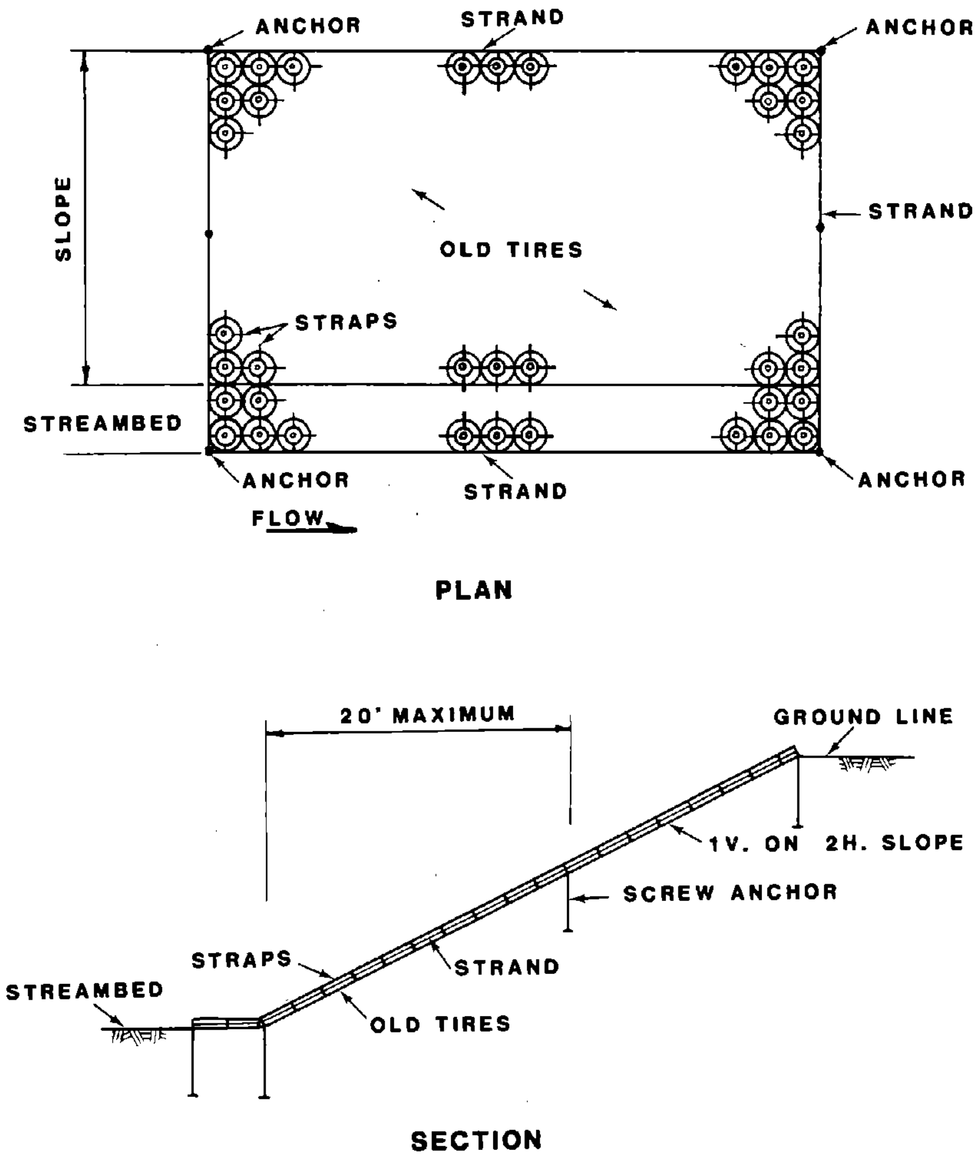


FIGURE 54. TYPICAL USED TIRE REVETMENT
(MODIFIED FROM U.S ARMY CORPS OF ENGINEERS, 1981)

Precast cellular blocks are available from several commercial sources (see U.S. Army Corps of Engineers, 1981, Appendix A). The cellular block mattress usually is constructed by bonding precast cellular blocks to rectangular sheets of filter fabric (referred to as fabric carrier). Each mattress section is fabricated with a sufficient margin of filter fabric extending beyond the blocks on at least three sides. This extra margin of filter fabric permits the mattress sections to be lifted by mobile crane onto the streambank to be protected. The blocks making up the mattress are cast in cells that create the flexibility and permeability. Also, the cells allow vegetation to grow through the blocks and enhance the structural integrity of the bank.

Articulated concrete blocks are made of precast concrete blocks held together by steel rods or cables as shown in Figure 55. Block size may vary to suit the bank contour. It is particularly difficult to make a continuous mattress of uniformly-sized blocks to fit sharp curves. Open spacing between blocks permits removal of bank material, unless a filter blanket of gravel or plastic filter cloth is placed underneath. For embankments that are subjected only to occasional flood flows, the spaces between blocks may be filled with earth, and vegetation can be established.

Because of the high cost of the plant required for placement of the mattress beneath the water surface, use of articulated concrete mattresses has been limited primarily to the Mississippi River. Thus, it is economically feasible to use articulated concrete mattresses only on rivers that require extensive bank protection. The expense of the installation plant is not a factor in the placement of articulated concrete mattresses above the water surface. Thus, paving the upper bank with articulated concrete mattresses has been done occasionally in the United States and Europe.

Vegetation

The use of vegetation is one of the least expensive means of bank stabilization. Both woody plants and herbaceous vegetation can play an important role in stabilizing and controlling channelbank erosion. Planting of vegetation, however, seldom is used as a primary protection mechanism. Its chief use has been for bank protection in conjunction with other structural measures. Vegetation frequently is used as an upper-bank protection measure and as a supplement to other schemes. Other protective measures are used on the lower bank. Vegetation has been found to be particularly effective when used in conjunction with retardance structures, spurs, and some revetments.

Vegetating formed bank slopes without the use of structural measures usually is limited to agricultural waterways or small channels with stable beds and fairly low gradients. If bank undercutting or unstable bed conditions are evident, bank shaping without the support of structural materials is not recommended. The checking or elimination of scouring forces creating channelbed degradation is necessary before satisfactory results can be expected from the use of vegetative treatments.

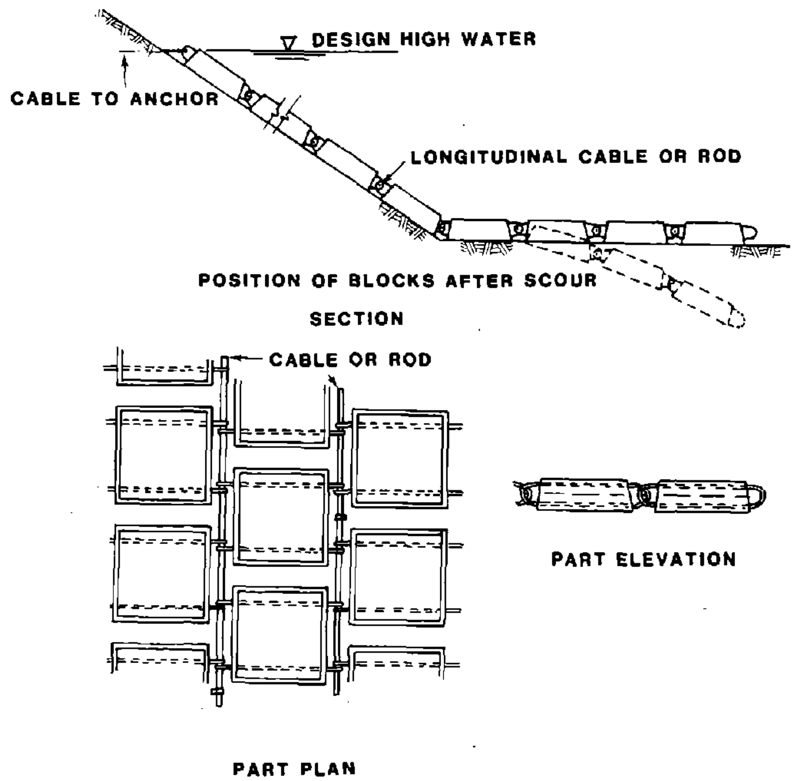


FIGURE 55. ARTICULATED CONCRETE BANK REVETMENTS
(MODIFIED FROM CALIFORNIA DEPT. OF PUBLIC WORKS, 1970)

The principal function of vegetation is to improve the structural integrity of the bank soil structure. It also will keep fast-moving water and transported coarse materials away from the surface of the streambank slope. Above the mean high-water line of bank slopes and in backwater areas, the major soil- erosive action results from the mechanical disintegration of soil masses by wind, and alternate wetting and drying. Vegetative treatments, particularly grasses, have proven to be excellent deterrents to soil erosion under these conditions. They also are quite effective when used to strengthen the bank surface to resist other subsurface flow-erosion mechanisms.

Factors influencing the applicability or success of vegetation schemes include channel size, bank material height and stability, maximum velocities and flow characteristics along the channel banks, channelbed stability, and the degree of bend curvature. Another important consideration is the time of year during which the plantings are undertaken. If sufficient time is not allowed for the plantings to take hold prior to the high-water season, the success of the scheme will be jeopardized.

The primary advantages of vegetative treatments are low cost and esthetic appeal. As discussed in Chapter 4 and illustrated in Figure 49, vegetative treatments are among the least expensive means of stabilizing channel banks. As for esthetic appeal, vegetative plantings will provide a natural appearing channelbank as the plants reach maturity.

The advantages mentioned above must be weighed against some obvious disadvantages. First, the above discussions indicated that vegetative treatments are useful primarily as a secondary or upper-bank treatment; not as a primary treatment. Second, the time required to establish a dense protective cover must be considered. Even under good environmental conditions, with a proper balance of soil moisture and plant nutrients, at least two growing seasons are required to establish a dense cover for many of the grasses; more than five years will be required to obtain appreciable growth for most woody species. The obvious problem here is that the risk of losing the bank protection is high, because several high-flow periods probably will occur before the vegetative cover is well established. Another disadvantage is that channelbank vegetation is subject to change from destructive physical action and through natural laws of plant succession (i.e. seasonal changes in plant development). It also has been indicated that too much plant growth can reduce the channel capacity, particularly around bridge openings.

There are two principal areas that must be considered when designing or specifying vegetation for bank protection. The first of these is the selection of an appropriate plant species. As mentioned above, plant species can be classified either as woody plants or grasses. Woody plants require a longer time to become established than grasses, but they provide more effective long-term protection. Trees raised in nurseries are preferred over local plants because they are usually healthier, bushier, and have well-developed root systems at maturity. When selecting trees or other woody plants, native species are preferred because they usually are the easiest to propagate. This applies to grasses as well. Other considerations include the length of time for the stand to reach maturity, the soil and air temperature, total rainfall and rainfall distribution, type of soil available for planting, bank slope, and the ability of the soil to store water for plant growth during dry periods.

Bank preparation also must be considered. Steep banks must be graded. Bank slopes to be vegetated should be no steeper than 1:2. Better results will be achieved, however, using slopes of 1:3 or flatter. Topsoil on the bank to be protected generally is stripped because it provides a fertile bed that enhances the growth of weeds, which tend to choke grasses. The exposed soil usually is rolled and then scarified prior to planting.

Perhaps the most important design consideration when using vegetative treatments is the time of year when the plantings are made. The vegetation should be planted at the beginning of the growing season and/or immediately after any rainy or flood periods. It also is important for the plantings to be monitored and repaired as necessary until they are established.

Rigid Revetments

Rigid or monolithic revetments are solid, continuous protective layers supported by the bank. The most common rigid revetment designs include pavements, concrete-filled mats, sand/cement bags, and grouted riprap. Advantages of monolithic revetments include the following:

- The implied structural integrity of rigid revetments makes them more resistant to damage from debris, ice, and other floating objects.
- Rigid revetments generally are smoother than flexible types; this can be an advantage where maximum flow efficiency is needed.
- Because they are solid, the surfaces of most rigid revetment are immune to erosion, making them well-suited for use under extreme hydraulic conditions.

The primary disadvantage of rigid revetments is that the effectiveness of these designs may be impaired by any action that may rupture the surface. Such breakage or misalignment of the surface may result from the removal of foundation support by subsidence, undermining, outward displacement by hydrostatic pressure, slide action, or erosion of the supporting embankment at its ends. Also, Figure 49 indicates that rigid revetments are among some of the most expensive streambank protection designs.

Considering these advantages and disadvantages, the use of rigid revetments is best reserved for situations where bank-surface erosion is being caused by excessive hydraulic conditions, and where the value of the structure being protected justifies the high cost of these structures.

Pavements

Streambank pavements are usually made of either portland cement concrete (PCC) or asphalt concrete. All references to concrete are to PCC, since it is more common than asphalt concrete.

After field inspection of many channelbank revetment sites, Brice et al., (1978) report the following as common causes of failure of bank pavements:

- undermining of the toe of the pavement,
- erosion at the ends of the paving,
- hydrostatic pressure build-up behind the pavement,
- vertopping and erosion at the interface between the bank surface and the slab, and
- erosion from high velocities at the bank/slab interface due to the smooth slab surface.

As with other revetments, concrete and asphalt pavements should be used only where the toe of the paving can be adequately protected from undermining. This is a particularly important consideration where general channel degradation occurs. Also, if subsurface flow conditions are a problem at a site, pavements should not be used unless adequate drainage is provided to relieve hydrostatic pressures behind the slab. In the past, concrete or other pavements have been best utilized as subaqueous revetments (on the bank below the water surface) with vegetation or some other less expensive upper-bank treatment. Pavements are particularly adaptable to locations where the hydraulic efficiency of smooth surfaces is important. Because many of the causes of pavement failure listed above are related to high flow velocities and fluctuating water levels, the use of slope paving is recommended only where gradients are small, flow is controlled, and/or maximum flow is limited.

As is the case with rigid revetments in general, the initial construction cost of pavements is high. Also, since pavement failures tend to be progressive, repairs usually are extensive and costly. However, the cost on an area-covered basis usually is less for pavements than for some other rigid revetments (California Department of Public Works, 1970).

Although bank pavements are among the most expensive types of countermeasures, there are advantages associated with their use. The primary advantage is that they will provide a high degree of reliability over a long life with a minimum of maintenance, if they are properly designed. This refers primarily to the surface durability. Other advantages include the hydraulic efficiency and neat appearance of bank pavements.

The design of pavement revetments is covered in detail in California Department of Public Works (1970). The primary design components are slab foundation and thickness. Foundations for concrete pavement (as well as for other rigid revetments) must be well-designed to form a stable bank. Continuity of the surface is important, and the bank should be well-compacted and stable to maintain continuity.. Although reinforcement will enable pavements to bridge small settlements of the embankment face, extensive movements would be disastrous. Slab thickness may range from 3 to 6 inches, and the slabs may be plain or reinforced. A typical design might specify a 6-inch thickness, reinforced with welded-wire mesh, placed on a 2:1 slope.

Other considerations include relief of excess hydrostatic pressures and protection of the slope toe and other edges. As was mentioned earlier, excess hydrostatic pressures can develop behind rigid revetments as water pressure builds behind the pavement. Every precaution must be taken to exclude stream water from pervious zones behind the slope paving. The light slabs can be lifted by comparatively small hydrostatic pressures, opening joints or cracks at other points in a series of progressive failures leading to extensive or complete failure.

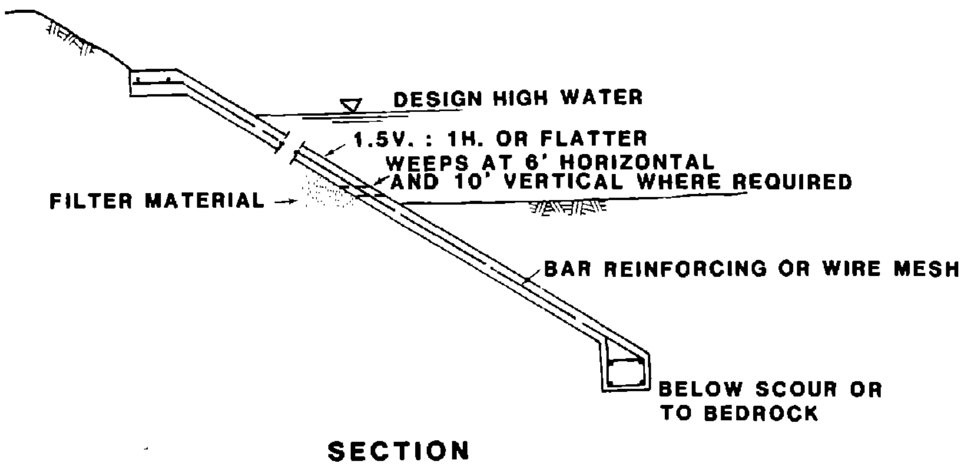


FIGURE 56. DETAILS OF CONCRETE SLOPE PAVING (MODIFIED FROM CALIFORNIA DEPT. OF PUBLIC WORKS, 1970)

The toe of slope pavement must either be on a firm foundation or extend below possible scour. If this is not practical, some other means of toe protection must be provided, such as a cutoff wall. Cutoff walls often are needed at the ends of the slab and to prevent undermining of the ends of the protection.

All of the pavement-design components mentioned above are illustrated in Figure 56.

Concrete-Filled Mats

Concrete- filled mats consist of fabric envelopes filled with a pumpable sand and cement grout. This product is marketed under the names "Fabriform," "Fabricast," and "Enkamat". The Fabriform process is protected by U.S. patent numbers 3396542, 3396545, and R.E. 27460. Other U.S. and foreign patents also have been issued or are pending. Of these products, Fabriform is referred to most often in the literature. For this reason, the following comments are based primarily on Fabriform installations. Because of their similarities, however, the following comments also will apply to the other variations.

Concrete-filled mat revetments are best suited for protection against surface erosion from wind and boat-generated waves in lakes, reservoirs, impoundments, or other backwater or low-velocity areas. They have not had good performance records in high-velocity environments. This is because the high-velocity currents create excessive turbulence and eddies that frequently cause the erosion of bedding material from the edges of the Fabriform layer. This is the primary cause of failure with concrete-filled mat revetments. This is a particular problem in sand-bed channels where erosion of bed material at the toe of the bank can be excessive. In some cases, cutoff trenches have been constructed at all edges of the Fabriform to eliminate the loss of bank-foundation material. However, this practice complicates construction, particularly under water, thus increasing the revetment's cost.

The cost comparison given in Chapter 4 indicates that the cost of concrete-filled mat revetments to range from $62 to $472 per foot of bank protected (1982 dollars). These figures are based on only four sites. They indicate, however, the extreme variability in costs associated with this type of protective scheme. Part of this variability is due to site preparation and contractors' inexperience in the application of the required construction methods. Additional fluctuation can be attributed to the variability of bank heights. The cost range reported indicates that concrete-filled mat revetments will not be the most economical choice in some instances. However, Brice et al. (1978) report that in several cases, concrete-filled mats were chosen over riprap revetments partially because of cost considerations. This indicates that they can be an economical solution in some instances.

Concrete-filled revetments have advantages similar to other rigid revetments. These include strength, durability, and hydraulic efficiency. In addition, some concrete-filled revetments allow for the relief of the hydrostatic pressures that can build up in the bank behind the revetment and often cause the failure of rigid revetments. Another advantage is that these revetments are extremely stable, even on slopes as steep as 1 vertical to 1 horizontal. Therefore, where steep slopes are unavoidable, and stability of protection is essential, concrete-filled mat revetments are a good choice. Additionally, concrete-filled mat revetments are not labor intensive, as are some of the flexible designs. One manufacturer estimates that labor, equipment, and field overhead account for only 20 percent of the cost of a particular job. The remaining 80 percent represents material costs. However, the construction of concrete-filled mats does require the use of some special equipment and processes.

The design and construction of concrete-filled mats is relatively simple. The mats are placed directly on the prepared channelbanks. Filter fabrics general are not required, as some of the fabric envelopes have filter qualities of their own. The fabric envelope is then filled by pumping a highly fluid sand/cement mortar into it.

Sand/Soil-Cement Bags

Sand/soil-cement bags are monolithic revetments that allow a simplistic approach to the construction of a pavement by using filled bags as building blocks. This revetment is composed of sacks that are prefilled with a dry mixture. Sacks made of burlap, plastics, and more recently, biodegradable materials have been used. Fill materials have included soil, sand, soil-cement, and most commonly, sand-cement mixtures. The trend in bag revetments is toward degradable bags. Therefore, sand and soil fill alone should not be used, since they will lose their structure and be washed away as the bags decay. Also, when soil and sand alone are used, the revetment assumes qualities similar to flexible designs.

Sand/soil-cement bag revetments are constructed by stacking individual sacks on the eroding bank. In many cases they can be stacked to conform to the existing bank geometry. When in place, the sacks are wetted so that they will become bonded to each other, thus forming a pavement. Sand/soil-cement bag revetments are subject to failures from the same causes as other rigid revetments. Primary among these are

- undermining of the revetment at its toe,
- erosion at the ends of the protection, and
- hydrostatic uplift.

It has been observed that almost all failures of sand/soil-cement bag revetments have resulted from stream water eroding the embankment material; either from the toe or from the ends of the pavement.

Sand/soil-cement bag revetments are simple to build and adaptable to almost any embankment contour. They have been used as an effective alternative to riprap where suitable stone is not available; sand and gravel can be taken out of the streambed for use in the sand/soil-cement mixture.

As for costs, Chapter 4 indicates that sand/soil-cement bag revetments are among the more economical streambank stabilization schemes; they are comparable in cost to riprap revetments. As indicated, the sites monitored ranged in cost from $57 to $186 per foot of bank protected. The average cost for the sites evaluated was $114. In contrast to this, the California Department of Public Works (1970) reports sand/soil-cement sack revetments to be expensive, costing (on the average) twice that of riprap designs. The Department only recommends its use where sand and gravel materials for making up the mix are available in the streambed.

There also are disadvantages associated with the use of sand/soil-cement sack revetment designs. These designs do act as a monolithic or rigid revetment, and as such, have the addition, sand/soil-cement revetments have very little internal strength, making them more susceptible to damage from ice and debris impact loads than other rigid designs. This has been well-documented on the Lower Chippewa River in Wisconsin (U.S. Army Corps of Engineers, 1981). Combined with their lack of flexibility, this low internal strength means that sand/soil-cement bag revetments must depend almost entirely upon the stability of the embankment for support, and therefore, should not be placed on face slopes much steeper than the angle of repose of the embankment material.

Several design recommendations were found. With regard to bank slope, sand/soil-cement bags placed edge to edge should be limited to areas of slopes equal to or flatter than 2.5 horizontal to 1 vertical, because the bags have a tendency to slide. Otherwise, bags should be placed horizontally and overlapped to form the steeper slope (see Figure 57). If the bags are to be stacked, slopes up to 1.5 horizontal to 1 vertical have been used successfully (U.S. Army Corps of Engineers, 1981). Also, on the steeper

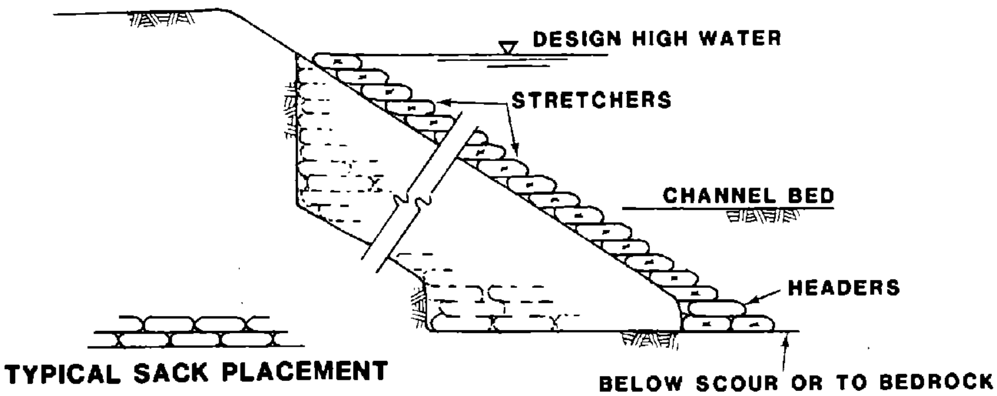


FIGURE 57. TYPICAL SAND/SOIL-CEMENT BAG REVETMENT DESIGN
(MODIFIED FROM CALIFORNIA DEPT. OF PUBLIC WORKS, 1970)

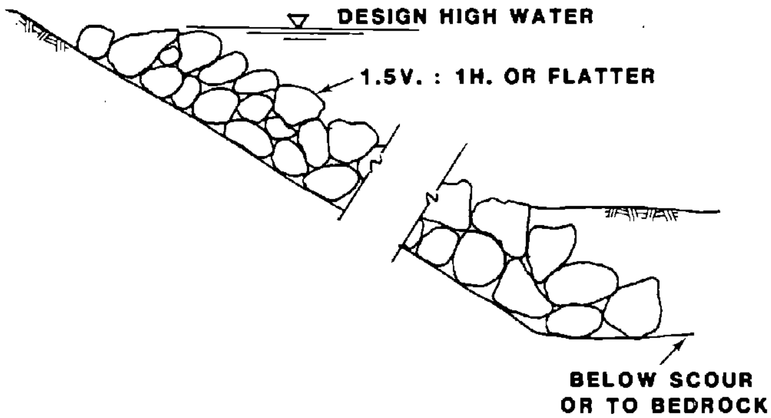
slopes, burlap bags should be used instead of paper or a synthetic plastic to create greater bonding between bags. Since one of the primary modes of failure for sand/soil-cement bag revetments is from undermining at the toe, the bottom should be founded on bedrock or below the depth of possible scour. If the ends are not tied into rock or other nonerosive material, cutoff returns should be provided, and if the protection is long, cutoff stubs should be built at regular intervals (Calif. Dept. of Public Works, 1970). A typical design sketch is shown in Figure 57.

Grouted Riprap

There is not much field information available about grouted riprap revetments on which to base application or design recommendations. However, since grouted riprap can be a useful revetment in some instances, the following discussion, condensed from California Department of Public Works (1970), is included.

Grouted riprap revetment consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. A typical grouted riprap section is shown in Figure 58. It has application in areas where rock of sufficient size for ordinary rock-slope protection is not economically available; it also can be used to reduce the quantity of rock required. Grouting not only protects the stones from the full force of high-velocity water, but integrates a greater mass to resist its pressure.

Grouting usually will more than double the cost per unit volume of stone, but the use of smaller stones in grouted-rock riprap slope protection than in an equivalent protection using ungrouted stones permits a lesser



SECTION

FIGURE 58. GROUTED RIPRAP SLOPE PROTECTION
(MODIFIED FROM CALIFORNIA DEPT. OF PUBLIC WORKS, 1970)

thickness of protection, which offsets the additional cost of the grout. Also, if the embankment material is fine grained, grouting will eliminate the need for filter material that may be necessary with ordinary rock slope-protection. General advantages and disadvantages associated with other rigid revetments also apply to grouted riprap designs.

Since grouted riprap slope-protection is rigid, but not extremely strong, support by the embankment must be maintained. Slopes steeper than the angle of repose of the embankment are risky, but with rocks grouted in place, little is to be gained with slopes flatter than 1.5 horizontal to 1 vertical. Measures to prevent undermining of embankment are particularly important. The grouted rock must be founded on solid rock or below the depth of possible scour. Similarly, ends should be protected by tying them into solid rock or forming smooth transitions with embankment subjected to lower velocities. As a precaution, cutoff stubs may be provided; these were discussed in the sand/soil-cement bag revetments section of this report. Detailed design information can be found in California Department of Public Works (1970).

Composite Revetments

Composite revetments are channelbank revetments consisting of vertical zones of protection; each zone is composed of a different revetment material or level of protection. Most composite revetments consist of two zones: a lower-bank zone to protect the toe and lower portions of the bank, and an upper-bank treatment covering the zone of normal seasonal fluctuations in water surface. Occasionally, a third zone providing protection of a freeboard area also is provided.

The primary advantage of composite revetments is that different revetment materials can be used on different bank zones according to the needs of that particular zone. In many cases, the level of protection required for upper portions of the bank will be different (and in most cases less) than that required for the lower bank. Composite designs simply use different revetment materials to meet the needs of different portions of the bank. There really is nothing novel about this concept, but it is rarely used. Composite revetments are economical. On the Missouri River they were found to be one of the most economical and effective protection schemes tested during studies conducted by the U.S. Army Corps of Engineers (1981).

Composite revetments are effective treatments for many bank-line and channel conditions. They are particularly well- suited for for deep channels with high banks. Additionally, composite revetments designs are well-suited for rivers whose water levels usually stay within a well-defined range; for example, rivers controlled by dams and those influenced by backwater from reservoirs and other impoundments. In these cases the zone of normal water fluctuations can be provided with heavier revetment than the portion of the bank impacted only a few times a year. Composite revetments also are useful on channels whose upper and lower banks are being eroded by different mechanisms; for example, when the lower bank is being impacted by toe-erosion processes and the upper bank by subsurface drainage erosion. In this case, a heavy, flexible revetment is needed at the toe of the bank while some minor grading and a gravel or fabric filter, or vegetative plantings perhaps would be enough for the upper bank.

One disadvantage in the use of composite revetments is that the resulting full-bank protection requires some upper-bank clearing, which makes this zone very visible during high flows. In many cases, however, the lower-bank protection would have been extended to the upper bank, and these areas would have been exposed anyway. When selecting an upper bank revetment, it is important to select one that requires a minimum of grading or other bank preparation.

RETARDANCE STRUCTURES

Retardance structures are permeable devices generally placed parallel to embankments and river banks. Their primary purpose is to offer protection to the bank toe by reducing riparian velocity. Besides protecting the banks from erosion resulting from flow impingement, the reduction in velocity also will induce deposition of transported sediments at the toe of the bank. Both the structure and the deposited material will keep the primary erosive currents away from the bank. The most common types of retardance structures include jacks and tetrahedrons, wood-and- wire fence structures, and lines of timber-pile bents.

As indicated in Chapter 3, retardance structures have been used to resist erosion from streamflow, abrasion, and waves. They will not counter erosion from subsurface flow, surface weathering, or chemical action. Also, retardance structures are best suited for protecting low banks or lower portions of the bank, and not high banks or upper portions of the bank. Retardance structures can be used to protect an existing bankline, or to reestablish some previous flowpath or alignment. The latter function, however, is better served by spurs or longitudinal dikes. As compared with revetment along a bankline, the retard can be oriented to provide better flow alignment, and it can provide an outlying (away from the bank) line of defense against bank erosion. Also, retards do not require bank support, and therefore, are applicable where bank materials will not support a revetment material. Retardance structures become ineffective if undermined, however, and therefore should not be used on actively degrading streams unless special considerations are made to prevent loss of the structure from undermining.

Additionally, the following advantages for the use of retardance structures have been identified:

- The construction of retardance structures will, in most cases, only require a minimum of upper bank disturbance during construction.

- Retards can be oriented to provide a more positive action in maintaining an existing alignment, or providing a new alignment than other countermeasure types.

- The sediment deposition caused by the retardance structure can create an environment acceptable to the volunteering of vegetation.

Several disadvantages are:

- Retardance structures usually will limit access to the river along the bank they protect.

- Retardance structures have a history of being easily outflanked in cases where sufficient upper-bank protection is not provided and/or where a smooth transition from retardance structure to bank is not provided at the upstream and downstream terminus of the structure.

- Retardance structures are hazardous to those who use the river for recreational purposes. Specifically, the pose a hazard to boat traffic; especially if they are constructed to a height lower than normal water level so that they are just under the water surface in normal flow conditions.

Cost of construction is an advantage in the use of retardance structures. The cost data available (as shown in Figure 49) indicate that retardance structures are among the more cost-effective designs available for countering streambank erosion. Considering all types of retardance structures, the average cost for the 149 sites surveyed was $82.91/ft of bank protected. The range of costs reported was $13/ to $342/ft of bank protected.

Several recommendations can be given for the design of retardance structures. The alignment of retardance structures should provide a smooth transition from bendway to bendway. Both the high-water and low-water paths should be considered in alignment design. Also, as mentioned above, care must be taken to design adequate transitions between the structure and the bank at the extremities of the retardance structures to prevent outflanking. Acheson (1968) reports that the height of retardance structures should be approximately at the annual flood level. This confirmed the findings of O'Brian (1951), who found through model experiments that structure heights at or just below the normal water level were as effective as structures twice as high.

Information about design and applications is provided below for the major types of retardance structures. Additional information about the design and construction of these countermeasures can be found in U.S. Army Corps of Engineers (1981), and California Department of Public Works (1970).

Jacks and Tetrahedrons

Jacks and tetrahedrons are skeletal frames adaptable to permeable retards by tying a number of similar units together in longitudinal alignment. Cables are used for ties between units and for anchorage of key units to deadmen. Struts and wires are added to the basic frames as desired to increase flow impedance directly by their own resistance and indirectly by the debris they collect. A typical jack design is shown in Figure 59. Its basic frame is a triaxial assembly of three (3) mutually perpendicular bars acting as six (6) cantilevered legs from their central connection. Wires are strung on these members to relieve stress in the legs and to collect drift and impede the flow of water. The basic frame of the tetrahedron, shown in Figure 60, is assembled from six (6) equal members; three (3) form the triangular base, and the others form the three (3) faces sloping upward from the base to an apex. These faces are like the base in all respects, so that it can be supported equally well on all sides.

As with other retardance structures, jacks and tetrahedrons function as flow-control measures by reducing the water velocity along the bank, which in turn results in accumulation of sediment and establishment of vegetation. The flow-retardance features of these structures are enhanced by the collection of light, floating debris among the structural members. As might be expected, these structures function best in environments with significant bed material load and light, floating debris. In fact, if debris and sediments do not accumulate at these structures, they offer little or no protection to the bank. While light floating debris enhances the effectiveness of jacks and tetrahedrons, heavy debris and ice can damage these structures severely, thus making them ineffective.

Several additional recommendations for the application of jacks and tetrahedrons can be made. Brice et al., (1978) and Keeley (1970) indicate that these retardance structures are not an effective means of altering flow direction. They also have not been effective in halting erosion on sharp bends or in high-velocity flow environments. They are best suited for use on mildly bending and straight reaches of low-energy streams. In addition, jacks and tetrahedrons have been used effectively to reduce the velocities of shallow overbank or floodplain flows.

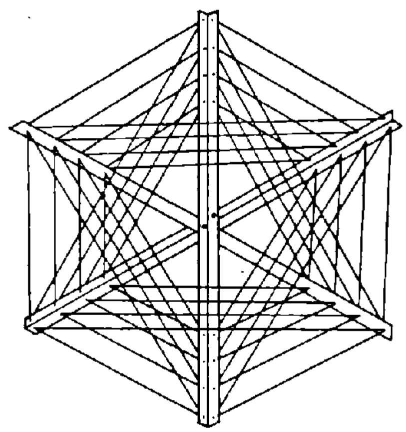


FIGURE 59. TYPICAL JACK UNIT

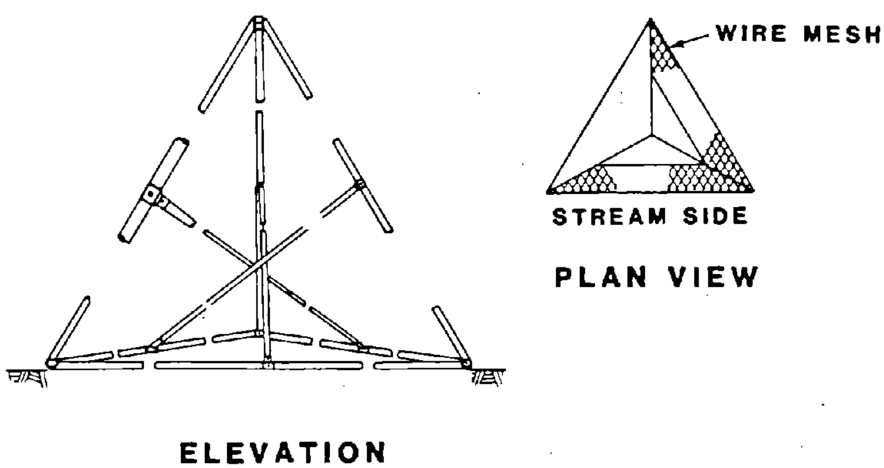


FIGURE 60. TYPICAL TETRAHEDRON DESIGN

There are several significant disadvantages to the use of jacks and tetrahedrons. These structures are unacceptable for use in urban areas or near recreational areas for reasons of safety and esthetics. In these areas the units can become an attractive nuisance; the sharp edges and wire can be dangerous features to children running around and/or climbing on them. Jacks and tetrahedrons also pose hazards to boaters. Their unprotected metal surfaces will corrode, and become unsightly and dangerous as time passes. Also, excessive settlement often renders these structures ineffective through burial or displacement. This is a problem particularly in channels or reaches with extremely dynamic streambed movement. Displacement also can result from damage caused by large, floating debris and ice. After displacement, these structures often can become flow obstructions, causing flow deflection towards the bank and a more serious erosion condition than originally existed.

Jacks and tetrahedrons can be arranged either in linear or area configurations. Linear designs, often referred to as arrays, are constructed with one, two, or three rows of jacks or tetrahedrons aligned parallel to the bank at the bank toe (see Figures 36 and 37). Double and triple rows have been found to be more effective than single rows. It also is possible to use variable row lengths, thus providing more erosion resistance at the critical points in the bend. Linear configurations are used on narrow channels and in other situations where flow constriction is neither needed nor desired.

Area configurations often are referred to as retardance fields because of their area coverage. Figure 40 shows area installations. Area designs are made up of lateral and longitudinal rows of jacks or tetrahedrons. The lateral rows usually are angled about 45 to 70 degrees downstream from the bank. The spacing varies depending upon the debris and sediment content in the stream; the structures may be 50 to 200 feet apart. A typical area schematic is shown in Figure 61. Area designs are well-suited for design situations where flow retardance and sediment deposition are required over an area of streambed or floodplain. They also are well-suited for producing a smooth flow alignment along irregular banks.

Fence Retardance Structures

Fence retardance structures provide bank protection in a fashion similar to other retardance structures. Fence structures have been successfully used on small to moderately sized low-gradient streams that have infrequent flood-flows of short duration. As with other types of retardance structures, they provide protection for lower portions of the bank. They also can be used to break up and reduce wave action as it approaches the bank. However, excessive wave action from tow and other boat traffic has been known to damage fence structures.

Many types of locally available materials can be used for fence construction. The fence posts can be of treated or untreated wood, used rails, pipe or steel beams, or concrete. Additional supporting members can be constructed of the same materials. The fencing material generally is composed of wood planks or wire. If wire is used, the required tensile

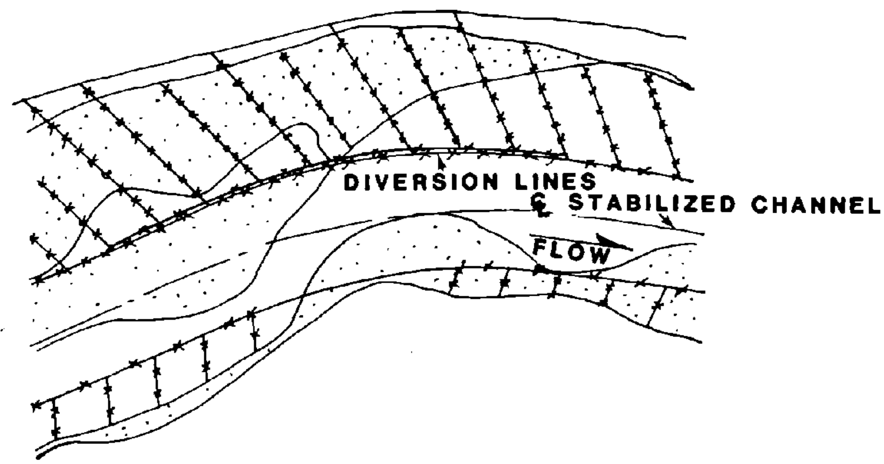


FIGURE 61. RETARDANCE FIELD SCHEMATIC

strength depends on the design loading by the water and debris. Common field fencing, welded-wire fencing, and chain-link fencing have been used.

As mentioned previously, toe scour along longitudinal fence retards has been identified as the primary cause of failure of these countermeasures. This has been verified by both field and laboratory observations. To protect against this, the fence material either must be extended to a level below the expected depth of scour, or a protective rock toe must be provided along the base of the fence.

As was the case with jacks and tetrahedrons, fence structures have been susceptible to damage from ice and heavy debris. Since they are constructed parallel to the bank, however, they are less susceptible to damage from these sources than permeable spurs that project into the channel. Also, excess debris accumulation can cause flow deflection behind the structure, which can result in additional bank erosion.

Flow channelization behind retardance structures also can be a problem. The development of flow channels between these structures and the bank frequently will occur if a sufficient vegetative cover does not develop on the bank prior to the first flood or high-flow season subsequent to construction. To counter this, tiebacks can be used to break up the potential flow path. Tiebacks are sections of fence constructed between the linear retard and the bank. O'Brian (1951) reports that tiebacks should be constructed at a 45-degree angle to the retard line for best performance. No criteria for the spacing of tiebacks have been developed; until additional information becomes available, spacing should be based on the design engineer's best judgement. Figure 62 shows a wood fence retardance structure with perpendicular tiebacks.

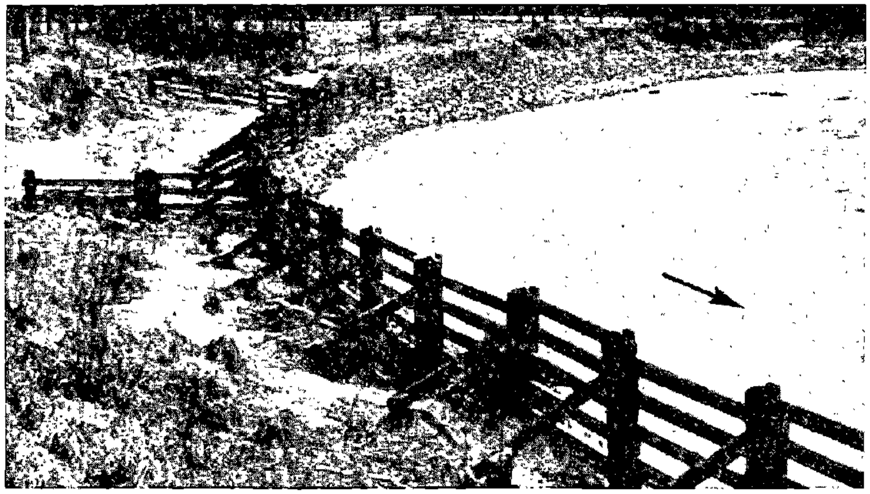


FIGURE 62. WOOD RETARDANCE STRUCTURE WITH TIEBACKS AND ROCK TOE PROTECTION (SOURCE: BRICE ET AL., 1978)

Wood Fence Designs

Wooden fence retardance structures, often referred to as training fences, have been used on small, sand-bed channels in Mississippi by the COE and SCS. Results of the Section 32 Program indicate these structures to be an effective means of bank protection on small channels having moderate to sharp bends. In fact, they have been found to provide a more positive action in maintaining an existing alignment, and can be more effective in preventing lateral erosion at sharp bends than other retardance structures. However, their use should be limited to areas where banks can be well-vegetated. Also, it is important that adequate toe protection be provided to resist undermining, particularly on sharp bends in sand-bed channels. This is most often provided by the addition of a rock toe (longitudinal rock-toe dikes will be discussed in a later section). A typical design drawing for a wood-board fence retardance structure, including a rock toe, is shown in Figure 63.

The cost analysis mentioned in Chapter 4 indicates that wood-type retardance structures are an economical alternative. Forty-five sites were used in the analysis, and the average cost was found to be $83.31/ft of bank protected; the low and high values were $39/ft and $162/ft, respectively. Note that the cost analysis was based on linear designs only, and at some sites the reported costs included a rock-toe dike and tiebacks.

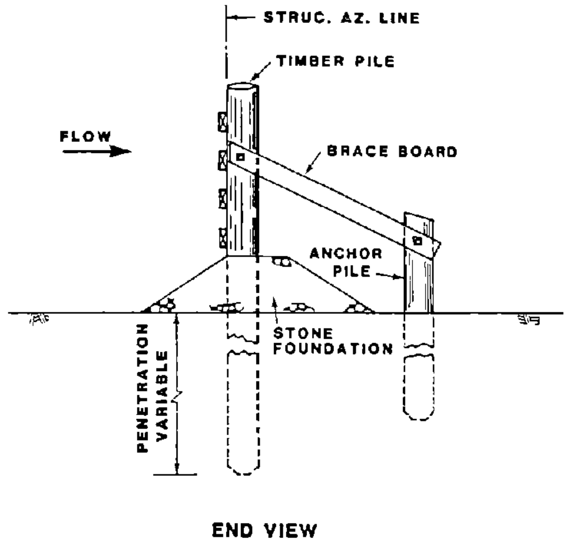


FIGURE 63. TYPICAL WOOD FENCE RETARDANCE STRUCTURE (MODIFIED FROM U.S. ARMY CORPS OF ENGINEERS, 1981)

Wire Fence Designs

Wire-fence retardance structures have not been found to be as effective as wood-fence designs at providing flow alignment and bank protection. They have, however, been found to be effective when used on mild to moderate bends on small to moderately sized channels, and in areas of less frequent streamflow attack. Where the collection of brush and other small debris might pose a fire hazard, wire-fence retards supported by metal posts and other supports are preferred. Illk (1963) reports that wire-fence retardance structures were successful on the Colorado River as long as there was a fairly high sediment concentration. However, as channelization activities began to reduce the sediment load of the river, it was found that these structures no longer performed satisfactorily.

Wire-fence retardance structures are susceptible to the same hazards as other types of retardance structures. Again, the primary cause of failure of these structures is undermining resulting from scour at the toe of the fence. They are also susceptible to damage from ice and heavy debris.

As with jacks and tetrahedrons, wire-fence designs have been constructed in both linear and area configurations. The criteria for selecting either a linear or area configuration is the same as for jacks and tetrahedrons. Again, linear configurations can consist of single or multiple fence rows,

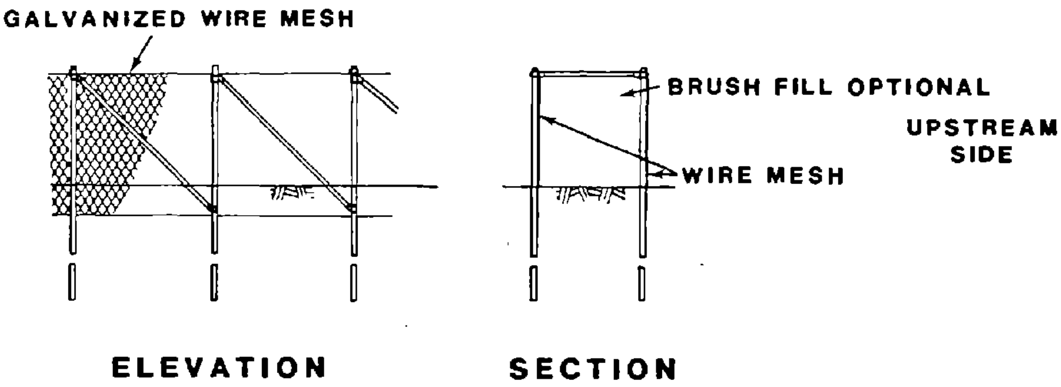


FIGURE 64. LIGHT DOUBLE-ROW WIRE FENCE RETARDANCE STRUCTURE

depending on the level of retardance needed at a site. Double-row linear-wire retardance structures are sometimes are filled with brush to increase their retardance capabilities. As was the case with wood fence designs, linear-wire fence designs can include tiebacks where required. The geometric layout of area designs are similar to those given above for jacks and tetrahedrons. Wire-fence area retards differ from jack and tetrahedron designs in that they are rigidly attached to the channelbed. This could be an advantage or disadvantage, depending on the situation. If the desire is to prevent movement or dislodging of the retardance structures, then a fence structure should be used. However, if a design having the flexibility to shift vertically with the channelbed is desired, jacks or tetrahedrons would be the better choice.

Several typical wire fence retards are illustrated in Figures 64 and 65.

Heavy Timber-pile Bent Retardance Structures

Timber-pile bent retards have all the attributes of wood- fence retardance structures, except that they are generally are much larger. As retardance structures, pile bents are particularly adaptable to large streams and rivers. Pile-bent retardance structures have proven to be particularly useful for alignment problems that occur very near a bridge or roadway embankment, particularly those involving sharp channel bends and direct impingement of flow against a bank. Because of their structural mass, they also are useful in environments characterized by heavy drift, debris, and ice loads, or where there is danger of the structure being damaged by barge or other boat traffic. The primary disadvantage of timber-pile bent structures is their cost. Although there was no direct comparative cost data available for these structures, they can generally be expected to be more expensive than other retardance structures because of their size.

Timber-pile retardance structures may be of single, double, or triple rows of piles with the outside of the upstream row faced with wire mesh or

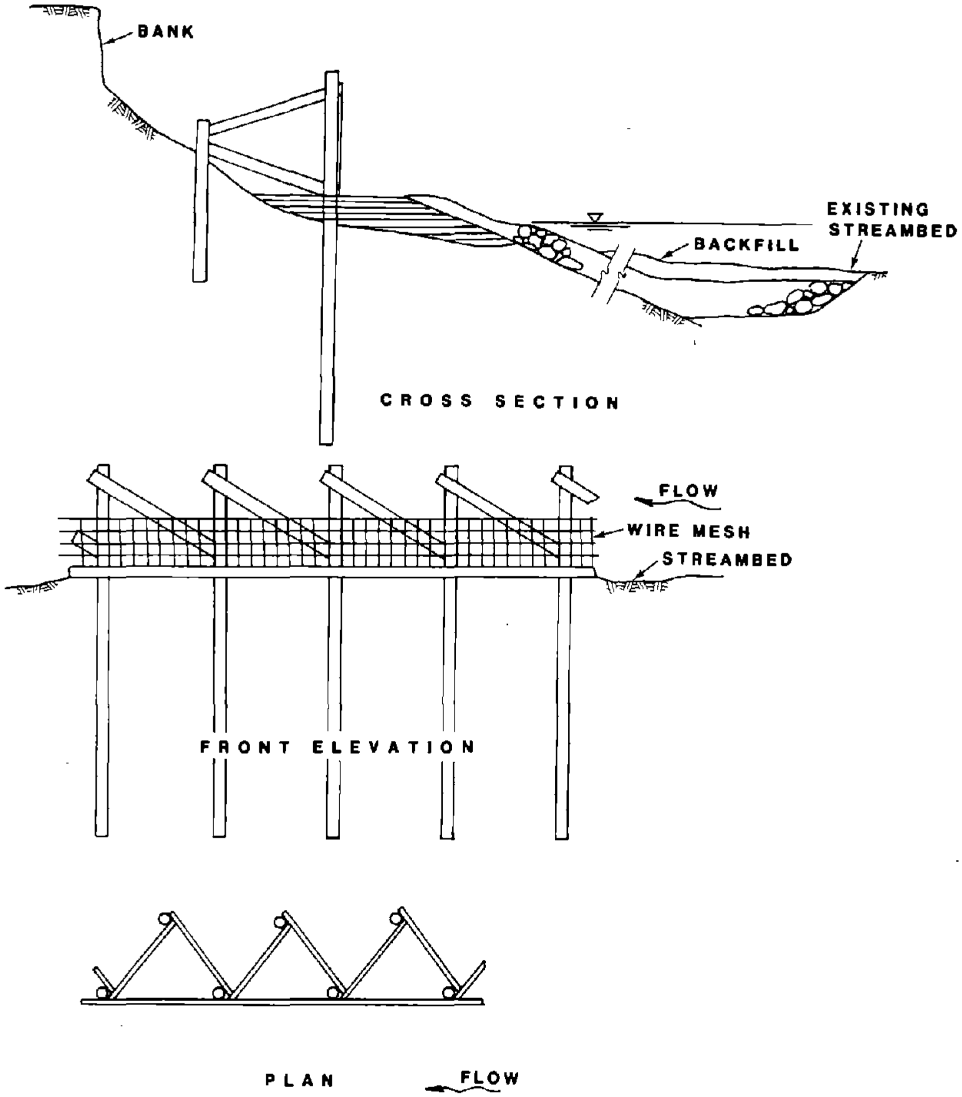


FIGURE 65. HEAVY TIMBER-PILE AND WIRE FENCE RETARDANCE STRUCTURES (MODIFIED FROM U.S. ARMY CORPS OF ENGINEERS, 1981)

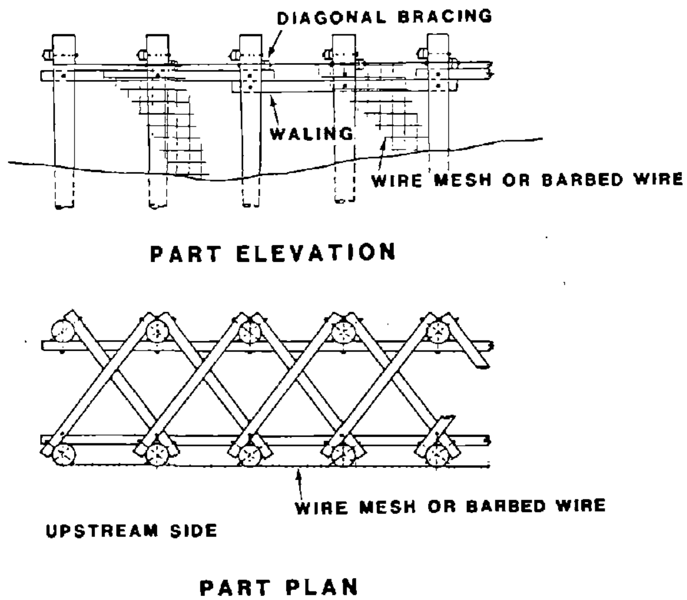


FIGURE 66. TIMBER PILE BENT RETARDANCE STRUCTURE (MODIFIED FROM CALIFORNIA DEPT. OF PUBLIC WORKS, 1970)

other fencing material. These additions add to the retarding effect of the structure, and may even trap light brush or debris to supplement its purpose. The number of pile rows and the amount of wire may be varied to control the deposition of material within and behind the structure. Another design consideration is the depth of penetration needed to avoid loss from scour. California Department of Public Works (1970) recommends that with velocities of 10 to 15 fps and sandy streambeds, piles should be driven to refusal, preferably with a penetration of at least 15 to 20 feet. If there is a question as to the adequacy of the attainable penetration depth, the pile should be protected by a layer of rock placed on the streambed. A typical pile bent retardance structure design detail is shown in Figure 65.

LONGITUDINAL DIKES

Longitudinal dikes are barriers constructed parallel to the bankline or the desired flow alignment for the control or containment of channel or floodplain flows. They differ from linear retardance structures in that they are essentially impermeable to flow conveyance (even though most designs are permeable; they let water pass through, but not any significant current). Longitudinal dikes differ from spurs in that they are continuous along and parallel to the bank or desired flowpath. They are most commonly thought of as flood-control devices constructed on floodplain areas. However, dikes constructed for bank-erosion control and flow control are typically constructed on the channelbed at or near the base of the channelbank, or desired flow path.

Longitudinal dikes can be used for any of the purposes or functions listed in Chapter 3; that is, to protect an existing bankline, to reestablish some previous flowpath or alignment, or to control and/or constrict channel flows. For erosion control purposes, longitudinal dikes function by removing the transporting mechanism (the flowing water) away from an eroding bank. By eliminating the transporting mechanism, it can be said that longitudinal dikes will consequently resist any of the bank weakening/particle displacement mechanisms listed in Table 1. They are, however, best suited for flow control and realignment situations.

The primary advantage of longitudinal dikes is that they provide a smooth, continuous flow-control path throughout the length of realigned channel. Also, in areas subject to river freezing, protective works constructed parallel to the direction of the current are subject to less ice damage than those constructed perpendicular to the flow.

There are three major classifications of longitudinal dikes. They are earth or rock embankment dikes, crib dikes, and rock-toe dikes.

Earth or Rock Embankments

As the name implies, earth or rock embankments are constructed of earth or rock mounded to form an embankment or new channelbank. These embankments usually are faced with riprap or some other revetment material. Embankment dikes are designed to function primarily as flood-flow control and flow-alignment structures; erosion control is a secondary function. By removing the transporting mechanism from the original bank, however, earth or rock embankments can be considered erosion control and bank- stabilization mechanisms.

Embankment dikes usually are constructed in such a way that they are equal or greater in height than the original bank. Because of their size, they are very expensive to construct, and therefore, their use usually is reserved for large channel-realignment projects. Also, most of these structures must be faced with a revetment material to protect against erosion, making them more costly. Where the construction of a large embankment dike is required, impermeable spurs usually will provide adequate protection at a reduced cost.

Other features of embankment dikes are similar to the features discussed above for dikes in general.

Crib Dikes

Longitudinal crib-dike designs consist of a linear structural crib filled with a material that will not allow the passage of flow currents. As is the case with other longitudinal dikes, these structures are constructed at or near the toe of the channelbank and parallel to the bank or desired flow direction. The crib structure has most commonly been constructed of wire fence supported by pipe or wood-pile supports and braces (see Figure 43). However, heavy timber-pile bents, logs, precast concrete beams, and other structural materials have been used; the most common fill material is rock or stone, Other materials have been used, but with limited success. These include straw, hay, brush, and used automobile tires.

Crib dikes function similar to other longitudinal dikes. That is, they function primarily as flow- control and redirection devices, and as such, protect the bank by removing the transporting mechanism. However, crib dikes usually are designed only to protect low banks, or the lower portions of high banks. Upper-bank protection is usually required to protect the upper portions of high banks from surface erosion when the lower bank is protected with a crib dike. This is particularly true at sharp bends where the high energy curvilinear flow currents have been known to attack the upper portions of the bank and outflank the dike structure.

Another common problem occurs when the crib dikes are undermined. This has been observed to be a common failure mechanism when crib dikes are used at sharp bends or in channels with dynamic sand beds (U.S. Army Corps of Engineers, 1981). When scour occurs at the base of the crib to a depth below the level of the cribs restraining members, the fill material can funnel out, greatly reducing the effectiveness of the dike. It often is impractical or uneconomical to run the crib's restraining members to a depth below anticipated scour. Therefore, to avoid the loss of all fill material, a material that has some resistance to transport on its own, such as rock, should be chosen. If a large enough volume of rock is used initially in the dikes, and lost material is replaced after each major flow event, the danger of loss of the structure from undermining can be greatly reduced.

Longitudinal dikes are best suited for use on channels having low to medium height banks where flow control and flow realignment are the primary objectives. They also are useful on narrow channels where flow construction might be a problem with other flow control/bank protection measures, and where the channelbank is not of sufficient integrity to support revetment materials on its own. In addition, they are useful where an embankment is desired, but construction material of sufficient size and integrity is not available. In this case, a wire crib can be used to hold the smaller rock material in place and form the dike.

As discussed above, longitudinal crib dikes can be constructed of a variety of materials. Figures 67, 68, and 69 illustrate three typical designs. Figure 69 includes a detail of a tieback. Bank tiebacks are used to prevent flow from concentrating in the space between the dike and the bank, which could cause additional erosion of the bank and result in outflanking of the structure. Criteria for the spacing of tiebacks are not well-defined, but generally are in the range of two to six times the tieback length.

Cost data for longitudinal crib dikes are generally unavailable. However, the U.S. Army Corps of Engineers (1981) reports two sites; one a hay-filled timber pile and wire crib and the other a used automobile tire-filled timber pile and wire crib, both of which cost approximately $36.00/ft of bank protected. These sites were on channels having widths of less than 200 ft. It could be expected that comparable rock-fill cribs would cost more.

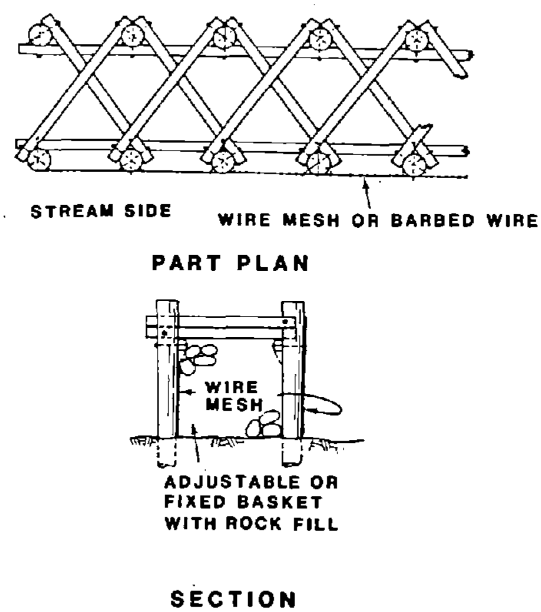


FIGURE 67. TIMBER PILE WIRE MESH STONE FILL CRIB
(MODIFIED FROM CALAIFORNIA DEPT. OF PUBLIC WORKS, 1970)

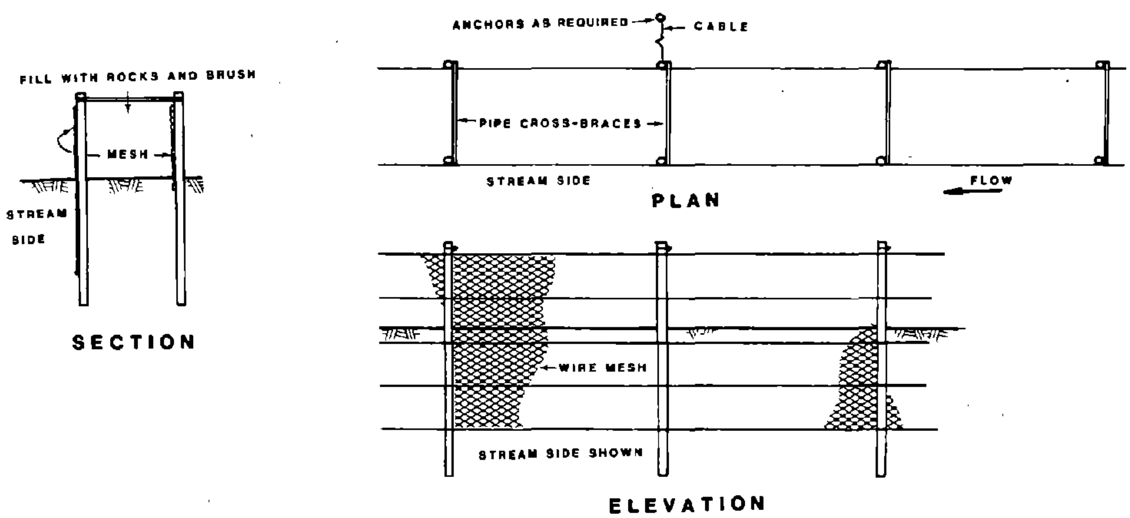


FIGURE 68. DOUBLE WIRE FENCE CRIB
(MODIFIED FROM CALIFORNIA DEPT. OF PUBLIC WORKS, 1970)

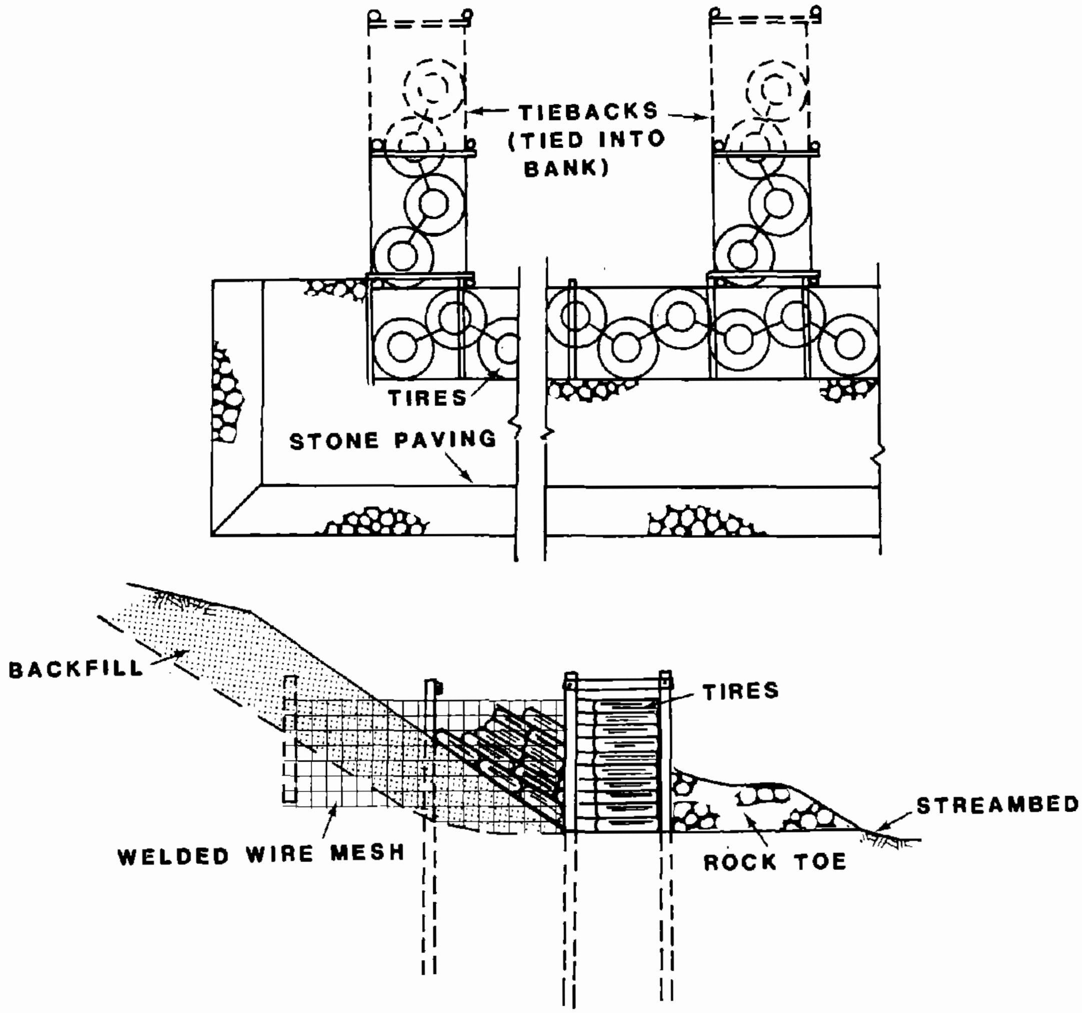


FIGURE 69. TIMBER PILE, WIRE MESH CRIB WITH TIEBACK
(MODIFIED FROM U.S. ARMY CORPS OF ENGINEERS, 1981)

Rock-Toe Dikes

Rock-toe dikes consist of a continuous stone-fill dike of rock riprap placed parallel to the channelbank at its toe or base. These structures have been referred to as reinforced revetments and composite revetments by the U.S. Army Corps of Engineers (1981). They are low structures designed to protect the bank toe from undermining caused by dynamic scour and general channel degradation. As such, they do not protect upper portions of the bank. Where bank erosion above the toe dike is anticipated, other upper-bank treatments are used along with the toe dikes; vegetation or other low-cost revetment treatments often are used. The stone-toe dikes often are accompanied by intermittent stone-fill tiebacks, similar in concept to those discussed for crib dikes. Here again, the stone tiebacks are used to prevent high flows from concentrating land causing erosion of the bank and possible outflanking of the dike structure. A typical longitudinal rock toe dike is illustrated in Figure 70.

Bank-toe erosion was identified as a major cause of bank erosion in Chapter 2 of this document. The U.S. Army Corps of Engineers (1981) found that bank stabilization measures without toe protection rarely were successful, particularly at bends in sand bed channels having more than slight curvature. Also, if general channelbed degradation is apparent or anticipated, toe protection is essential. Of all the bank stabilization measures studied by the Corps of Engineers in the Yazoo Basin, the Corps reports that longitudinal stone dikes provided the most effective toe protection and were the most successful bank-stabilization measures studied in channels having very dynamic and/or actively degrading channelbeds.

When used on their own, rock-toe dikes are best suited for situations where only lower-bank protection is required. More specifically, they are applicable where toe erosion and undermining of the bank is the primary cause of bank-material loss. However, because of their success at providing toe protection, rock-toe dikes have been used in combination with other bank-protection schemes as well as on their own. As mentioned above, they often are used in combination with revetments, particularly vegetative treatments, and other low cost types. They also have been used in combination with retardance structures, and embankment and crib dikes.

The primary advantages of rock-toe dikes are their effectiveness, their economy, their ease of construction and maintenance, and the almost universal availability of rock construction materials. Their parallel design and low profile also makes them less susceptible to damage from floating debris and ice, and makes them less of a flow constriction. This last item makes rock toe dikes useful on small, narrow channels where it is necessary to maintain as wide a conveyance channel as possible. They also are easily constructed along irregularly eroding banks and will generally not require excessive bank preparation or grading (when used alone). Also, since the bank-line structure is relatively low, the structure will blend in with the bank and be less visible than other treatments, leaving a natural appearing bank-line.

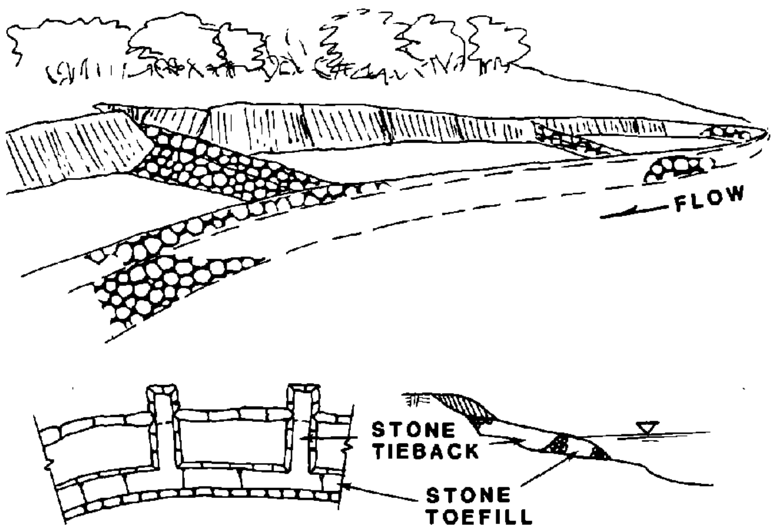


FIGURE 70. LONGITUDINAL ROCK TOE TIEBACKS

Stone-toe dikes also have been found to be effective against wave erosion in backwater impoundment areas such as above dams and locks. In these instances they should be designed with crest heights higher than wave-plus-splash height.

A disadvantage in the use of rock toe dikes is that they can require a sizable volume of stone to provide adequate protection along the entire length of the eroding bank. This could become costly, depending on the size of the river. In some cases a spur design would be more economical since they only require sufficient material volume at the riverward tip to resist undermining, and not along the entire length of the bank. This is an economic consideration and must be evaluated on a case-by- case basis. Another disadvantage in the use of rock-toe dikes is that when the toe-crown elevation is constructed to a design height less than the normal water surface, the structure will present a near-bank hazard to small boats.

As is indicated in Figure 49, stone-toe dikes (with and without tiebacks) are among the less expensive bank-stabilization structures available. The costs analysis of longitudinal toe dikes was based on review of data from 105 sites. These sites were located on rivers as large as the Missouri to the small streams of the Yazoo River Basin (100 - 150 ft in width). The average cost based on the 105 sites was $94.07/ft of bank protected. The reported high and low costs were $171/ and $18/ft of bank protected. Figure 49 shows the costs broken down into two classifications: longitudinal stone dikes with tiebacks and without tiebacks. As would be expected, longitudinal dikes with tiebacks are more expensive than those without. Cost data was reported for 91 sites having tiebacks. The average cost of these structures was $97.71/ft with a reported high of $215, and a reported low of $34/ft. Fourteen sites without tiebacks also were described. The average cost of these sites was $70.14/ft; the high and low costs reported were $148 and $18/ft of bank protected, respectively.

Review of Table 2 indicates that the construction costs typically associated with longitudinal toe dikes are more per foot of bank protected than spurs, but less than bulkheads and rigid revetments. The cost of longitudinal toe dikes with tiebacks are, on the average, comparable to costs for retardance structures and flexible revetments. However, longitudinal toe dikes without tiebacks generally are a little less expensive to construct than either retardance structures or flexible revetments.

Typical design details for longitudinal stone-toe dikes were illustrated in Figure 70. As discussed above, toe dikes can be designed with or without tiebacks, depending on the specific application. If the toe dike is not constructed directly against the toe of the existing bank, it is recommended that tiebacks be used. The volume of rock required for the dike will depend on specific site conditions. The primary consideration is the anticipated depth of scour. A volume of material equal to 1-1/2 to 2 times the volume that would be required to armor the sides of the anticipated toe scour to a thickness of 1-1/2 times the diameter of the largest stone specified. The rock fill for the dike should be placed from an elevation of slightly above the normal water surface to up to 2 feet above the normal water surface, depending upon the frequency and duration of high flow fluctuations. If waves are a problem, this height should be increased to cover the anticipated splash zone. Criteria for establishing the stone size should be similar to that presented earlier for standard riprap revetments. The generally accepted cross-sectional geometry for rock-toe dikes (as well as their tiebacks) is a trapezoidal or peaked shape. Figure 71 shows a comparison of typical geometries for longitudinal toe dikes.

BULKHEADS

Brice et al. (1978) define bulkheads as a "steep or vertical wall against a natural or artificial slope, for the purpose of supporting the slope and/or protecting it from erosion." Bulkheads differ from revetments in that they support the bank instead of being supported by the bank. Retards and longitudinal dikes differ from bulkheads in that they provide no bank support. Also, bulkheads differ from retardance structures in that they are generally impermeable.

Bulkheads most often are used as lower-bank and toe protection. Placed at the foot of a slope, these structures help to stabilize the slope against mass movement and protect the toe and face of the slope against scour and erosion. A toe wall at the foot of a slope permits local oversteepening of the slope at its base and flattening of the slope above (see Figure 72). The latter makes it possible to establish vegetation on the slope and reduces erosion potential; the former reduces the amount of clearance required between the base of the slope and the top of the slope. As toe protection, bulkheads often are used in combination with another countermeasure that provides upper- bank protection. Bulkheads are most frequently used at

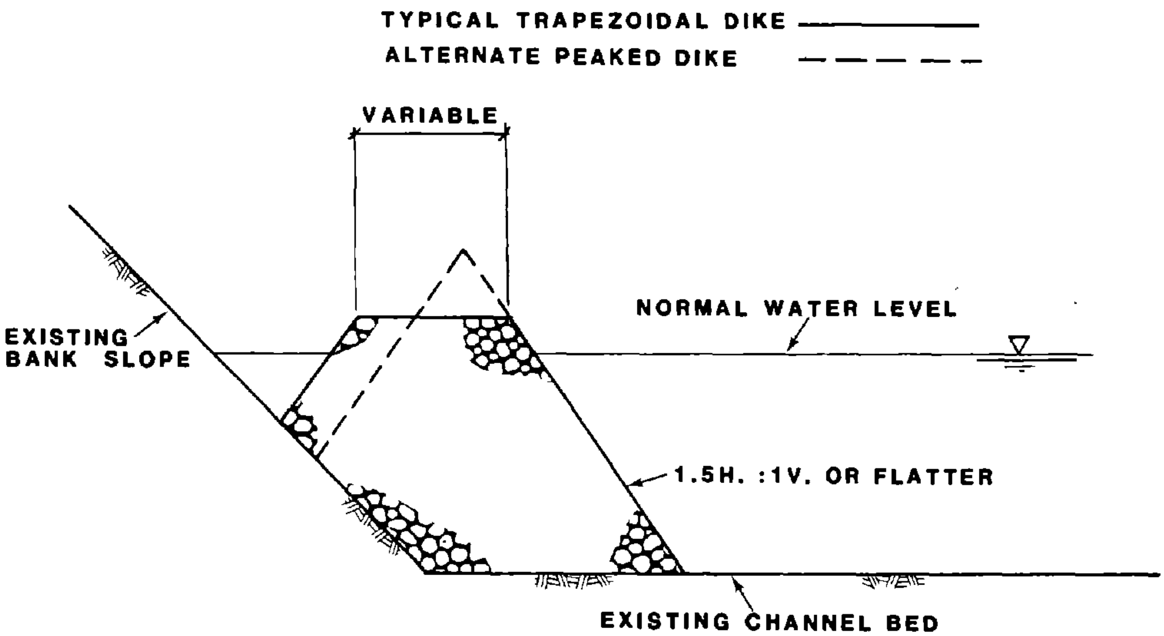


FIGURE 71. TYPICAL LONGITUDINAL ROCK TOE DIKE GEOMETRIES.

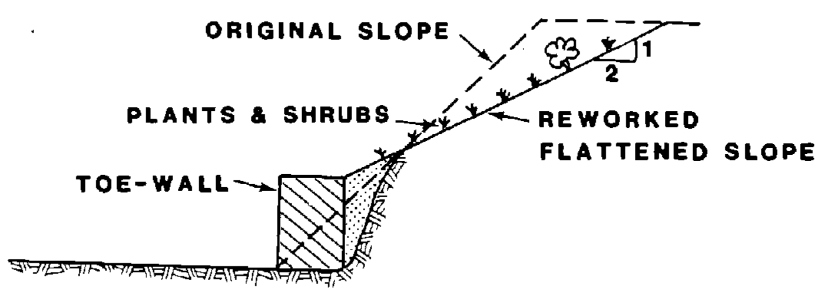


FIGURE 72. LOW TOE WALL THAT PERMITS SLOPE FLATTENING AND ESTABLISHMENT OF VEGETATION ON SLOPE ABOVE.

bridge abutments to protect them from slumping and undermining. They also provide additional support to abutment foundations. In addition, bulkheads have proven to be particularly effective for situations where economy of space is important; for example, where there is not room to construct other types of stabilization structures. They are also useful to provide a transition between the streambanks and bridge opening where stream alignment is poor or to provide a smooth transition around a bend.

Many design types of bulkheads have been used. Using a classification scheme based on construction methods and materials, the following types of bulkheads have been identified:

- Concrete or masonry walls
- Crib or bin walls
- Sheet bulkheads
- Pile bulkheads
- Stacked walls
- Reinforced Earth (registered trademark) walls
- Tie-back walls

Also, within each of these categories several design and material variations exist. Of the types listed, all but stacked-wall bulkheads strictly fit the definition of bulkheads given previously. Stacked- wall types are a cross between revetments and true bulkheads because they sometimes are partially supported by the bank, and generally do not support the bank behind them. However, because of their steep angle and the fact that they are supported primarily through their own structure, they are included in this classification.

As was indicated above, the selection of a suitable bulkhead structure entails a wide variety of choices. The selection of a suitable bulkhead design will depend upon such considerations as site constraints, availability of materials, appearance of the wall, ease of construction, and costs. Most of these designs are suitable as earth retaining walls as well as bulkheads for the protection and stabilization of channelbanks. The most commonly used designs for channel stabilization are concrete or masonry walls, crib or bin walls, sheet bulkheads, and stacked walls. The other design types usually are used as other types of embankment support.

Concrete and masonry walls generally are constructed from stone or concrete. They resist erosion from hydraulic forces and earth pressure by their weight and mass. These walls are essentially monolithic and must be designed with seeps or some other drainage mechanism for releasing hydrostatic pressures that can build up in the bank behind them. These walls must also be capable of resisting other external forces such as overturning, bending, and sliding. If high walls are needed, cantilever and counterfort walls can be used. These walls, are constructed from reinforced concrete and can be built to heights up to 30 feet. Figure 73 shows sketches of typical concrete and masonry wall designs. Additional design and construction details for concrete and masonry wall bulkheads can be found in California Department of Public Works (1970) and Gray and Leiser (1982).

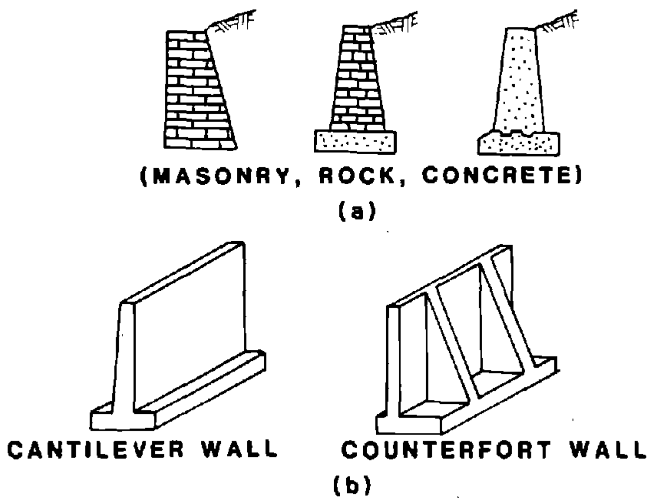


FIGURE 73. TYPICAL CONCRETE AND MASONRY WALL DESIGNS

Timber and concrete crib walls have been used for bulkheads in locations where some flexibility is permissible or desired. A crib is basically a structure formed by joining a number of cells together and filling them with soil or rocks to give them strength and weight. In crib structures the members are essentially assembled in "log cabin" fashion. The frontal, horizontal members are termed stretchers; the lateral, vertical members are termed headers. The structural mass of the crib is provided by the backfill material. Crib structures usually are economical because they can usually be constructed of locally available materials. As with other types of bulkheads, backfill used for cribs must be self-draining and secure against erosion through the louvers of the stretcher system. Of particular importance is security of the base of the crib from loss of backfill due to scour along its toe. A typical crib design is illustrated in Figure 74. Additional design and construction information for various crib designs can be found in Gray and Leiser (1982).

Timber, concrete, asbestos fiber and metal piling have been used for bulkheads. Any of these materials is adaptable to sheet piling. Asbestos fiber and metal sheet bulkheads are available from commercial sources; most commercial designs are patented. Sheets are either worked into the soil with a compressed air jet or are driven into the ground with mechanical drivers. The stability of these structures depends on the depth of penetration and type and strength of the supporting foundation materials. The structural design of pile bulkheads is highly specialized and not adaptable to standard plans. A sketch showing a metal sheet-pile bulkhead geometry is presented in Figure 75. Additional discussions of pile designs can be found in California Department of Public Works (1970) and Mineral Fiber Products Bureau (1966).

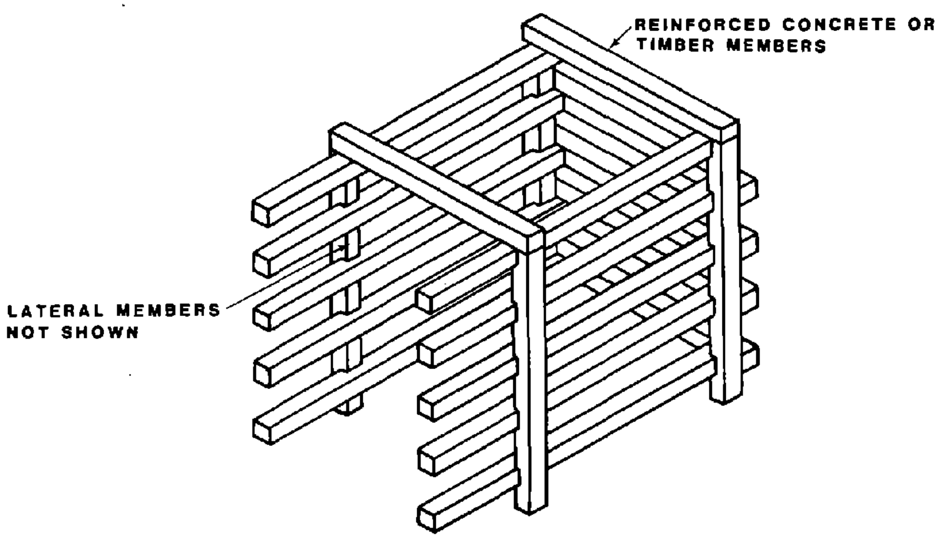


FIGURE 74. CRIB WALL

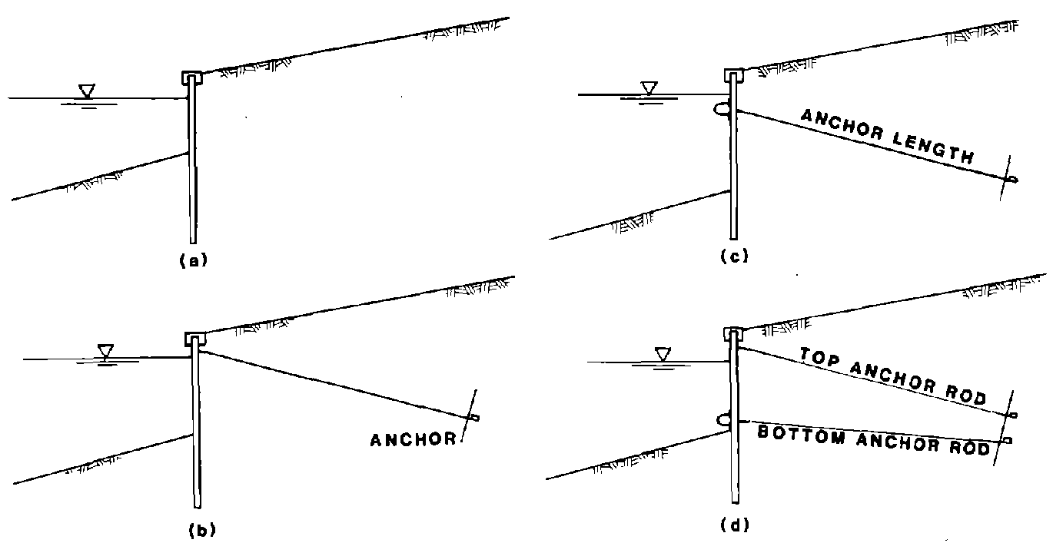


FIGURE 75. TYPICAL GEOMETRIC DESIGN FOR SHEETPILE BULKHEAD

During The U.S. Corps of Engineers Section 32 Program, two types of stacked-wall bulkheads were tested. These included used automobile tire wall and gabion wall designs. As discussed previously, stacked wall designs actually are a cross between revetments and bulkheads since they do not provide structural support for the bank. The Corp's evaluation found that both used automobile tire walls and gabion walls provided acceptable protection. Tire walls and gabion walls were found applicable for bank toe and lower-bank protection; gabion walls have been found applicable for high banks as well. However, both types of stacked walls have been found to be prone to vandalism in urban areas. This is particularly true of the used automobile tire wall. Used automobile tire walls should be constructed by stacking the tires in a staggered arrangement and filling them with gravel. They are also usually placed on a rock or gravel toe or base. Figure 76 illustrates a typical used automobile tire wall design. Several different design configurations are possible with gabions. They may have either battered or stepped-back fronts. The choice of type depends upon application, although the stepped-back type generally is easier to build when the wall is more than 10 feet high. The number and arrangement of gabion units also depend upon whether a level or an inclined backfill is used behind the wall. Figure 77 illustrates two design configurations. Additional design information and details can be found in Gray and Leiser (1982), and in gabion manufacturers' literature.

Reported installed costs for bulkheads range widely. Costs reported in the literature ranged from \$23/ft of bank protected to over \$300/ft of bank protected. The cost analysis discussed in Chapter 4 was based on sites documented by the U.S. Army Corps of Engineers (1981) and indicates costs of bulkhead designs ranging from \$100/ft to \$306/ft of bank protected. Construction costs for stacked used tire and gabion bulkheads are reported in Figure 49. The average cost for the six sites reporting the use of stacked tire bulkheads was \$159.16/ft. The reported low and high values were \$111/ft and \$264/ft of bank protected. The average reported cost for the six gabion bulkheads was \$183.66/ft of bank protected. The low and high values were \$100/ and \$288/ft of bank protected. This places these structures in the middle of the cost spectrum for all erosion/flow control countermeasures. It should be noted, however, that used automobile tire and gabion bulkheads are among the least expensive bulkhead designs. This is evidenced in Table 2, where a direct comparison between gabion bulkheads and concrete cribwall bulkheads on the same channelbend is reported. In this case the gabion design cost \$179/ft to construct, and the concrete cribwall cost \$306/ft to construct. The U.S. Army Corps of Engineers (1981) also reports cost data for an additional concrete crib installation and a timber-crib installation. The reported construction costs of these installations was \$330/ft and \$241/ft of bank protected, respectively. Based on this additional information, it is evident that most bulkhead designs will usually be in the mid- to upper- price range when compared with other countermeasure types.

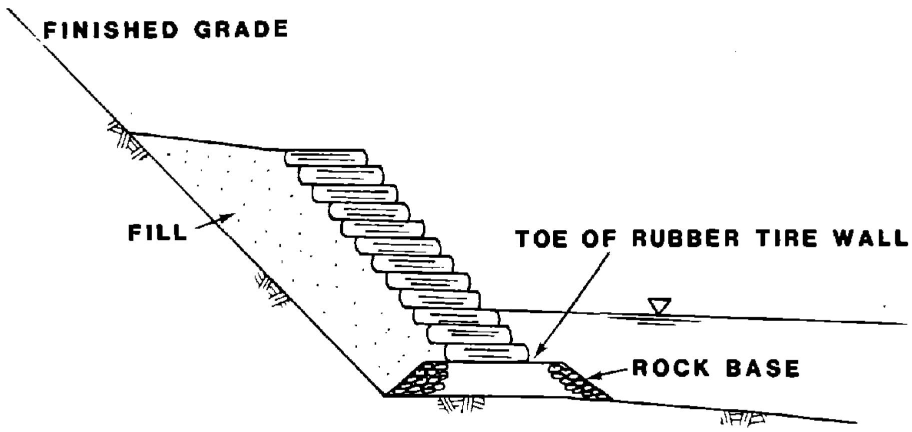


FIGURE 76. TYPICAL TIRE WALL BULKHEAD

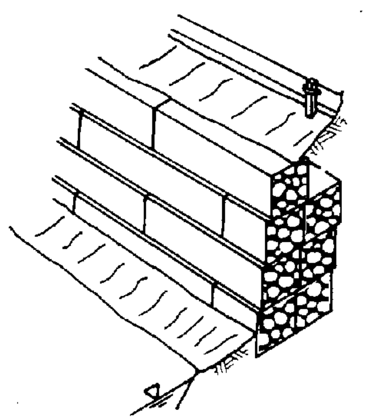


FIGURE 77. TYPICAL GABION WALL BULKHEADS

As is implied above, bulkheads usually will be one of the more expensive alternatives for bank stabilization and flow control. Therefore, they are generally only economically justifiable for situations where there is not sufficient room to construct some other less expensive structure. Several example situations where the added expense of bulkheads might be justified are as follows:

- In situations where the stream flows directly alongside a highway embankment in a confined valley.
- In situations where urban development has encroached on the stream to a point where there is only enough room for the steep vertical banks bulkheads afford.
- Where encroachment on the channel dictates that the bank stabilization structure provide structural support to the bank for some other construction activity, such as the construction of a roadway, bridge abutment, of some other structure.
- In cases where valuable riparian property or improvements immediately adjacent to the streambank must be protected.

The design of bulkheads consists of the following components:

- evaluation of foundation condition,
- choice of material and design configuration,
- determination of line and grade, and
- structural design.

As was discussed with other countermeasure designs, allowance must be made for the increased hazard from scour at the toe and at the downstream limit of bulkhead schemes. This is a primary concern when selecting the type of foundation, grade of footing, penetration of piling, and transition and anchorage at the downstream end. Another consideration is the permeability of backfill; backfill for all bulkheads, particularly crib types, must be self-draining and secure against erosion through structural layers. Standard bulkhead designs have been developed by and are available from a number of sources; these include manufacturers of retaining-wall systems (e.g., gabions, cribwalls,etc.), trade associations (e.g., American Wood Preservers Institute), and other State and Federal agencies (e.g., U.S. Forest Service, U.S. Soil Conservation Service, U.S. Army Corps of Engineers, etc.).

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FEDERALLY COORDINATED PROGRAM (FCP) OF HIGHWAY RESEARCH, DEVELOPMENT, AND TECHNOLOGY

The Offices of Research, Development, and Technology (RD&T) of the Federal Highway Administration (FHWA) are responsible for a broad research, development, and technology transfer program. This program is accomplished using numerous methods of funding and management. The efforts include work done in-house by RD&T staff, contracts using administrative funds, and a Federal-aid program conducted by or through State highway or transportation agencies, which include the Highway Planning and Research (HP&R) program, the National Cooperative Highway Research Program (NCHRP) managed by the Transportation Research Board, and the one-half of one percent training program conducted by the National Highway Institute.

The FCP is a carefully selected group of projects, separated into broad categories, formulated to use research, development, and technology transfer resources to obtain solutions to urgent national highway problems.

The diagonal double stripe on the cover of this report represents a highway. It is color-coded to identify the FCP category to which the report's subject pertains. A red stripe indicates category 1, dark blue for category 2, light blue for category 3, brown for category 4, gray for category 5, and green for category 9.

FCP Category Descriptions

1. **Highway Design and Operation for Safety**
   Safety RD&T addresses problems associated with the responsibilities of the FHWA under the Highway Safety Act. It includes investigation of appropriate design standards, roadside hardware, traffic control devices, and collection or analysis of physical and scientific data for the formulation of improved safety regulations to better protect all motorists, bicycles, and pedestrians.

2. **Traffic Control and Management**
   Traffic RD&T is concerned with increasing the operational efficiency of existing highways by advancing technology and balancing the demand-capacity relationship through traffic management techniques such as bus and carpool preferential treatment, coordinated signal timing, motorist information, and rerouting of traffic.

3. **Highway Operations**
   This category addresses preserving the Nation's highways, natural resources, and community attributes. It includes activities in physical maintenance, traffic services for maintenance zoning, management of human resources and equipment, and identification of highway elements that affect the quality of the human environment. The goals of projects within this category are to maximize operational efficiency and safety to the traveling public while conserving resources and reducing adverse highway and traffic impacts through protections and enhancement of environmental features.

4. **Pavement Design, Construction, and Management**
   Pavement RD&T is concerned with pavement design and rehabilititation methods and procedures, construction technology, recycled highway materials, improved pavement binders, and improved pavement management. The goals will emphasize improvements to highway performance over the network's life cycle, thus extending maintenance-free operation and maximizing benefits. Specific areas of effort will include material characterizations, pavement damage predictions, methods to minimize local pavement defects, quality control specifications, long-term pavement monitoring, and life cycle cost analyses.

5. **Structural Design and Hydraulics**
   Structural RD&T is concerned with furthering the latest technological advances in structural and hydraulic designs, fabrication processes, and construction techniques to provide safe, efficient highway structures at reasonable costs. This category deals with bridge superstructures, earth structures, foundations, culverts, river mechanics, and hydraulics. In addition, it includes material aspects of structures (metal and concrete) along with their protection from corrosive or degrading environments.

9. **RD&T Management and Coordination**
   Activities in this category include fundamental work for new concepts and system characterization before the investigation reaches a point where it is incorporated within other categories of the FCP. Concepts on the feasibility of new technology for highway safety are included in this category. RD&T reports not within other FCP projects will be published as Category 9 projects.

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