CALIFORNIA
DEPARTMENT OF TRANSPORTATION

manual change transmittal

TITLE		APPROVED BY	NO.
	DIVISION OF DESIGN HIGHWAY DESIGN MANUAL SIXTH EDITION – CHANGE 07/15/16	TIMOTHY CRAGGS, Chief	Date Issued: 07/13/16 Page 1 of 1
SUBJECT AREA	Table of Contents; List of Figures; List of Tables; Chapters: 870, 880; and, Index	ISSUING UNIT DIVISION OF DESIGN	
SUPERCEDES	SEE BELOW FOR SUMMARY DESCRIPTION	DISTRIBUTION **ALL HOLDERS OF THE 6TH EDITION, HIGHWAY DESIGN MANUAL**	

The Table of Contents; List of Figures; List of Tables; Chapters: 870, 880; and the Index of the Sixth Edition, Highway Design Manual (HDM) have been revised. The changes to the HDM are summarized below with change sheets available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm.

Changes reflect the reorganization of river, stream, and natural channel bank protection guidance within its own Chapter 870. New Chapter 880 has been added to focus on coastal shore and inland lake bank protection guidance. In addition:

- New discussions on geomorphology, stream processes, bio-diversity and sustainability are included in Chapter 870.
- Revised rock slope protection (RSP) guidance based on FHWA Hydraulic Engineering Circular (HEC) No. 23 has been included in Chapter 870, presenting guidelines for a range of applications including RSP on streams and river banks, bridge piers and abutments, and bridge scour countermeasures such as guide banks and spurs.
- New Chapter 880 includes a primary focus on quantifying and mitigating exposure of coastal and inland lakes to sea level rise, storm surge, and wave action.

These changes are consistent with comprehensive FHWA guidance documents (i.e., HEC 23 and HEC 25), NCHRP's bio-technical methods, Caltrans Design Information Bulletin No. 87 for Hybrid RSP, and Caltrans Revised Standard Specifications and new BEES item codes for Section 72-2 "Rock Slope Protection".

Per the July 15, 2016, Design Memorandum entitled "IMPLEMENTATION OF ROCK SLOPE PROTECTION (RSP) DESIGN", "**All new projects that include RSP starting August 1ST, 2016 shall be designed using the new guidelines and gradations.** Additionally, current projects not already at the 30 percent PS&E stage shall be designed using the new guidelines and gradations. All other current projects may continue to use the California Bank and Shore (CaBS) RSP layered design methodology and existing Standard Specifications and BEES item codes."

HDM Holders are encouraged to use the most recent version of the HDM available on-line at the above website. Should a HDM Holder choose to maintain a paper copy, the Holder is responsible for keeping their paper copy up to date and current. Using the latest version available on-line will ensure proper reference to the latest design standards and guidance. If you would like to be notified automatically of any significant changes or updates to the HDM, go to http://www.dot.ca.gov/hq/oppd/hdm/hdmlist.htm.

Enclosures available on the Department Design website at: http://www.dot.ca.gov/hq/oppd/hdm/hdmtoc.htm.

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CHAPTER 870
BANK PROTECTION - EROSION CONTROL

Topic 871 - General

Index 871.1 - Introduction

Highways, bikeways, pedestrian facilities and appurtenant installations are often attracted to parallel locations along man-made channels, streams, and rivers. These locations may be effected from the action of flowing water, and may require protective measures.

Bank protection can be a major element in the design, construction, and maintenance of highways. This section deals with procedures, methods, devices, and materials commonly used to mitigate the damaging effects of flowing water on transportation facilities and adjacent properties. Potential sites for such measures should be reviewed in conjunction with other features of the project such as long and short term protection of downstream water quality, aesthetic compatibility with surrounding environment, and ability of the newly created ecological system to survive with minimal maintenance. See Index 110.2 for further information on water quality and environmental concerns related to erosion control. See Chapter 880 for shore protection along coastal zones and lake shores that are subjected to wave attack.

Refer to Index 806.2 for definitions of drainage terms.

871.2 Design Philosophy

In each district there should be a designer or advisor, usually the District Hydraulic Engineer, knowledgeable in the application of bank protection principles and the performance of existing works. Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from involvement with Design, Environmental, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802). For channel and habitat characterization and assessment relative to design and obtaining project specific permits, the designer may also require input from fluvial geomorphologists (or engineers with geomorphology training), geologists and biologists. The District Hydraulic Engineer will typically be able to assist with flood analysis, water surface elevations/profiles, shear stress computations, scour analysis, and hydraulic analysis for placement of in-stream structures. A geomorphologist can provide input regarding characterization of channel form and dominant geomorphic processes and hydraulic geometry relationships such as an analysis of lateral and longitudinal channel adjustment. The geomorphologist can also make an identification of the processes responsible for forming and maintaining key habitats and assist in making an assessment of the long-term project effects.

There are a number of ways to deal with the problem of bank erosion as follows:

- Although not always feasible or economical, the simplest way and generally the surest of success and permanence, is to locate the facility away from the erosive forces. Locating the facility to higher ground or solid support should be considered, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for bank protection.
- The most commonly used method of bank protection is with a more resistant material like rock slope protection. Other protection methods (e.g., training systems) are discussed in Index 873.4 and summarized in Table 872.1.
- A third method is to reduce the force of the attacking water. This is often done by various plantings such as willows. Plantings once established not only reduce stream velocity near the bank during heavy flows, but their roots add structure to the bank material.
- Another method is to re-direct flows away from the embankment. In the case of stream attack, a new channel can be created or the stream can be diverted away from the embankment by the use of baffles, deflectors, or spurs.

Combinations of the above four methods may be used. Even protective works destroyed in floods have proven to be effective and cost efficient in minimizing damage to transportation facilities.

Design of protective features should be governed by the importance of the facility and appropriate design principles. Some of the factors which should be considered are:

- *Roughness*. Revetments generally are less resistant to flow than the natural channel bank. Channel roughness can be significantly reduced if a rocky vegetated bank is denuded of trees and rock outcrops. When a rough natural bank is replaced by a smooth revetment, the current is accelerated, increasing its power to erode, especially along the toe and downstream end of the revetment. Except in narrowed channels, protective elements should approximate natural roughness and simulate the effect of trees and boulders along natural banks and in overflow channels.

- *Undercutting*. Particular attention must be paid to protecting the toe of revetments against undercutting caused by the accelerated current along smoothed banks, since this is the most common cause of bank failure.

- *Standardization*. Standardization should be a guide but not a restriction in designing the elements and connections of protective structures.

- *Expendability*. The primary objective of the design is the security of the transportation facility, not security of the protective structure. Less costly replaceable protection may be more economical than expensive permanent structures.

- *Dependability*. An expensive structure is warranted primarily where transportation facilities carry high traffic volumes, where no reasonable detour is available, or where facility replacement is very expensive.

- *Longevity*. Short-lived structures or materials may be economical for temporary situations. Expensive revetments should not be placed on banks likely to be buried in widened embankments, nor on banks attacked by transient meander of mature streams.

- *Rock Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity of stone is a major factor in bank protection and the specified minimum should not be lowered without increasing the mass of stones. See Index 873.3(3)(a)(2)(b) for equation to estimate rock size.

- *Selection*. Selection of class and type of protection should be guided by the intended function of the installation.

- *Limits*. Horizontal and vertical limits of protection should be carefully designed. The bottom limit should be secure against toe scour. The top limit should not arbitrarily be at high-water mark, but above it if overtopping would cause excessive damage and below it if floods move slowly along the upper bank. The end limits should reach and conform to durable natural features or be secure with respect to design parameters.

871.3 Selected References

Hydraulic and drainage related publications are listed by source under Topic 807. References specifically related to slope protection measures are listed here for convenience.

(a) FHWA Hydraulic Engineering Circulars (HEC) -- The following seven circulars were developed to assist the designer in using various types of slope protection and channel linings:

- HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006)
- HEC 15, Design of Roadside Channels with Flexible Linings (2005).
- HEC 18, Evaluating Scour at Bridges (2012)
- HEC 20, Stream Stability at Highway Structures (2012)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
- HEC 25, Highways in the Coastal Environment (2008 with 2014 supplement)
- HEC 26, Culvert Design for Aquatic Organism Passage (2010)

(b) FHWA Hydraulic Design Series (HDS) No. 6, River Engineering for Highway Encroachments (2001) -- A comprehensive treatise of natural and man-made impacts and responses on the river environment, sediment transport, bed and bank stabilization, and countermeasures.

(c) AASHTO Highway Drainage Guidelines -- General guidelines for good erosion control practices are covered in Volume III - Erosion and Sediment Control in Highway Construction

(d) AASHTO Drainage Manual (2014) – Refer to Chapters; 11 – Energy Dissipators; 16 – Erosion and Sediment Control; 17 – Bank Protection. The manual provides guidance on engineering practice in conformance with FHWA's HEC and HDS publications and other nationally recognized engineering policy and procedural documents.

(e) U.S. Army Corps of Engineers EM 1110-2-1601 Hydraulic Design of Flood Control Channels Manual.

(f) California Department of Fish and Wildlife California Salmonid Stream Habitat Restoration Manual.

Topic 872 - Planning and Location Studies

872.1 Planning

The development of sustainable, cost effective and environmentally friendly protective works requires careful planning and a good understanding of both the site location and habitat within the stream reach and overall watershed. Planning begins with an office review followed by a site investigation.

Google Earth can be a useful tool for determining site location, changes to stream planform (pattern), bend radius to channel width ratio (to estimate rock size per Index 873.3(3)(a)(2)(b), and location within the overall watershed. USGS StreamStats will facilitate simple watershed delineation and provide basin characteristics such as area, cover and percentage of impervious cover, average elevation, stream slope, mean annual precipitation, and peak flow from regression equations. When more detailed watershed delineation is required, United States Geological Survey (USGS) 7.5-minute quadrangle maps are used to trace the tributary area and sub-basins. The USGS maps are found in graphic image form, such as TIFF and JPEG, and are also found in the form of a Digital Elevation Model (DEM). A DEM contains x-y-z topographic data points usually at 10 or 30-meter grid intervals, where ”x” and “y” represent horizontal position coordinates of a topographic point and “z” is its elevation. These data files and the USGS 7.5-minute quadrangle image files can be imported into software programs, including the Watershed Modeling System (WMS), AutoCAD Civil 3D, and ArcGIS.

Nearby bridges that are located along the same stream reach should be reviewed for site history and changes in stream cross-section. All bridge files are located in the Division of Maintenance, Office of Structures Maintenance.

District biologist staff should be consulted early on during the project planning phase for subject matter expertise regarding fisheries, habitat, and wildlife and to perform an initial stream habitat assessment.

Contact information for Department biologists can be accessed through the CalBioRoster.

For channel and habitat characterization and preliminary assessment relative to design and acquisition of project specific permits, the initial site investigation team should include the project engineer, the district hydraulic engineer, and a biologist. Depending on the complexity of the project, it may be necessary to include Caltrans staff that are trained to perform a geomorphic assessment and/or a geologist during the site investigation.

The selection of the type of protection can be determined during or following the site investigation. For some sites the choice is obvious; at other sites several alternatives or combinations may be applicable. See the FHWA's HDS No. 6, River Engineering for Highway Encroachments for a complete and thorough discussion of hydraulic and environmental design considerations associated with hydraulic structures in moveable boundary waterways.

Some specific site conditions that may dictate selection of a type of protection different from those shown in Table 872.1 are:

- Available right of way.
- Available materials.
- Possible damage to other properties through streamflow diversion or increased velocity.
- Environmental concerns.

- Channel capacity or conveyance.
- Conformance to new or existing structures.
- Provisions for side drainage, either surface waters or intersecting streams or rivers.

The first step is to determine the limits of the protection with respect to length, depth and the degree of security required. For more detailed stream reconnaissance considerations, see HEC 20, Index 4.2.1 (Appendix C and D) and the FHWA's HDS No. 6, River Engineering for Highway Encroachments (Table 8.1).

Considerations at this stage are:

- The severity of stream attack.
- The present alignment of the stream or river and potential meander changes.
- The ratio of cost of highway replacement versus cost of protection.
- Whether the protection should be permanent or temporary.
- Analysis of foundation and materials explorations.
- Access for construction.
- Bank slope (H:V).
- Bed and bank material gradations.
- Stream stability (lateral and vertical). Caltrans Hydromodification Requirements Guidance Storm Water Best Management Practices Rapid Assessment of Stream Crossings Higher Level Stream Stability Analysis presents 13 channel characteristics that are indicators of present stream stability. See Index 4.1.
- Local stream profile.
- Vegetation type and location.
- Physical habitat (temperature, shade, pools, riffles, sediment supply).
- Toe scour/bank failure mode (see Table 872.2).
- Thalweg location.
- Hardpoint location(s).
- Total length of protection needed.

The second step is the selection and layout of protective elements in relation to the highway facility.

872.2 Class and Type of Protection

Protective devices are classified according to their function. They are further categorized as to the type of material from which they are constructed or shape of the device. For additional information on specific material types and shapes see Topic 873, Design Concepts.

There are two basic classes of protection, armor treatment and training works. Table 872.1 relates different location environments to these classes of protection.

872.3 Geomorphology and Site Consideration

The determination of the lengths, heights, alignment, and positioning of the protection are affected to a large extent by the facility location environment.

An evaluation is required for any proposed highway construction or improvement that encroaches on a floodplain. See Topic 804, Floodplain Encroachments for detailed procedures and guidelines.

(1) *Geomorphology*. An understanding of stream morphology is important for identifying both stream instability and associated habitat problems at highway-stream locations. A study of the plan and profile of a stream is very useful in understanding stream morphology. Plan view appearances of streams are varied and result from many interacting variables. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a highway crossing or encroachment. This is particularly true for alluvial streams. Conversely, a highway crossing or encroachment can inadvertently change multiple variables such as Manning's "n-value", channel width, and average velocity, which may adversely affect the stream.

Chapter 2 in HEC 20 presents an overview of general landform and channel evolutionary processes to illustrate the dynamics of alluvial channel systems. It discusses lateral stability, factors effecting bed elevation changes, and the

Table 872.1

Guide to Selection of Protection

Location	Armor										Training										
	Flexible				Rigid						Guide Banks				Bendway Weirs and Spurs				Check Dams		
			Mattresses																		
	Vegetation	Rip Rap	Gabions	Conc. F	Rock	Grouted Rock	Conc. Rock	Conc. Lined	Cribs	Bulk Heads	Earth	Rock	Piling	Other	Rock	Grouted Rock	Piling	Other	Drop Structure	Piling	Rock
Cross Channel																					
Young Valley		X				X	X		X	X											
Mature Valley		X				X	X		X	X	X		X	X	X	X			X		X
Parallel Encroachment																					
Young Valley		X				X	X		X	X											
Mature Valley	X	X				X	X		X	X	X		X	X	X	X	X	X	X		X
Desert-wash																					
Top debris cone		X				X	X			X	X										
Center debris cone		X				X	X												X		X
Bottom debris cone		X				X	X												X		X
Overflow and Floodplain	X	X				X					X		X	X							
Artificial Channel or Roadside Ditch (Ch. 860)	X	X	X	X	X	X	X	X													
Culvert																					
Inlet		X				X				X											
Outlet		X				X				X											
Bridge																					
Abutment		X				X		X													
Upstream		X				X					X	X	X	X							
Downstream		X				X					X	X	X	X					X	X	X

sediment continuity principle to provide an introduction to alluvial channel response to natural and human-induced change.

River morphology and river response is discussed in detail in Chapter 5 of FHWA's HDS No. 6, River Engineering for Highway Encroachments.

(2) *Stream Processes.* Prior to the current interest in ecology, water quality, and the environment, few engineers involved with highway crossings and encroachments considered the short-term and long-term changes that were possible or the many problems that humans can cause to streams. It is imperative that anyone working with rivers, either on localized areas or entire systems, have an understanding of the many factors involved, and of the potential for change within the river system. Highway construction can have significant general and local affects on the geomorphology and hydraulics of river systems. Hence, it is necessary to consider induced short-term and long-term effects of erosion and sedimentation on the surrounding landscape and the river. The biological response of the river system should also be considered and evaluated. Certain species of fish can only tolerate large quantities of suspended sediment for relatively short periods of time. This is particularly true of the eggs and fry. It is useful for the project engineer to understand what is important for regulators. Some of the most common topics include:

- Site geomorphology and steam stability
- Stressors to historic aquatic organism habitat
- Locations of hydraulic constrictions

Only with such knowledge can the project engineer develop the necessary arguments to make the case that erosion control measures must be designed to avoid significant deterioration of the stream environment not only in the immediate vicinity of the highway encroachment or crossing, but in many instances for great distances downstream.

Fluvial geomorphology is the science dealing with the shape of stream channels and includes the study of physical processes within river systems, such as bank erosion, sediment transport, and bed material sorting.

This section is intended to give the engineer background, perspective, and respect of stream processes and their dynamics when designing and constructing bank protection for natural streams and to lay the groundwork for application of the concepts of open-channel flow, fluvial geomorphology, sediment transport, and river mechanics to the design, maintenance, and environmental challenges associated with highway crossings and encroachments. Encroachment is any occupancy of the river and floodplain for highway use. Encroachments usually present no issues during normal stages, but require special protection against floods. Classifying the regions requiring protection, the possible types of protection, the possible flow conditions, the possible channel shapes, and the various geometric conditions aids the engineer in selecting the design criteria for the conditions encountered.

(a) Types of Encroachment. In the vicinity of rivers, highways generally impose a degree of encroachment. In some instances, particularly in mountainous regions or in river gorges and canyons, river crossings can be accomplished with absolutely no encroachment on the river. The bridge and its approaches are located far above and beyond any possible flood stage. More commonly, the economics of crossings require substantial encroachment on the river and its floodplain, the cost of a single span over the entire floodplain tends to be prohibitive. The encroachment can be in the form of earth fill bridge approach embankments on the floodplain or into the main channel itself, reducing the required bridge length; or in the form of piers and abutments or culverts in the main channel of the river. Longitudinal encroachments may exist that are not connected with river crossings. Floodplains often appear to provide an attractive low cost alternative for highway location, even when the extra cost of flood protection is included. As a consequence, highways, including interchanges, often encroach on a floodplain over long distances. In some regions, such as mountainous regions, river valleys (or canyons) provide the only feasible route for highways. This is true in areas where a floodplain does not exist. In many locations the highway encroaches on the main channel itself and the channel is partly filled to allow room for the roadway. See Figure 872.4. In some instances, this encroachment becomes severe, particularly as older highways are upgraded and widened.

(b) Effects of River Development Works. These works may include water diversions to and from the river system, dams, cutoffs (channel straightening), levees, navigation works, and the mining of sand and gravel. It is essential to consider the probable long-term plans of all agencies and groups as they pertain to a river when dealing with the river in any way. For example, dams serve as traps for the sediment normally flowing through the river system. With sediment trapped in the reservoir, essentially clear water is released downstream of the dam site. This clear water has the capacity to transport more sediment than may be immediately available. Consequently the channel begins to supply this deficit with resulting degradation of the bed or banks. The degraded or widened main channel causes steeper gradients on tributary streams in the vicinity of the main channel. The result is degradation in the tributary streams. It is entirely possible, however, that the additional sediments supplied by the tributary streams would ultimately offset the degradation in the main channel. Thus, it must be recognized that downstream of storage structures the channel may either aggrade or degrade (most common) and the tributaries will be affected in either case.

(c) Alluvial Streams. Most streams that highways cross or encroach upon are alluvial; that is, the streams are formed in materials that have been and can be transported by the stream. In alluvial stream systems, it is the rule rather than the exception that banks will erode; sediments will be deposited; and floodplains, islands, and side channels will undergo modification with time. Alluvial channels continually change position and shape as a consequence of hydraulic forces exerted on the bed and banks. These changes may be gradual or rapid and may be the result of natural causes or human activities. At any location in a stream, the cross-sectional shape is dependent upon the volume flow-rate (flow), the composition of sediment transported through a section, and the integrity or gradation of the bed and bank materials. As water flows through the stream channel, it exerts a fluid shear stress on the bed and banks. For a constant and stable cross-sectional shape for a given flow at a specific location, the resisting bed and bank material shear stress must be equal to the fluid stress at every point in the stream cross section perimeter. In this state, a stream is in the threshold condition where each point along the perimeter is at the threshold of movement or incipient motion. This condition also indicates a dynamic equilibrium with scour and deposition of sediment being equal. As flow, velocity, and fluid shear stress increase, the amount of scour and sediment deposition will change, which will also change the stream cross section for a given bed/bank gradation.

Alluvial streams are commonly trapezoidal in cross section through their straight reaches and become asymmetric through their bends. When streams incise in response to possible instability, their depth increases and the stream takes on a more rectangular cross-sectional shape. Also, streams with very large flows may become rectangular as the bed width increases to convey the large flows, especially if bedrock outcroppings are present on the banks preventing them from flattening.

(d) Non-Alluvial Streams. Some streams are not alluvial. The bed and bank material is very coarse, and except at extreme flood events, do not erode. These streams are classified as sediment supply deficient, i.e., the transport capacity of the streamflow is greater than the availability of bed material for transport. The bed and bank material of these streams may consist of cobbles, boulders, or bedrock. In general these streams are stable, but should be carefully analyzed for stability at large flows. A study of the plan and profile of a stream is useful in understanding stream morphology. Plan view appearances of streams are varied and result from many interacting variables. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a highway crossing or encroachment. This is particularly true for alluvial streams. Conversely, a highway crossing or encroachment can inadvertently change a variable, adversely affecting the stream.

(e) Dynamics of Natural Streams. Long-term climatic and tectonic fluctuations have caused major changes of river morphology, but rivers can display a remarkable propensity for change of position and morphology in time periods of a century. For shorter time periods river channels will shift through erosion and deposition at bends and may form chutes, islands or oxbow lakes. Lateral migration, erosion and deposition rates are not linear; i.e., a river may maintain a stable position for several years and then experience rapid movement. At low flow the bed of a sand bed stream can be dunes, but at large flows the bed may become plane or have antidune flow. With dunes, resistance to flow is large and bed material transport is low. Whereas, with plane bed or antidune flow the resistance to flow is small and the bed material transport is large. Much, therefore, depends on flood events, bank stability, permanence of vegetation on banks and the floodplain and watershed land use.

In summary, archaeological, botanical, geological, and geomorphic evidence supports the conclusion that most rivers are subject to constant change as a normal part of their morphologic evolution. Therefore, stable or static channels are the exception in nature.

If an engineer modifies a river channel locally, this local change may cause unintended modification of channel characteristics both up and down the stream. The response of a river to human-induced changes often occurs in spite of attempts by engineers to keep the anticipated response under control. The points that should be stressed are that a river through time is dynamic and that human-induced change frequently sets in motion a response that can be propagated upstream or downstream for long distances.

In spite of their complexity, all rivers are governed by the same basic forces. The design engineer must understand, and work with these natural forces:

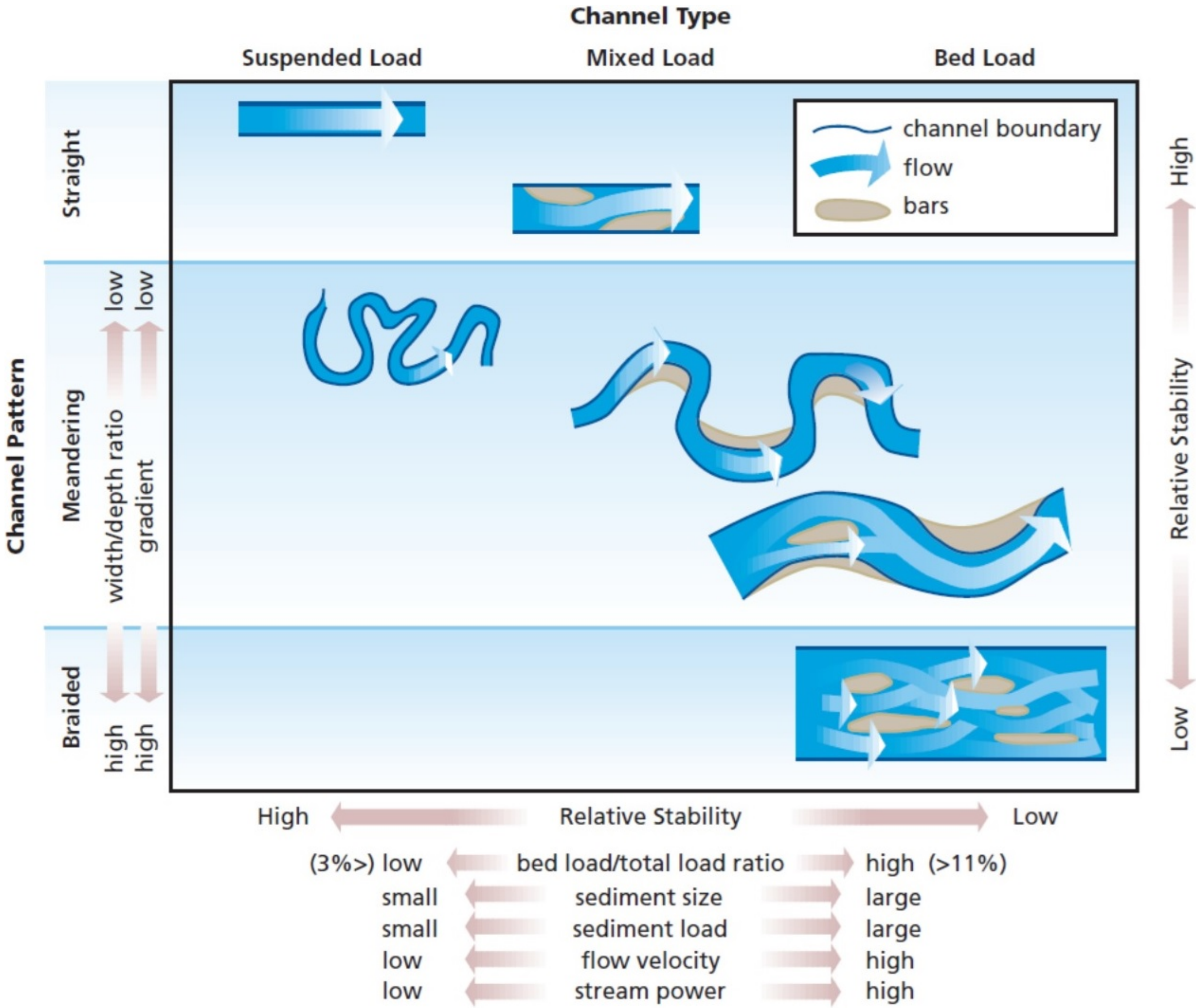
- Geological factors, including soil and seismic conditions.
- Hydrologic factors, including possible changes in flows, runoff, and the hydrologic effects of changes in land use.
- Geometric characteristics of the stream, including the probable geometric alterations that will be activated by the changes a project and future projects will impose on the channel.
- Hydraulic characteristics such as depths, slopes, and velocity of streams and what changes may be expected in these characteristics in space and time.
- Sea level rise may also cause river instability, particularly when the 75-year design life of a bridge is considered.

(f) Basic Stream Pattern. The three basic stream patterns are straight, braided, and meandering as seen in aerial or plan view. Pattern is one way of classifying a stream and generalizing its behavior, another is sediment load. See Figure 872.1.

Commonly, stream patterns are identified by sinuosity, which is defined as channel length divided by valley (floodplain) length. For straight and braided streams, sinuosity varies between 1.0 and 1.5, while meandering streams have sinuosity greater than 1.5. These different patterns and their associated gradients contribute to changes and adjustments in streams, and specifically influence flow resistance that effects sediment transport and formation of cross-sectional shape. Engineers using any stream

Figure 872.1

Stream Classification



classifications should be aware that they are artificial constructs, and no strict science laws or principles of classification (such as used in biology) are possible. Although we may assign channel reaches to discrete categories based on arbitrary thresholds of slope, sinuosity, bed material size, sediment load, width-depth ratio, etc., these quantities vary continuously, and channels tend to behave in rather individualistic fashion. Different types of streams occur within a given subregion. Index 3.9 of Caltrans Hydromodification Requirements Guidance presents the various stream forms within each of the physiographic subregions of California available at the following website:

http://www.dot.ca.gov/hq/oppd/stormwtr/guidance/CT-Hydromodification-Requirements-Guidance.pdf

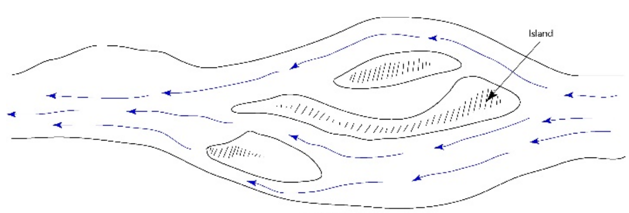
(g) Straight Streams. Straight river channels can be of two types. The first forms on a low-gradient valley slope, has a low width-depth ratio channel, and is relatively stable. The first type of straight channel may contain alternate islands or bars that result in a sinuous thalweg (flow path connecting deepest points in successive cross sections) within the straight channel. It may seem that the first type of straight stream is very stable because of low slope and energy, but alternating sediment deposits can cause lateral instability. In general, it is more natural for a stream to meander than to have a straight stream pattern, therefore it is difficult to find low-gradient straight streams in the field, especially long reaches.

The second type is a steep gradient, high width-depth ratio, high energy river that has many islands or bars, and at low flow is braided. It is relatively active.

In general, the designer should not attempt to develop straight channels fully protected with riprap. In a straight channel the alternate islands or bars and the thalweg are continually changing; thus, the current is not uniformly distributed through the cross-section but is deflected toward one bank and then the other.

(h) Braided Streams. Similar to straight streams, streams with braided pattern have low sinuosity, but have the highest gradient of any of the stream patterns. Braided streams have many sub-channels within the main stream channel that interweave and crisscross. The sub-channels are separated by islands or bars which are visible during low flows and normally submerged under high flows. Because braided streams have steep slopes, they possess the higher energy necessary to erode and transport sediment that comprises the bars and islands. Even though braided streams have high energy, these streams will deposit their coarser and larger material that cannot be physically transported by the stream's average velocity and shear stress. In other words, the process of braiding occurs during flood events as a stream adjusts in response to the larger sediment and debris loads that cannot be sustained while trying to find dynamic equilibrium. This deposition of larger material creates the bars and islands. See Figure 872.2. As flow and velocity fluctuate during a flood event, it is common to see movement and re-creation of bars, islands, and sub-channels.

Figure 872.2
Diagram of a Braided River Channel



(i) Meandering Streams. Meandering is the most common stream pattern, having a series of alternating curves or bends, and is associated with flatter valleys. Meandering stream types have the highest sinuosity because of their longer stream length, due to several alternating curves, with respect to valley length, see Figure 872.6. One way that streams seek dynamic equilibrium is to dissipate energy through erosion of their banks, creating meandering patterns. When meanders are created, overall stream length is increased, and energy is released through the work necessary to scour its banks, which brings a stream closer to dynamic equilibrium. Streambank revetments are often constructed through these meanders to prevent excessive erosion that may cause instability of nearby or adjacent transportation facilities.

Once curves have been created in a stream's alignment, velocity increases as the flow of water moves through the outside bank of a bend caused by secondary circulation currents. Given the geometry of a curve, velocity is resolved into three components described in the longitudinal, width-wise, and vertical directions, contrary to straight reaches of stream.

As flow moves through a curve, the circulation currents and their turbulence are influenced by radius of curvature, stream bottom width, flow depth, curve deflection angle, and Reynolds Number. As often occurs, turbulence is magnified by counter-circulating currents from an upstream bend merging with circulating currents of an immediate downstream bend. The increased turbulence usually increases the amount of scour at the outside bend, and the transported material is deposited on the inside bend at the downstream reversing curve creating a point island or bar.

Another characteristic of flow through a curve is that the top of the water surface will superelevate along the outside bank of a curve as it is pulled by centrifugal forces while the bottom water surface at the bed is being pulled toward the inside of a bend. These two actions will cause skewing of the circulating current contributing to increased erosion around a bend.

(j) Sediment Transport. For engineering purposes, the two sources of sediment transported by a stream are: (1) bed material that makes up the stream bed; and (2) fine material that comes from the banks and the watershed (washload). Geologically both materials come from the watershed, but for the engineer, the distinction is important because the bed material is transported at the capacity of the stream and is functionally related to measurable hydraulic variables. The washload is not transported at the capacity of the stream. Instead, the washload depends on availability and is not functionally related to measurable hydraulic variables.

The division size between washload and bed sediment load is sediment size finer than the smallest 10 percent of the bed material. It is important to note that in a fast flowing mountain stream with a bed of cobbles the washload may consist of coarse sand sizes. For these conditions, the transport of sand sizes is supply limited. In contrast, if the bed of a channel is silt, the rate of bed load transport of the silt sizes is less a question of supply than of capacity.

When a river reaches equilibrium, its transport capacities for water and sediment are in balance with the rates supplied. In fact, most rivers are subject to some kind of control or disturbance, natural or human-induced that gives rise to non-equilibrium conditions. HDS No. 6, Index 4.3.2, states total sediment load can be expressed by three equations:

1. By type of movement

$$L_T = L_b + L_s$$

2. By method of measurement

$$L_T = L_m + L_u$$

3. By source of sediment

$$L_T = L_w + L_{bm}$$

Where:

L_T = Total load;

L_b = Bed load which is defined as the transport of sediment particles that are close to or maintain contact with the bed;

L_s = Suspended load defined as the suspended sediment passing through a stream cross-section above the bed layer;

L_m = Measured sediment;

L_u = Unmeasured sediment that is the sum of bed load and a fraction of suspended load below the lowest sampling elevation;

L_w = Wash load which is the fine particles not found in the bed material ($D_s < D_{10}$), and originates from available bank and upstream supply;

L_{bm} = Capacity limited bed material load.

Streams are unique from other hydraulic conveyance facilities, such as engineered channels and pipes, in that its boundaries are mobile, and they move sediment within their water column or along the bed by skipping and rolling, which is a complicated interrelationship. The suspended sediment load is carried through the flow by turbulence and is typically fine sand, silt, and clay. Bedload is coarser possibly as large as boulders, and moves along the bed by fluid stress action, see Figure 872.3. Sediment supply and its movement are the life of a stream that can become unstable when this process is interrupted if supply becomes limited or if a stream is unable to transport its excess downstream.

Instability can be seen through channel incision, where the stream bed degrades and banks over steepen, excessive meandering, or large alignment shifts as a stream attempts to control energy as it searches for dynamic equilibrium. The ability of a stream to control and manage its sediment is not the only influence on stream stability, but one of the more important factors.

Within a stream bed, immersed sediment particles resting on the stream bed over other particles exert their effective weight in the form of a vertical force, which can be divided into normal and tangential components based on the stream bed slope. Simply stated, in order for sediment particles to become mobile, a force greater than their normal weight must be applied to them. This force that causes mobility is a drag force or fluid stress acting on the particle as water flows over them. The fluid stress can be expressed as an average boundary shear stress acting on a stream bed considering steady, uniform flow:

$$\sigma_0 = gDS_f$$

Where:

σ_0 = Shear stress = Force per unit area in flow direction;

g = Specific weight of water;

D = Flow-depth;

S_f = Friction slope.

Particle movement can be further expressed at a specific point in a stream bed as incipient motion, which is the initial movement of a particle. The calculation of a critical shear stress or critical velocity can be performed at the threshold movement condition that assumes active hydraulic forces are equal to particle resistant forces. At the point of critical shear stress or critical velocity, a particle is just about to move. This means that values of shear stress or velocity greater than critical shear stress or critical velocity cause particles to be in motion, while particles will be at rest with values of shear stress and velocity lower than critical shear stress and velocity. An incipient motion calculation can provide an indication of erosion potential and stream stability. Fischenich (2001) provides a variation of the widely accepted and industry standard Shields equation for approximated critical shear stress considering different materials:

Clays: $\sigma_{cr} = 0.5d(g_s - g_w)\tan F$

Silts & Sands:
$$\sigma_{cr} = 0.25d_0 - 0.6d(g_s - g_w)\tan F$$

Gravels & Cobbles:
$$\sigma_{cr} = 0.06d(g_s - g_w)\tan F$$

Where:

$d_0 = d[(G-1)g\nu^{-2}]^{1/3}$;

σ_{cr} = Critical shear stress;

F = Soil grain angle of repose;

Figure 872.3

Bed Load and Suspended Load

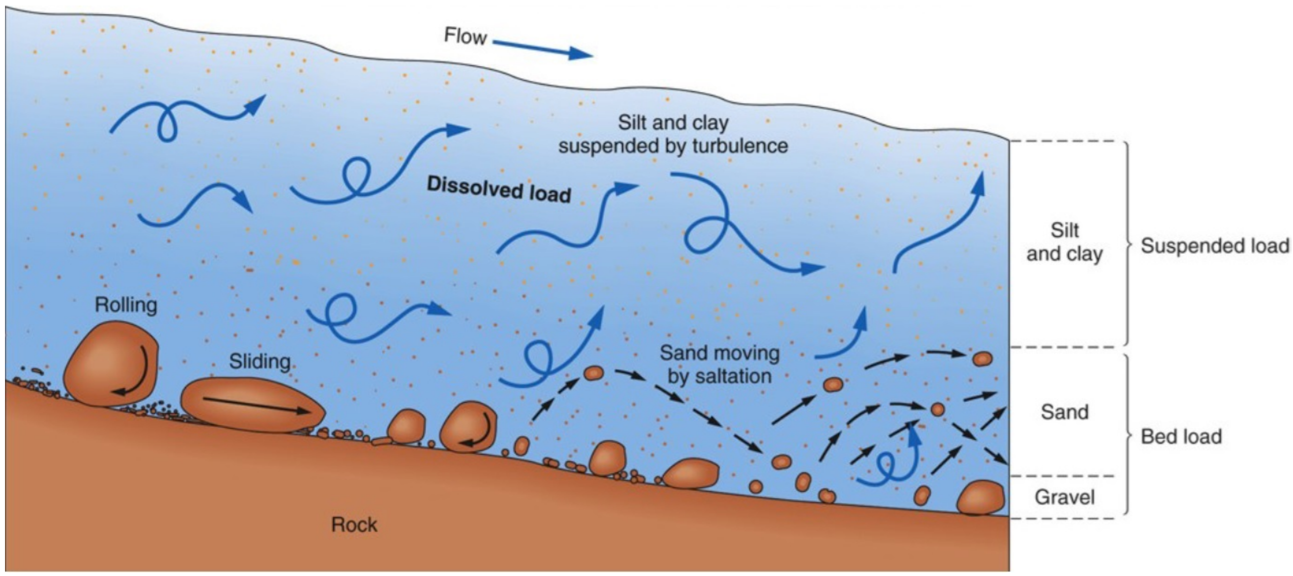


Figure 872.4
Longitudinal Encroachments

Highway 49, North Fork Yuba River (Near Downieville) and Highway 190, Furnace Creek, (Death Valley)

d = Soil diameter;

g_s = Sediment unit weight;

g_w = Water unit weight;

G = Sediment specific gravity;

g = Gravity;

ν = Water/sediment mixture kinematic viscosity.

The Shields equation and the beginning of motion is described in more detail in Index 3.5 of HDS No. 6.

Modeling of a stream reach, although complex, can be performed in order to predict sediment transport potential on a larger scale, transport rates, volume, and capacity modeling. Several empirical sediment transport functions used in modeling have been developed and named after their creators, such as Einstein, Acker and White, Laursen-Copeland, Meyer-Peter Muller, and Yang. These functions are complex and notoriously data intensive. Three classic sediment transport formulae are discussed in detail in Index 4.5 of HDS No. 6 to illustrate sediment transport processes. While not often, resource agencies and flood control districts may request this type of analysis during the permit review process. If sediment modeling is necessary, HEC-RAS v4.1 (or higher), the Army Corps of Engineers' river and stream modeling software, contains sediment transport modeling capabilities using these transport functions and others.

(k) Stream Channel Form. Major factors affecting alluvial stream channel forms are:

- stream discharge, viscosity, temperature;
- sediment discharge;
- longitudinal slope;
- bank and bed resistance to flow;
- vegetation;
- geology, including types of sediments and;
- human activity.

At any location in a stream, the cross-sectional shape is dependent upon the volume flow-rate (flow), the composition of sediment transported through a section, and the integrity or gradation of the bed and bank materials. As water flows through the stream channel, it exerts a fluid shear stress on the bed and banks. For a constant and stable cross-sectional shape for a given flow at a specific location, the resisting bed and bank material shear stress must be equal to the fluid stress at every point in the stream cross section perimeter. In this state, a stream is in the threshold condition where each point along the perimeter is at the threshold of movement or incipient motion. This condition also indicates a dynamic equilibrium with scour and deposition of sediment being equal. As flow, velocity, and fluid shear stress increase, the amount of scour and sediment deposition will change, which will also change the stream cross section for a given bed/bank gradation.

The form and appearance of a stream can also be influenced by features within the stream profile, such as riffles and pools because of their affects on the acting fluid shear stress and velocity. Riffles are longitudinal sections of streams with higher velocity, where lower flow-depth usually caused by obstructions, such as gravels, cobbles, and boulders created by island or bar development. On the contrary, pools have higher flow-depth and lower velocity, and are typically comprised of finer silts and sands compared to a riffle. These bed materials associated with pools and riffles have an affect on resisting bed shear stress that will influence stream shape and stability. The alternating pool and riffle sequence is common for nearly all perennial streams that have gravel to boulder size bed formations. Different types of streams occur within a given subregion. Index 3.9 of Caltrans Hydromodification Requirements Guidance presents the various stream forms within each of the physiographic subregions of California, see:

http://www.dot.ca.gov/hq/oppd/stormwtr/guidance/CT-Hydromodification-Requirements-Guidance.pdf

(l) Floodplain Form. From a geomorphic perspective, floodplains are flatter lands adjacent to a river main channel that are dry until larger flows force water out of the stream channel into these overbank lands during significant flood events. Floodplains typically include the following features: the main stream channel itself, point islands or bars, oxbows and lakes, natural raised berms (levees) above floodplain surface, terraces, sloughs and depressions, overbank fine and coarse sediment deposition, scattered debris, and vegetation.

When water exceeds the capacity of the main channel, the conveyance of flow through the floodplain overbanks will differ from the main channel due to uniqueness of form (shape), gradient, alignment, and likely the flow resistance (roughness) of the floodplain versus the stream channel. Therefore, water will move and deposit varying sediment types differently, also at different frequency, creating a separate floodplain form. Once sediment is moved to the floodplain, coarser sediment is generally deposited along the streambanks forming levees, while finer sediment is dropped between the valley walls and the levees on the floodplain floor. Sediment is stored and becomes dormant until larger flows return to the floodplain that may convey the sediment down-valley.

Similar to the stream channel, floodplain form is directly linked to the sediment transport process, as well as floodplain stability affected by sediment supply and its movement. Fluid shear stress and velocity control the sediment/debris degradation and deposition properties within the floodplain that impact its form, landscape, and appearance. Because the floodplain can be dormant for considerable time depending on watershed hydrology, its form can remain relatively constant and preserved for extended periods, as well as be less dynamic than the stream channel.

(m) Streambank Erosion. Simply defined, streambank erosion occurs when the soil resisting strength is less than the driving forces acting on the bank. It can occur through bank-toe scour below the water line and bank mass failure from above. This erosion occurs first as a geotechnical failure followed by the hydraulic action that removes the failed soil and sediment by fluid shear stress. The hydraulic action further causes lateral scour of the bank and is the principal contributor to bank-toe failure. This is a natural process for both stable and unstable streams, but is exaggerated in the latter case. The degree of erosion can be influenced by impervious development in the watershed, agricultural use, and changes in climate. With or without these influences and whether a stream is stable or unstable, streambank erosion will take place at some level. Therefore, scour must be reduced at critical locations to protect highway structures and preserve public safety, although restraint needs to be exercised during the project development process so that a stream does not greatly change its morphology in response to the protection measures.

The driving and resisting forces for streambank erosion, mentioned above, are controlled by a series of factors. The factors that influence the calculation of the driving (active) forces within geotechnical failure are soil saturated unit weight, pore pressure, bank height, and angle of repose, as well as object surcharges within and above the bank such as vegetation. The effects of driving forces are commonly seen through soil saturation as a result of intense precipitation with subsequent increase in pore pressure and bank soil saturated unit weight that can cause mass bank failure. The forces that will resist and give soil its strength from geotechnical type failure are dependent upon effective soil cohesion, normal stress, pore pressure, and soil effective angle of friction.

During streambank erosion, the bank soil can fail by different modes. Generally speaking, steep slopes present slab-type or toppling failures where large slabs (blocks of soil) of the bank break away from the top and fall into the stream, while mild slopes show a rotational failure that begins at the bank toe causing soil to slide from above into the stream. Once the eroded soil reaches flowing water, it is usually transported downstream depending upon its size and composition.

As for bank-toe scour, its main influences are derived from bank soil composition and gradation, volume of sediment in transport, stream flow and stream gradient. These factors and the principles of scour and sediment movement from hydraulic forces are a reoccurring theme in fluvial geomorphology. The following paragraphs summarize the characteristics of unstable and stable banks;

(1) Unstable banks with moderate to high erosion rate occur when the slope angle of unstable banks typically exceed 30 percent, where a cover of woody vegetation is rarely present. At a bend, the point island or bar opposite of an unstable cut bank is likely to be bare at normal stage, but it may be covered with annual vegetation and low woody vegetation, particularly willows. Where very rapid erosion is occurring, the bankline may have irregular indentations. Fissures, which represent the boundaries of actual or potential slump blocks along the bankline indicate the potential for rapid bank erosion.

(2) Unstable banks with slow to moderate erosion rate occur when a bank is partly graded (smooth slope) and the degree of instability is difficult to assess where reliance is placed mainly on vegetation. The grading of a bank typically begins with the accumulation of slumped material at the base such that a slope is formed, and progresses by smoothing of the slope and the establishment of vegetation.

(3) Stable banks with very slow erosion rate occur where banks tend to be graded to a smooth slope and the slope angle is usually less than about 30 percent. In most regions, the upper parts of stable banks are vegetated, but the lower part may be bare at normal stage, depending on bank height and flow regime of the stream. Where banks are low, dense vegetation may extend to the water's edge at normal stage. Mature trees on graded bank slopes are particularly convincing evidence for bank stability. Where banks are high, occasional slumps may occur on even the most stable graded banks. Shallow mountain streams that transport coarse bed sediment tend to have stable banks.

For a more detailed discussion of bank stability and the mechanics of bank failure see HEC 20.

(n) Young Valley. Typically young valleys are narrow V-shaped valleys with streams on steep gradients. Relief elevation greater than

1,000 ft is regarded as mountainous, while relief in the elevation range of 100 to 1,000 ft is regarded as hilly. Streams in mountainous regions are likely to have steep slopes, coarse bed materials (gravel or cobble-boulder), narrow floodplains, and have nonalluvial characteristics (i.e., supply-limited sediment transport rates). At flood stage, the stream flow covers all or most of the valley floor. The usual situation for such locations is a structure crossing a well-defined channel in which the design discharge will flow at a moderate to high velocity.

(1) Cross-Channel Location. A cross channel location is a highway crossing a stream on normal or skewed alignment. The erosive forces of parallel flow associated with a normal crossing are generally less of a threat than the impinging and eddy flows associated with a skewed crossing. The effect of constriction by projection of the roadway embankment into the channel should be assessed.

Characteristics to be considered include:

- Stream velocity.
- Scouring action of stream.
- Bank stability.
- Channel constrictions (artificial or natural).
- Nature of flow (tangential or curvilinear).
- Areas of impingement at various stages.
- Security of leading and trailing edges.

Common protection failures occur from:

- Undermining of the toe (inadequate depth/size of foundation), see Figure 872.5 and Table 872.2.
- Local erosion due to eddy currents.
- Inadequate upstream and downstream terminals or transitions to erosion-resistant banks or outcrops.
- Structural inadequacy at points of impingement overtopping.
- Inadequate rock size, see Table 872.2.
- Lack of proper gradation/ layering/ RSP fabric, leading to loss of embankment, see Table 872.2.

Any of the more substantial armor treatments can function properly in such exposures providing precautions are taken to alleviate the probable causes of failure. If the foundation is questionable for concreted-rock or other rigid types it would not be necessary to reject them from consideration but only to provide a more acceptable treatment of the foundation, such as heavy rock or sheet piling.

Whether the highway crosses a stream channel on a bridge or over a culvert, economic considerations often lead to constriction of the waterway. The most common constriction is in width, to shorten the structure. Next in frequency is obstruction by piers and bents of bridges or partitions of multiple culverts.

Figure 872.5

Slope Failure Due to Loss of Toe

Table 872.2

Failure Modes and Effects Analysis for Riprap Revetment

Failure Modes	Effects on Other Components	Effects on Whole System	Detection Methods	Compensating Provisions
Translational slope or slump (slope failure)	Disruption of armor layer	Catastrophic failure	• Mound of rock at bank toe • Unprotected upper bank	• Reduce bank slope • Use more angular or smaller rock • Use granular filter rather than geotextile fabric
Particle erosion (rock undersized)	Loss of armor layer, erosion of filter	Progressive failure	• Rock moved downstream from original location • Exposure of filter	• Increase rock size • Modify rock gradation
Piping or erosion beneath armor (improper filter)	Displacement of armor layer	Progressive failure	• Scalloping of upper bank • Bank cutting • Void beneath and between rocks	• Use appropriate granular or geotextile filter
Loss of toe or key (under designed)	Displacement or disruption or armor layer	Catastrophic failure	• Slumping of rock • Unprotected upper bank	• Increase size, thickness, depth or extent of toe or key

The risk of constricting the width of the waterway is closely related to the relative conveyance of the natural waterway obstructed, the channel scour, and to the channel migration. Constricting the width of flow at structures has the following effects:

- Increase in the upstream water surface elevation (backwater profile).
- Increase in flow velocity through the structure opening (waterway).
- Causes eddy currents around the upstream and downstream ends of the structure.

Unless protection is provided the eddy currents can erode the approach roadway embankment and the accelerated flow can cause scour at bridge abutments. The effects of erosion can be reduced by providing transitions from natural to constricted and back to natural sections, either by relatively short wingwalls or by relatively long training embankments or structures.

Channel changes, if properly designed, can improve conditions of a crossing by reducing skew and curvature and enlarging the main channel. Unfortunately there are "side effects" which actually increase erosion potential. Velocity is almost always increased by the channel change, both by a reduction of channel roughness and increase of slope due to channel shortening. In addition, channel changes affecting stream gradient may have upstream and/or downstream effects as the stream adjusts in relation to its sediment load.

At crossing locations, lateral erosion can be controlled by positive protection, such as armor on the banks, rock spurs to deflect currents away from the banks, retards to reduce riparian velocity, or vertical walls or bulkheads. The life cycle cost of such devices should be considered in the economic studies to choose a bridge length which minimizes total cost.

Accurate estimates of anticipated scour depths are a prerequisite for safe, cost effective designs. Design criteria require that bridge foundations be placed below anticipated scour depths. For this reason the design of protection to control scour at such locations is seldom necessary for new construction. However, if scour may undercut the toes of dikes or embankments positive methods including self-adjusting armor at the toe, jetties or retards to divert scouring currents away from the toe, or sill-shaped baffles interrupting transport of bedloads should be considered.

There is the potential for instability from saturated or inundated embankments at crossings with embankments projecting into the channel. Failures are usually reported as "washouts", but several distinct processes should be noted:

- Saturation of an embankment reduces its angle of repose. Granular fills with high permeability may "dissolve" steadily or slough progressively. Cohesive fills are less permeable, but failures have occurred during falling stages.
- As eddies carve scallops in the embankment, saturation can be accelerated and complete failure may be rapid. Partial or total losses can occur due to an upstream eddy, a downstream eddy, or both eddies eroding toward a central conjunction. Training devices or armor can be employed to prevent damage.
- If the fill is pervious and the pavement overtopped, the buoyant pressure under the slab will exceed the weight of slab and shallow

overflow by the pressure head of the hydraulic drop at the shoulder line. A flat slab of thickness, t, will float when the upstream stage is 4t higher than the top of the slab. Thereafter the saturated fill usually fails rapidly by a combination of erosion and sloughing. This problem can occur or be increased when curbs, dikes, or emergency sandbags maintain a differential stage at the embankment shoulder. It is increased by an impervious or less pervious mass within the fill. Control of flotation, insofar as bank protection is concerned, should be obtained by using impervious armor on the upstream face of the embankment and a pervious armor on the downstream face.

Culvert problem locations generally occur in and along the downstream transition. Sharp divergence of the high velocity flow develops outward components of velocity which attack the banks directly by impingement and indirectly by eddies entrained in quieter water. Downward components and the high velocity near the bed cause the scour at the end of the apron.

Standard plans of warped wingwalls have been developed for a smooth transition from the culvert to a trapezoidal channel section. A rough revetment extension to the concrete wingwalls is often necessary to reduce high velocity to approximate natural flow. Energy dissipaters may be used to shorten the deceleration process when such a transition would be too long to be economical. Bank protection at the end of wingwalls is more cost effective in most cases.

(2) Parallel Location. With parallel locations the risk of erosion damage along young streams increases where valleys narrow and gradients steepen. The risk of erosion damage is greatest along the outer bend of natural meanders or where highway embankment encroaches on the main channel.

The *encroaching* parallel location is very common, especially for highways following mountain streams in narrow young valleys or canyons. Much of the roadway is supported on top of the bank or a berm and the outer embankment encroaches on the channel in a zone of low to moderate velocity. Channel banks are generally stable and protection, except at points of impingement, is seldom necessary.

The *constricting* parallel location is an extreme case of encroaching location, causing such impairment of channel that acceleration of the stream through the constriction increases its attack on the highway embankment requiring extra protection, or additional waterway must be provided by deepening or widening along the far bank of the stream.

In young valleys, streams are capable of high velocity flows during flood stages that may be damaging to adjacent highway facilities. Locating the highway to higher ground or solid support is always the preferred alternative when practical.

Characteristics to be considered include:

- High velocity flow.
- Narrow confined channels.
- Accentuated impingement.
- Swift overflow.
- Disturbed flow due to rock outcrops on the banks or within the main channel.
- Alterations in flow patterns due to the entrance of side streams into the main channel.

Protective methods that have proven effective are:

- Rock slope protection.
- Concreted-rock slope protection.
- Walls of masonry and concrete.
- Articulated concrete block revetments.
- Sacked concrete.
- Cribs walls of various materials.

(o) Mature Valley. Typically mature valleys are broad V-shaped valleys with associated floodplains. See Figure 872.6. The gradient and velocity of the stream are low to moderate. Streams in regions of lower relief are usually alluvial and exhibit more problems because of lateral erosion in the channels. Vegetative cover, land use, and flow depth on the floodplain are also significant factors in stream channel stability. Changes in channel geometry with time are particularly significant during periods when alluvial channels are subjected to high flows, and few changes occur during relatively dry periods. Erosive forces during high-flow periods may have a capacity as much as 100 times greater than those forces acting during periods of intermediate and low-flow rates.

Figure 872.6
Mature Valley with Meandering Stream

Russian River near Geyserville

When considering the stability of alluvial streams, in most instances it can be shown that approximately 90 percent of all changes occur during that small percentage of the time when the flow equals or exceeds dominant discharge. A discussion of dominant discharge may be found in Hydraulic Design Series No. 6, but the bankfull flow condition is recommended for use where a detailed analysis of dominant discharge is not feasible. In addition to the general information previously given, the following applies to mature valleys:

(1) Cross-Channel Location. The usual situation is a structure crossing a braided or meandering normal flow channel. The marginal area subject to overflow is usually traversed by the highway on a raised embankment and may have long approaches extending from both banks.

Characteristics to be considered include:

- Shifting of the main channel.
- Skew of the stream to the structure.
- Foundation in deep alluvium.
- Erodible embankment materials.
- Channel constrictions, either artificial or natural, which may affect or control the future course of the stream.
- Variable flow characteristics at various stages.
- Stream acceleration at the structure.

Armor protection has proven effective to prevent erosion of road approach embankments, supplemented if necessary by stream training devices such as guide dikes, permeable retards or jetties to direct the stream through the structure. The abutments should not depend on the training dikes to protect them from erosion and scour. At bridge ends one of the more substantial armor types may be required, but bridge approach embankments affected only by overflow seldom require more than a light revetment, such as a thin layer of rocky material, vegetation, or a fencing along the toe of slope. For channel flow control upstream, the size and type of training system ranges from pile wings for high velocity, through permeable jetties for moderate velocity, to the earth dike suitable for low velocity.

The more common failures in this situation occur from:

- Lack of upstream control of channel alignment.
- Damage of unprotected embankments by overflow and return flow.
- Undercut foundations.
- Formation of eddies at abrupt changes in channel.
- Stranding of drift in the converging channel.

(2) Parallel Location. Parallel highways along mature rivers are often situated on or behind levees built, protected and maintained by other agencies. Along other streams, rather extensive protective measures may be required to control the action of these meandering streams.

Channel change is an important factor in locations parallel to mature streams. The channel change may be to close an embayment, to cut off an oxbow, or to shift the alignment of a long reach of a stream. In any case, positive means must be adopted to prevent the return of the stream to its natural course. For a straight channel, the upstream end is critical, usually requiring bank protection equivalent to the facing of a dam. On a curved channel change, all of the outer bend may be critical, requiring continuous protection. Continuous and resistive bank protection measures, such as riprap and longitudinal rock toes are primarily used to armor outer bends or areas with impinging flows. These continuous and concentrated high velocity areas will generally result in reduced aquatic habitat. Since streambank protection designs that consist of riprap, concrete, or other inert structures alone may be unacceptable for lack of environmental and aesthetic benefits. Resource agencies have increased interest in designs that combine vegetation and inert materials into living systems that can reduce erosion while providing environmental and aesthetic benefits.

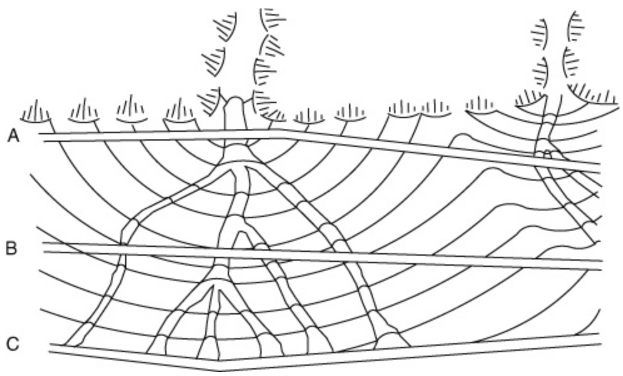
(3) *Desert Wash Locations*. Particular consideration should be given to highway locations that traverse natural geographical features of desert washes, sand dunes, and other similar regions.

Desert washes are a prominent feature of the physiography of California. Many long stretches of highway are located across a succession of outwash cones. Infrequent discharge is typically wide and shallow, transporting large volumes of solids, both mineral and organic. Rather than bridge the natural channels, the generally accepted technique is to concentrate the flow by a series of guide dikes leading like a funnel to a relatively short crossing.

An important consideration at these locations is instability of the channel, see Figure 872.7. For a location at the top of a cone (Line A), discharge is maximum, but the single channel emerging from the uplands is usually stable. For a location at the bottom of the cone (Line C), instability is maximum with poor definition of the channel, but discharge is reduced by infiltration and stream dispersion. The energy of the stream is usually dissipated so that any protection required is minimal. The least desirable location is midway between top and bottom (Line B), where large discharge may approach the highway in any of several old channels or break out on a new line. Control may require dikes continuously from the top of the cone to such a mid-cone site with slope protection added near the highway where the converging flow is accelerated. See Figure 872.8, which depicts a typical alluvial fan.

Also common are roadway alignments which longitudinally encroach, or are fully within the desert wash floodplain, see Figure 872.9. Re-alignment to a stable location should be the first consideration, but restrictions imposed by federal or state agencies (National Park Service, USDA Forest Service, etc.) may preclude that option, somewhat similar to transverse crossings. The designer may need to consider allowing frequent overtopping and increased sediment removal maintenance since an "all weather design" within these regimes can often lead to large scale roadway washout.

Figure 872.7 Alternative Highway Locations Across Debris Cone



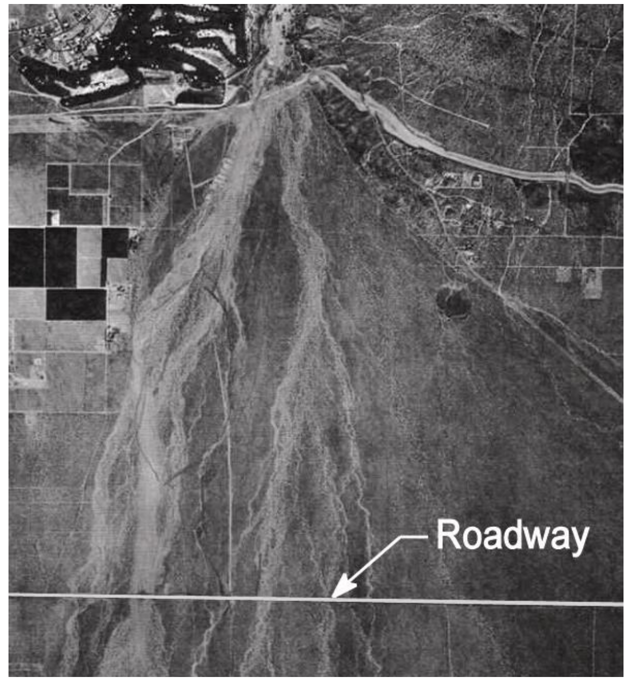
A. Cross at a single definite channel
B. A series of unstable indefinite channels and
C. A widely dispersed and diminished flow

Characteristics to be considered include:

- The intensity of rainfall and subsequent run-off.
- The relatively large volumes of solids that are carried in such run-off.
- The lack of definition and permanence of the channel.
- The scour depths that can be anticipated.
- The lack of good foundation.

Effective protective methods include armor along the highway and at structures and the probable need for baffles to control the direction and velocity of flow. Installations of rock, fence, palisades, slope paving, and dikes have been successful.

Figure 872.8 Alluvial Fan



Typical multi-channel stream threads on alluvial fan. Note location of roadway crossing unstable channels.

Figure 872.9 Desert Wash Longitudinal Encroachment

Road washout due to longitudinal location in desert wash channel

The Federal Emergency Management Agency (FEMA) Flood Hazard Mapping website contains information on recognizing alluvial fan landforms and methods for defining active and inactive areas. See their "Guidelines for Determining Flood Hazards on Alluvial Fans" at http://www.fema.gov/fhm/ft_tocs.shtm.

(4) *Construction, Easements, Access and Staging.* A primary site consideration for any bank protection design is its constructability. This may include the need for supplemental plans and temporary construction easements for stage construction to accommodate equipment access. See Figure 872.10.

Figure 872.10
Stage Construction

(5) *Biodiversity.* The riparian area provides one of the richest habitats for large numbers of fish and wildlife species, which depend on it for food and shelter. Many species, including coho and Chinook salmon, steelhead, yellow-billed cuckoo, and the red-legged frog, are threatened or endangered in California. Natural riparian habitat also includes the assortment of native plants that occur adjacent to streams, creeks and rivers. These plants are well adapted to the dynamic and complex environment of streamside zones. A key threat to fish species in any migrating corridor therefore will include loss of riparian habitat and instream cover affecting juvenile rearing and outmigration.

For channel and habitat characterization and preliminary assessment relative to designing and obtaining project specific permits, District biologist staff should be consulted early on within the project planning phase for subject matter expertise regarding fisheries, habitat, and wildlife. District biologist staff can also perform an initial stream habitat assessment. Contact information for Department biologists can be accessed through the CalBioRoster.

Numerous State and Federal agencies are responsible for fish management in California - including California Department of Fish and Wildlife, the National Marine Fisheries Service and the United Stated Army Corps of Engineers. Each agency has its own guidelines and jurisdiction. For example, detailed information on the requirements for fish habitat in riparian corridors may be found in Volume One and Two of the California Salmonid Stream Habitat Restoration Manual: http://www.dfg.ca.gov/fish/Resources/HabitatManual.asp

872.4 Data Needs

The types and amount of data needed for planning and analysis of channel protection varies from project to project depending upon the class and extent of the proposed protection, site location environment, and geographic area. See Index 872.1. The data that is collected and developed including preliminary calculations, and alternatives considered should be documented in project development reports (Environmental Document, Project Report, etc.) or as a minimum in the project file. These records serve to guide the detailed designs, and provide reference background for analysis of environmental impacts and other needs such as permit applications and historical documentation for any litigation which may arise. See Index 873.3(3)(a)(2)(b) for rock sizing equation parameters.

Recommendations for data needs can be requested from the District Hydraulics Engineer or determined from Chapter 8 of FHWA’s HDS No. 6, for a more complete discussion of data needs for highway crossings and encroachments on rivers. Further references to data needs are contained in Chapter 810, Hydrology and FHWA's HDS No. 2, Highway Hydrology and HEC 20, Stream Stability at Highway Structures.

872.5 Rapid Assessment

The National Pollutant Discharge Elimination System (NPDES) permit mandates a risk-based approach to be employed during planning and design for assessing stream stability at highway crossings. This approach involves conducting a rapid pre-project assessment of the vertical and lateral stability of the receiving stream channel related to an existing or planned highway crossing structure. If the rapid stability assessment (RSA) indicates potential problems, more detailed engineering analyses are required to determine if countermeasures are needed to stabilize the crossing to prevent the release of sediment. Therefore, if available, stream stability assessments for nearby highway crossings should be included in the site consideration for channel protection.

Section 3 of Caltrans Hydromodification Requirements Guidance Storm Water Best Management Practices Rapid Assessment of Stream Crossings Higher Level Stream Stability Analysis is an excellent resource for understanding the concepts of basic geomorphology and California earth science.

Table 8 of Assessing Stream Channel Stability at Bridges in Physiographic Regions (FHWA-HRT-05-072) presents an extensive listing of factors affecting stream stability.

Topic 873 - Design Concepts

873.1 Introduction

No attempt will be made here to describe in detail all of the various devices that have been used to protect embankments against scour. Methods and devices not described may be used when justified by economic analysis. Not all publicized treatments are necessarily suited to existing conditions for a specific project.

A set of plans and specifications must be prepared to define and describe the protection that the design engineer has in mind. These plans should show controlling factors and an end product in such detail that there will be no dispute between the construction engineer and contractor. To serve the dual objectives of adequacy and economy, plans and specifications should be precise in defining materials to be incorporated in the work, and flexible in describing methods of construction or conformance of the end product to working lines and grades.

Recommendations on channel lining, slope protection, and erosion control materials can be requested from the District Hydraulic Engineer, the District Materials Branch and the Office of Highway Drainage and Water Quality Design in Headquarters. The District Landscape Architect will provide recommendations for temporary and permanent erosion and sediment control measures. The Caltrans Bank and Shore Protection Committee is available on request to provide advice on extraordinary situations or problems and to provide evaluation and formal approvals for acceptable non-standard designs. See Index 802.3 for further information on the organization and functions of the Committee.

Combinations of armor-type protection can be used, the slope revetment being of one type and the foundation treatment of another. The use of rigid, non-flexible slope revetment may require a flexible, self-adjusting foundation for example: concreted-rock on the slope with heavy rock foundation below, or PCC slope paving with a steel sheet-pile cutoff wall for foundation.

Bank protection may be damaged while serving its primary purpose. Lower cost replaceable facilities may be more economical than expensive permanent structures. However, an expensive structure may be economically warranted for highways carrying large volumes of traffic or for which no detour is available.

Cost of stone is extremely sensitive to location. Variables are length of haul, efficiency of the quarry in producing acceptable sizes, royalty to quarry and, necessity for stockpiling and rehandling. On some projects the stone may be available in roadway excavation.

873.2 Design High Water and Hydraulics

The most important, and often the most perplexing obligation, in the design of bank and shore protection features is the determination of the appropriate design high water elevation to be used. The design flood stage elevation should be chosen that best satisfies site conditions and level of risk associated with the encroachment. The basis for determining the design frequency, velocity, backwater, and other limiting factors should include an evaluation of the consequences of failure on the highway facility and adjacent property. Stream stability and sediment transport of a watercourse are critical factors in the evaluation process that should be carefully weighted and documented. Designs should not be based on an arbitrary storm or flood frequency.

A suggested starting point of reference for the determination of the design high water level is that the protection withstands high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. For example, a modern highway embankment can reasonably be expected to have a service life of 100 years or more. It would therefore be appropriate to base the preliminary evaluation on a high water elevation resulting from a storm or flood with a 2 percent probability of exceedance (50 year frequency of recurrence). The first evaluation may have to be adjusted, either up or down, to conform with a subsequent analysis which considers the importance of the encroachment and level of related risks which may include consideration of historic high water marks and climate change. Scour countermeasures protecting structures designed by the Division of Engineering Services (DES) may include consideration of floods greater than a 1 percent probability of exceedance (100 year frequency of recurrence).

There is always some risk associated with the design of protection features. Special attention must be given to life threatening risks such as those associated with floodplain encroachments. Significant floodplain risks are classified as those having probability of:

- Catastrophic failure with loss of life.
- Disruption of fire and ambulance services or closing of the only evacuation route available to a community.

Refer to Topic 804, Floodplain Encroachments, for further discussion on evaluation of risks and impacts.

(1) *Streambank Locations.* The velocity along the banks of watercourses with smooth or uniformly rough tangent reaches may only be a small percentage of the average stream velocity. However, local irregularities of the bank and streambed may cause turbulence that can result in the bank velocity being greater than that of the central thread of the stream. The location of these irregularities is not always permanent as they may be caused by local scour, deposition of rock and sand, or stranding of drift during high water changes. It is rarely economical to protect against all possibilities and therefore some damage should always be anticipated during high water stages.

Essential to the design of streambank protection is sufficient information on the characteristics of the watercourse under consideration. For proper analysis, information on the following types of watercourse characteristics must be developed or obtained:

- Design Discharge
- Design High Water Level
- Flow Types
- Channel Geometry
- Flow Resistance
- Sediment Transport

Refer to Chapter 810, Hydrology, for a general discussion on hydrologic analysis and specifically to Topic 817, Flood Magnitudes; Topic 818, Flood Probability and Frequency; and Topic 819, Estimating Design Discharge. For a detailed discussion on the fundamentals of alluvial channel flow, refer to Chapter 3, HDS No. 6, and to Chapter 4, HDS No. 6, for further information on sediment transport.

(2) *Ocean & Lake Shore Locations.* Refer to Chapter 880 for information needed to design shore protection.

873.3 Armor Protection

(1) *General.* Armor is the artificial surfacing of bed, banks, shore or embankment to resist erosion or scour. Armor devices can be flexible (self-adjusting) or rigid.

Hard armoring of stream banks, primarily with rock slope protection (RSP), has been the most common means of providing long-term protection for transportation facilities, and most importantly, the traveling public. With many years of use, dozens of formal studies and thousands of constructed sites, RSP is the armor type for which there exists the most quantifiable data on performance, constructability, maintainability and durability, and for which there exist several nationally recognized design methods.

Due to the above factors, RSP is the general standard against which other forms of armoring are compared.

The results of internal research led to the publication of Report No. FHWA-CA-TL-95-10, "California Bank and Shore Rock Slope Protection Design". Within that report, the methodology for RSP design adopted as the Departmental standard for many years, was the California Bank and Shore, (CaBS), layered design. The CaBS layered design methodology and its associated gradations have become obsolete. For reference only, the full report is available at the following website:

http://www.dot.ca.gov/hq/oppd/hydrology/hydroidx.htm.

FHWA Hydraulic Engineering Circular No. 23 (HEC 23) presents guidelines for RSP for a range of applications, including: RSP on streams and river banks, bridge piers and abutments, and bridge scour countermeasures such as guide banks and spurs. These guidelines were formally adopted by the Caltrans Bank and Shore Protection Committee with a modified version of HEC 23 gradations. See Tables 873.3A and 873.3B as well as HEC 23, Volume 1, Chapter 5 and Design Guideline 4, 5, 11, 12, 15 and 16 from Volume 2. Section 72 of the Standard Specifications provides all construction and material specifications for RSP designs. While standards (i.e., Standard Plans, Standard Specifications and/or SSP's) do exist for some other products discussed in this Chapter (most notably for gabions, but also for certain rolled or mat-style erosion control products), their primary application is for relatively flat slope or shallow ditch erosion control (gabions are also used as an earth retaining structure, see Topic 210 for more details).

Rigid and other armor types listed below are viable and may be considered where conditions warrant. Although the additional step of headquarters approval of any nonstandard designs is required, designers are encouraged to consider alternative designs, particularly those that incorporate vegetation or products naturally present in stream environments. The District Landscape Architect can provide design assistance together with specifications and details for the vegetative portion of this work.

(a) Flexible Types.

- Rock slope protection.
- Gabions, Standard Plan D100A and D100B.
- Precast concrete articulated blocks.

(b) Rigid Types.

- Concreted-rock slope protection.
- Partially-grouted rock slope protection.
- Sacked concrete slope protection.
- Concrete filled cellular mats.

(2) Bulkheads. The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Crib walls
- Sheet piling

(a) General Design Criteria. In selecting the type of flexible or rigid armor protection to use the following characteristics are important design considerations.

(1) The lower limit, or toe, of armor should be below anticipated scour or on bedrock. If for any reason this is not economically feasible, a reasonable degree of security can be obtained by placement of additional quantities of heavy rock at the toe which can settle vertically as scour occurs.

(2) In the case of slope paving or any expensive revetment which might be seriously damaged by overtopping and subsequent erosion of underlying embankment, extension above design high water may be warranted. The usual limit of extension for streambank protection above design high water is 1 foot to 2 feet in unconstricted reaches

and 2 feet to 3 feet in constricted reaches.

(3) The upstream terminal can be determined best by observation of existing conditions and/or by measuring velocities along the bank.

The terminal should be located to conform to outcroppings of erosion-resistant materials, trees, shrubs or other indications of stability.

In general, the upstream terminal on bends in the stream will be some distance upstream from the point of impingement or the beginning of curve where the effect of erosion is no longer damaging.

(4) When possible the downstream terminal should be made downstream from the end of the curve and against outcroppings, erosion-resistant materials, or returned securely into the bank so as to prevent erosion by eddy currents and velocity changes occurring in the transition length.

(5) The encroachment of embankment into the stream channel must be considered with respect to its effect on the conveyance of the stream and possible damaging effect on properties upstream due to backwater and downstream due to increased stream velocity or redirected stream flow.

(6) A smooth surface will generally accelerate velocity along the bank, requiring additional treatment (e.g., extended transition, cut-off wall, etc.) at the downstream terminal. Rougher surfaces tend to keep the thread of the stream toward the center of the channel.

(7) Heavy-duty armor used in exposures along the ocean shore may be influenced or dictated by economics, or the feasibility of handling heavy individual units.

(3) Flexible Revetments.

(a) Streambank Rock Slope Protection.

(1) General Features. This kind of protection, commonly called riprap, consists of rock courses placed upon the embankment or the natural slope along a stream. Rock, as a slope protection material, has a number of desirable features which have led to its widespread application.

It is usually the most economical type of revetment where stones of sufficient size and quality are available, it also has the following advantages:

- It is flexible and is not impaired nor weakened by slight movement of the embankment resulting from settlement or other minor adjustments.
- Local damage or loss is easily repaired by the addition of similar sized rock where required.
- Construction is not complicated and special equipment or construction practices are not usually necessary. (Note that Method A placement of very large rock may require large cranes or equipment with special lifting capabilities).
- Appearance is natural, and usually acceptable in recreational and scenic areas.
- If exposed to fresh water, vegetation may be induced to grow through the rocks adding structural value to the embankment material and restoring natural roughness. See Index 873.3(3)(a)(2)(d) for further vegetative rock slope protection information.
- Additional thickness (i.e., mounded toe design) can be provided at the toe to offset possible scour when it is not feasible to found it upon bedrock or below anticipated scour.

- It is salvageable, may be stockpiled and reused if necessary.

In designing the rock slope protection for a given embankment the following determinations are to be made for the typical section.

- Depth at which the stones are founded (bottom of toe trench).
- Elevation at the top of protection.
- Thickness of protection.
- Need for geotextile or rock filter material.
- Face slope.

(a) *Placement.* Two different methods of placement for rock slope protection are allowed under Section 72 of the Standard Specifications: Placement under Method A requires considerable care, judgment, and precision and is consequently more expensive than Method B. Method A should be specified primarily where large rock is required, but also for relatively steeper slopes.

(b) *Foundation Treatment.* The foundation excavation must afford a stable base on bedrock or extend below anticipated scour.

Terminals of revetments are often destroyed by eddy currents and other turbulence because of nonconformance with natural banks. Terminals should be secured by transitions to stable bank formations, or the end of the revetment should be reinforced by returns of thickened edges.

While a significant amount of research is currently being conducted, few methods exist for estimating scour along stream banks. One of the few is the method contained in HEC 23 Volume 1, Index 4.3.5 and the CHANLPRO Program developed by the U.S. Army Corps of Engineers. Based on the flume studies at the Corps' Waterways Experiment Station, the program is primarily used by the Corps for RSP designs on streams with 2 percent or lesser gradients, but contains an option for scour depth estimates in bends for sand channels. CHANLPRO is available at the following USACE website: http://chl.erdc.usace.army.mil/chanlpro along with a user guide containing equations, charts, assumptions and limitations to the method and example problems.

(c) *Embankment Considerations.* Embankment material is not normally carried out over the rock slope protection so that the rock becomes part of the fill. With this type of construction fill material can filter down through the voids of the large stones and that portion of the fill above the rocks could be lost. If it is necessary to carry embankment material out over the rock slope protection a geotextile is required to prevent the loses of fill material.

The embankment fill slope is usually determined from other considerations such as the angle of repose for embankment material, or the normal 1V:4H specified for high-standard roads. If the necessary size of rock for the given exposure is not locally available, consideration should be given to flattening of the embankment slope to allow a smaller size stone, or substitution of other types of protection. On high embankments, alternate sections on several slopes should be compared, practically and economically; flatter slopes require smaller stones in thinner sections, but at the expense of longer slopes, a lower toe elevation, increased embankment, and perhaps additional right of way.

Where the roadway alignment is fixed, slope flattening will often increase embankment encroachment into the stream. When such an encroachment is environmentally or technically undesirable, the designer should consider various vertical, or near vertical, wall type alternatives to provide adequate stream width, allowing natural channel migration and the opportunity for enhancing habitat.

(d) *Rock Slope Protection Fabric*. Rock Slope Protection fabrics are described in Standard Specification Section 96. The RSP fabric placement ensures that fine soil particles do not migrate through the RSP due to hydrostatic forces and, thus, eliminate the potential for bank failure. The use of RSP fabric provides an inexpensive layer of protection retaining embankment fines in lieu of placing a gravel filter of small, well graded materials. See Index 873.3(3)(a)(1)(e) "Gravel Filter."

Stronger and heavier RSP fabrics than those listed in the Standard Specifications are manufactured. They are used in special designs for larger than standard RSP sizes, or emergency installations where placement of large RSP must be placed directly on the fabric. These heavy weight fabrics have unit weights of up to 16 ounces per square yard. Contact the Headquarters Hydraulic Engineer for assistance regarding usage applications of heavy weight RSP fabrics.

(e) *Gravel Filter*. Generally RSP fabric should always be used unless there is a permit requirement that precludes the placement of fabric. Where RSP fabric cannot be placed, such as in stream environments where CA Fish & Wildlife and NOAA Fisheries strongly discourage the use of RSP Fabric, a gravel filter is usually necessary with most native soil conditions to stop fines from bleeding through the typical RSP classes. A gravel filter will be specified and placed between the native base soil and RSP for hybrid revetments to avoid conflicts associated with planting vegetation and placing RSP fabric together. A universal gravel filter gradation is presented in Design Information Bulletin No. 87 (see Table H, Index 7.1.2), which should work for many stream sites in California and eliminate the need for a site-specific gravel filter design for every project.

When a gravel filter is to be placed, the designer is advised to work with the District Materials Office to get a recommendation for the necessary gradation to work effectively with both the native backfill and the base layer of the RSP that is being placed. Among the methods available for designing the gravel filter are the Terzaghi method, developed exclusively for situations where the native backfill is sand, and the Cisten-Ziems method, which is often used for a broad variety of soil types and recommended in HEC 23. Where streambanks must be significantly rebuilt and reconfigured with imported material before RSP placement, the designer must ensure that the imported material will not bleed through the designed gravel filter. See HEC 23 Volume 2, Design Guideline 16, Index 16.2.1 Granular Filter Design Procedure and 16.3.1 Granular filter (design example).

(2) Streambank Protection Design. In the lower reaches of larger rivers wave action resulting from navigation or

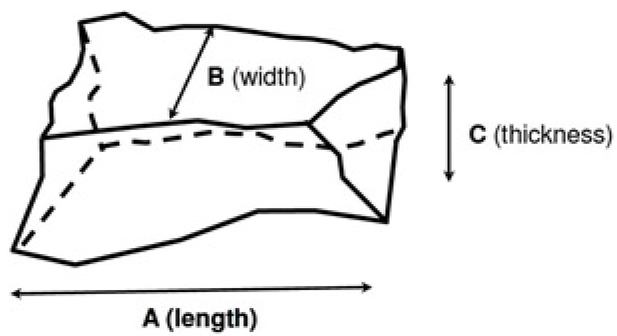
wind blowing over long reaches may be much more serious than velocity. A 2 foot wave, for example, is more damaging than direct impingement of a current flowing at 10 feet per second. Therefore, consideration of a wave attack based design may be necessary. See Chapter 880 for further information.

Well designed streambank rock slope protection should:

- Assure stability and compatibility of the protected bank as an integral part of the channel as a whole.
- Connect to natural bank, bridge abutments or adjoining improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.
- Eliminate or ease local embayments and capes so as to streamline the protected bank.
- Consider the effects of backwater above constrictions, superelevations on bends, as well as tolerance of occasional overtopping.
- Not be placed on a slope steeper than 1.5H:1V. Flatter slopes use lighter stones in a thinner section and encourage overgrowth of vegetation, but may not be permissible in narrow channels.
- Use stone of adequate weight to resist erosion, derived from Index 873.3(3)(a)(2)(b).
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric should be used.
- Rest on a good foundation on bedrock or extend below the depth of probable scour. If questionable, use heavy bed stones and provide a wide base section with a reserve of material to slough into local scour holes (i.e., mounded toe).
- Reinforce critical zones on outer bends subject to impinging flow, using heavier stones, thicker section, and deeper toe.
- Be constructed of rock of such shape as to form a stable protection structure of the required section. Rounded boulders or cobbles must not be used on prepared ground surfaces having slopes steeper than 2.5H:1V.

(a) *Stone Shape.* The shape of a stone can be generally described by designating three axes of measurement: major, intermediate, and minor, also known as the "A, B, and C" axes, as shown in Figure 873.3A.

Figure 873.3A
Stone Shape



Riprap stones should not be thin and platy, nor should they be long and needle-like. Therefore, specifying a maximum allowable value of the ratio A/C, also known as the shape factor, provides a suitable measure of particle shape, since the B axis is intermediate between the two extremes of length A and thickness C. A maximum allowable value for A/C of 3.0 is recommended.

Based on field studies, the recommended relationship between stone size and weight is given by:

Table 873.3A

RSP Class by Median Particle Size[3]

Nominal RSP Class by Median Particle Size[3]		d_{15}		d_{50}		d_{100}	Placement
Class [1], [2]	Size (in)	Min	Max	Min	Max	Max	Method
I	6	3.7	5.2	5.7	6.9	12.0	B
II	9	5.5	7.8	8.5	10.5	18.0	B
III	12	7.3	10.5	11.5	14.0	24.0	B
IV	15	9.2	13.0	14.5	17.5	30.0	B
V	18	11.0	15.5	17.0	20.5	36.0	B
VI	21	13.0	18.5	20.0	24.0	42.0	A or B
VII	24	14.5	21.0	23.0	27.5	48.0	A or B
VIII	30	18.5	26.0	28.5	34.5	48.0	A or B
IX	36	22.0	31.5	34.0	41.5	52.8	A
X	42	25.5	36.5	40.0	48.5	60.5	A
XI	46	28.0	39.4	43.7	53.1	66.6	A

NOTES:

(1) Rock grading and quality requirements per Standard Specifications.

(2) RSP-fabric Type of geotextile and quality requirements per Section 96 Rock Slope Protection Fabric of the Standard Specifications. For RSP Classes I thru VIII, use Class 8 RSP-fabric which has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric. For RSP Classes IX thru XI, use Class 10 RSP-fabric.

(3) Intermediate, or B dimension (i.e., width) where A dimension is length, and C dimension is thickness.

Table 873.3B

RSP Class by Median Particle Weight[3]

Nominal RSP Class by Median Particle Weight		W_{15}		W_{50}		W_{100}	Placement
Class [1], [2]	Weight	Min	Max	Min	Max	Max	Method
I	20 lb	4	11	15	27	140	B
II	60 lb	14	39	50	94	470	B
III	150 lb	32	94	120	220	1,100	B
IV	300 lb	63	180	250	440	2,200	B
V	1/4 ton	110	300	400	700	3,800	B
VI	3/8 ton	180	520	650	1,100	6,000	A or B
VII	1/2 ton	250	750	1000	1,700	9,000	A or B
VIII	1 ton	520	1,450	1,900	3,300	9,000	A or B
IX	2 ton	870	2,500	3,200	5,800	12,000	A
X	3 ton	1,350	4,000	5,200	9,300	18,000	A
XI	4 ton	1,800	5,000	6,800	12,200	24,000	A

NOTES:

(1) Rock grading and quality requirements per Standard Specifications.

(2) RSP-fabric Type of geotextile and quality requirements per Section 96 Rock Slope Protection Fabric of the Standard Specifications. For RSP Classes I thru VIII, use Class 8 RSP-fabric which has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric. For RSP Classes IX thru XI, use Class 10 RSP-fabric.

(3) Values shown are based on Table 873.3A dimensions and an assumed specific gravity of 2.65. Weight will vary based on density of rock available for the project.

$$W = 0.85(\gamma_s d^3)$$

Where:

W = Weight of stone, lb;

d = Size of intermediate ("B") axis, ft;

γ_s = Density of stone, lb/ft^3;

= $S_g \gamma_w$

Where:

γ_w = 62.4 lb/ft^3;

S_g = Specific gravity of stone.

Tables 873.3A and 873.3B provide recommended gradations for eleven standard classes of riprap based on median particle size d_{50} as determined by the dimension of the intermediate ("B") axis. The D or W refers to size or weight, respectively. The number is the percent finer by weight. Tables 873.3A and 873.3B are modified versions of Tables 4.1 and 4.2 in HEC 23, Volume 2, Design Guideline 4, which provide recommended gradations for ten standard classes of riprap and conform to those recommended in NCHRP Report 568 (Lagasse et al. 2006). The gradation criteria in Table 873.3A are based on a nominal or "target" d_{50}. See Index 873.3(3)(a)(2)(b) for equations to calculate d_{30} and d_{50}. The most significant modifications to Tables 873.A and 873.B from the gradations shown in Tables 4.1 and 4.2 are to the d_{100max} and W_{100max} gradation for classes VIII through XI, which have been truncated for practicality. An additional class XI is included in Tables 873.3A and 873.3B. Contact the Headquarters Hydraulic Engineer if more information is needed on the modification to the HEC 23 gradations.

Based on the recommended relationship between size and weight, which assumes the volume of the stone is 85% of a cube, Table 873.3B provides the equivalent particle weights for the same eleven classes as Table 873.3A using a specific gravity of 2.65 for the particle density.

(b) Stone Size. Where stream velocity governs, rock size may be estimated from the following formula, which can be used with uniform or gradually varying flow. Coefficients are included to account for the desired safety factor for design, specific gravity of the riprap stone, bank slope, and bendway character;

$$d_{30} = y(S_f C_S C_V C_T)\left[\frac{V_{des}}{\sqrt{K_1(S_g - 1)gy}}\right]^{2.5}$$

Where:

d_{30} = Particle size for which 30% is finer by weight, ft;

y = Local depth of flow, ft;

S_f = Safety factor (typically = 1.1);

C_S = Stability coefficient (for blanket thickness 1.5d_{50} or d_{100}, whichever is greater) = 0.30 for angular rock;

C_V = Velocity distribution coefficient;

= 1.0 for straight channels or the inside of bends;

= 1.283 – 0.2 log (R_c/W) for the outside of bends (1.0 for R_c/W > 26);

= 1.25 downstream from concrete channels;

= 1.25 at the end of dikes;

C_T = Blanket thickness coefficient = 1.0;

S_g = Specific gravity of stone (2.5 minimum);

g = Acceleration due to gravity, 32.2 ft/s^2;

V_{des} = Characteristic velocity for design, defined as the depth-averaged velocity at a point 20% upslope from the toe of the revetment, ft/s;

For natural channels, $V_{des} = V_{avg}\,(1.74 - 0.52 \log (R_c/W))$

$V_{des} = V_{avg}$ for $R_c/W > 26$

For trapezoidal channels, $V_{des} = V_{avg}\,(1.71 - 0.78 \log (R_c/W))$

$V_{des} = V_{avg}$ for $R_c/W > 8$

Where:

R_c = Centerline radius of curvature of channel bend, ft;

W = Width of water surface at upstream end of channel bend, ft;

V_{avg} = Channel cross-sectional average velocity, ft/s;

K_1 = Side slope correction factor;

$$K_1 = \sqrt{1 - \left[\frac{\sin(\theta - 14°)}{\sin 32°}\right]^{1.6}}$$

Where:

θ = is the bank angle in degrees.

The flow depth "y" used in the above equation is defined as the local flow depth. The flow depth at the toe of slope is typically used for bank revetment applications; alternatively, the average channel depth can be used. The smaller of these values will result in a slightly larger computed d_{30} size, since riprap size is inversely proportional to ($y^{0.25}$). The blanket thickness coefficient (C_T) is 1.0 for standard riprap applications where the thickness is equal to $1.5d_{50}$ or d_{100}, whichever is greater. Because limited data is available for selecting lower values of C_T when greater thicknesses of riprap are used, a value of 1.0 is reasonable for all applications. The recommended Safety Factor S_f is 1.1 for bank revetment. Greater values should be considered where there is significant potential for ice or impact from large debris, freeze-thaw degradation that would significantly decrease particle size, or large uncertainty in the design variables, especially velocity. The specific gravity (S_g) of stone is commonly taken as 2.65 for planning purposes, however, this will result in a less conservative design than utilizing a 2.5 specific gravity assumption, which would be the minimum accepted in the field. Therefore, the designer should contact the District Materials Engineer in the project's area and determine if there is any history of RSP materials used in that region. Where such information or history is unavailable, use of a 2.5 specific gravity within the design should be considered.

The d_{30} size of the riprap is related to the recommended median (d_{50}) size by:

$$d_{50} = 1.20 d_{30}$$

Using standard sizes the appropriate gradation can be achieved by selecting the next larger size class, thereby creating a slightly over-designed structure, but economically a less expensive one. For example, if a riprap sizing calculation results in a required d_{50} of 16.8 inches, Class V riprap should be specified because it has a nominal d_{50} of 18 inches. See Table 873.3A.

A limitation to the rock size equation above is that the longitudinal slope of the channel should not be steeper than 2.0% (0.02 ft/ft). For steeper channels, the riprap sizing approach for overtopping flows presented in HEC 23, Volume 2, Design Guideline 5 should be considered and the results compared with the rock size equation above .

Where wave action is dominant, design of rock slope protection should proceed as described for shore protection, see Chapter 880.

(c) *Design Height*. The top of rock slope protection along a stream bank should be carried to the elevation of the design high water plus some allowance for freeboard. Cost and severity of damage if overtopped as well as the importance of the facility should also be considered. The goal for the design high water is based on the 50-year (2% probability) flow, but can be modified using engineering judgment which may include consideration of historic high water marks and climate change. This stage may be exceeded during infrequent floods, usually with little or no damage to the upper slope. See Hybrid RSP cross section in Figure 873.3D for an example showing the top of rock slope protection.

When determining freeboard, or the height above design high water from which the RSP is to extend, one should consider: the size and nature of debris in the flow; the resulting potential for damage to the bank, the potential for streambed aggradation; and the confidence in data used to estimate design highwater. Freeboard may also be affected by regulatory or local agency requirements. Freeboard may be more generous on the outside bends of channels, or around critical bridges.

The 50-year design high water plus freeboard goal should be followed whenever possible, but the biggest exception to this goal occurs when the design height exceeds the main channel top of bank. Because floodplain overbank areas can be wide and extensive, the footprint of the RSP could grow exponentially if extended above and beyond the top of bank. This increased footprint would bring higher costs and permitting challenges that could make a project no longer viable. Given this possibility, the RSP vertical limit (height) should typically end at the main channel top of bank; however, a vegetation component may extend above and beyond the top of bank.

For cases where significant erosion has occurred above the main channel top of bank into its overbank(s), contact the District Hydraulic Engineer to discuss alternatives for repair and protection.

Design Example -- The following example reflects the HEC 23 method for designing RSP. The designer is encouraged to review Design Guideline 4, Riprap Revetment from HEC 23, Volume 2. The following example assumes that the designer has conducted the appropriate site assessments and resulting calculations to establish average stream velocity, flow depth at bank toe, estimated depth of scour, stream alignment (i.e., parallel or impinging flow), width of channel, radius of bend (if impinging flow), length and side slope of stream bank to be protected and locations of natural hard points (e.g., rock outcroppings). Field reviews and discussions with maintenance staff familiar with the site are critical to the success of the design.

Given for example:

- Average stream velocity for design event of 9.8 feet per second
- Flow depth of 11.4 feet at bank toe
- Estimated scour depth – 3.5 feet
- Length of bank requiring protection – 550 feet
- Bank slope – 2:1
- Specific gravity of rock used for RSP – 2.54 (based on data from local quarry)
- Embankment is on outside of stream bend of 100 ft wide natural channel on a bend that has a centerline radius (R_c) of 500 ft. The radius of curvature divided by width (R_c/W) is 5.0.
- A desired factor of safety (Sf) of 1.2.

Determine the target d_{50}, select appropriate RSP class from Table 873.3A and determine the blanket thickness:

Step 1: Compute the side slope correction factor:

$$K_1 = \sqrt{1 - \left(\frac{\sin(\theta - 14°)}{\sin 32°}\right)^{1.6}}$$

$$= \sqrt{1 - \left(\frac{\sin(26.6° - 14°)}{\sin 32°}\right)^{1.6}}$$

$= 0.87$

Step 2: Select the appropriate stability coefficient for riprap: C_s (for blanket thickness $1.5d_{50}$ or d_{100}, whichever is greater) = 0.30 for angular rock

Step 3: Compute the vertical velocity factor (C_v) for R_c/W = 5.0:

$C_v = 1.283 - 0.2 \log (R_c/W)$

$= 1.283 - 0.2 \log (5.0)$

$= 1.14$

Step 4: Compute local velocity on the side slope (V_{des}) for a natural channel with R_c/W = 5.0:

$V_{des} = V_{avg} [1.74 - 0.52 \log (R_c/W)]$

$= 9.8[1.74 - 0.52 \log (5.0)]$

$= 13.5$ ft/s

Step 5: Compute the d_{30} size using stone size equation from Index 873.2(2)(a)(2)(b):

$$d_{30} = S_f C_s C_v y \left[\frac{V_{des}}{\sqrt{(Sg-1)K_1 g y}}\right]^{2.5}$$

$$= (1.2)(0.3)(1.14)(11.4) \times \left[\frac{13.5}{\sqrt{(2.54-1)(0.87)(32.2)(11.4)}}\right]^{2.5}$$

$= 1.35$ ft

Step 6: Compute the d_{50} size = $1.2d_{30}$ = 1.2(1.35)

= 1.62 ft = 19 inches.

Note: Use next larger size class (see Table 873.3A)

Step 7: Select Class VI riprap from Table 873.3A: d_{50} = 21 inches

Step 8: Blanket thickness = $1.5d_{50}$ or d_{100}, whichever is greater

$1.5d_{50}$ = 1.5 (21 inches)

= 31.5 inches

d_{100} = 42 inches, therefore, use 42 inches

Step 9: Determine the depth of riprap embedment below the streambed at the toe of the bank slope:

Since toe scour is expected to be 3.5 ft, the 2H:1V slope should be extended below the ambient bed level 7 ft horizontally out from the toe to accommodate this scour. Alternatively, a mounded riprap toe 3.5 ft high could be established at the base of the slope and allowed to self-launch when toe scour occurs, see Figure 873.3D.

Step 10: Assess Stream Impact Due to Revetment. In some cases, the thickness of the completed RSP revetment creates a narrowing of the available stream channel width, to the extent that stream velocity or stage at the design event is increased to undesirable levels, or the opposite bank becomes susceptible to attack. In these cases, the bank upon which the RSP is to be placed must be excavated such that the constructed face of the revetment is flush with the original embankment.

Step 11: Exterior Edges of Revetment. The completed design must be compatible with existing and future conditions. Freeboard and top edge of revetments were covered in Index 873.3(2)(a)(2)(c) "Design Height." For depth of toe, the estimated scour was given as 3.5 feet. This is the minimum toe depth to be considered. Again, based on site conditions and discussions with maintenance staff and others, determine if any long-term conditions need to be addressed. These could include streambed degradation due to local aggregate mining or headcutting. Regardless of the condition, the toe must be founded below the lowest anticipated elevation that could become exposed over the service life of the embankment or roadway facility. As for the upstream and downstream ends, the given length of revetment is 550 feet. Again, this will typically be a minimum, as the designer should seek natural rock outcroppings, areas of quiescent stream flow, or other inherently stable bank segments to end the RSP.

(d) *Vegetated Rock Slope Protection*. The use of vegetation in streambank stabilization has positive attributes on stream integrity, such as improving stream ecology, increasing soil strength, and providing flow resistance, but vegetation can also have negative impacts on stream integrity by altering conveyance characteristics of the stream, affecting soil characteristics, in addition to being unpredictable in its long term establishment and performance.

Streams with stable vegetation typically have good water quality, as well as good biological and chemical health due in part to the ability of the vegetation to filter pollutants including nitrates and phosphates through their uptake of moisture in the soil. Vegetation will also promote good fish, wildlife, and aquatic organism habitat by providing cover, reducing stream temperature and controlling temperature fluctuations, and supplying an organic food source. In addition to ecological improvements, vegetation can strengthen the underlying soils. It can create additional cohesion and binding properties through its roots. The fibrous woody roots are strong in tension, but weak in compression, which is the opposite case for soil. Therefore, roots and soil working in tandem can complement the other providing a material that has both tension and compression resistance.

Vegetation can also improve soil strength by lowering pore-water pressure through its soil moisture extraction.

These benefits of the vegetation root system also carry some negative effects. Their additional mass and surcharge can increase slope failure potential under saturated conditions where the magnitude of saturation can actually be compounded because of root development. Another positive effect of vegetation use in revetments is its ability to improve flow resistance creating higher roughness that will dissipate energy, shear stress, and velocity. The vegetation deflects velocity upwards away from the streambank, which reduces the influence of drag and lift. For example, willows planted on a streambank have the capacity to deflect and resist velocities up to 10 feet/second in their mature state, which would equate to a 12-inch to 18-inch rock (RSP Class III to IV) having similar permissive velocity. To reach this point, it may take three to five years. In the first few years after planting, the vegetation is providing little resistance. During this establishment period, the streambanks can be subject to scour and erosion because of the lack of flow resistance without some other means of protection. Even after vegetation reaches maturity and beyond, potential exists for it to succumb to drought conditions or to yield to large flows/velocities and break apart rendering the vegetation ineffective to dissipating velocity and hydraulic forces. Because the stages of vegetation growth can be dynamic as it is affected by drought or high flows, the vegetation may go through a reestablishment process, and the n-value and velocity/flow resistance will also be dynamic making revetment performance unpredictable. Even though the use of vegetation in bank stabilization may have negative effects, its ecological benefits generally outweigh them.

The design premise is to use rock and vegetation together in a streambank revetment in such a way that will highlight their positive attributes while also addressing and managing their negative impacts. In the design of hybrid revetments, mounded toes referenced in Index 873.3(3)(a)(1) are not recommended because of their encroachment into the middle of the channel, which can impact cross-sectional area and capacity. With the use of vegetation on the bank and possible projection toward the middle of the channel, cross-sectional area could possibly be impacted as well. A mounded toe used with bank vegetation would only exacerbate this issue, therefore an embedded toe is chosen for hybrid revetment application. See Figure 873.3D for an example cross section of hybrid RSP with an embedded toe. For hybrid revetment design, the 50-year (2% probability) flood event should be used. Per Index 873.2, depending on the importance of the encroachment and level of related risks, subsequent analysis may consider historic high water marks and climate change for design. In order to manage possible negative impacts from vegetation use, planting needs to be performed in a controlled manner. Placement of vegetation within the bank-toe zone and the main channel is highly discouraged to keep turbulence intensity in check that could cause excessive sediment accumulation.

Plant mortality must be considered during the initial planting and establishment period. Overplanting must be avoided so that high density and projection does not occur causing increased sediment deposition and capacity/conveyance reduction. Given these issues, plant density in the design of a hybrid revetment and consideration of natural plant density is critical to the performance of the hybrid revetment. The goal for design should be medium density, where horizontal projection and cross-sectional area reduction at maturity are minimal. See Figure 873.3B. For woody vegetation, medium density is described as mature trees or shrubs with full foliage on a streambank, where preferably individual canopies or outer layers retain some free space between them, but may have minimal overlapping without being interwoven.

Figure 873.3B
Medium Density Vegetation

Lower limit of medium vegetation density

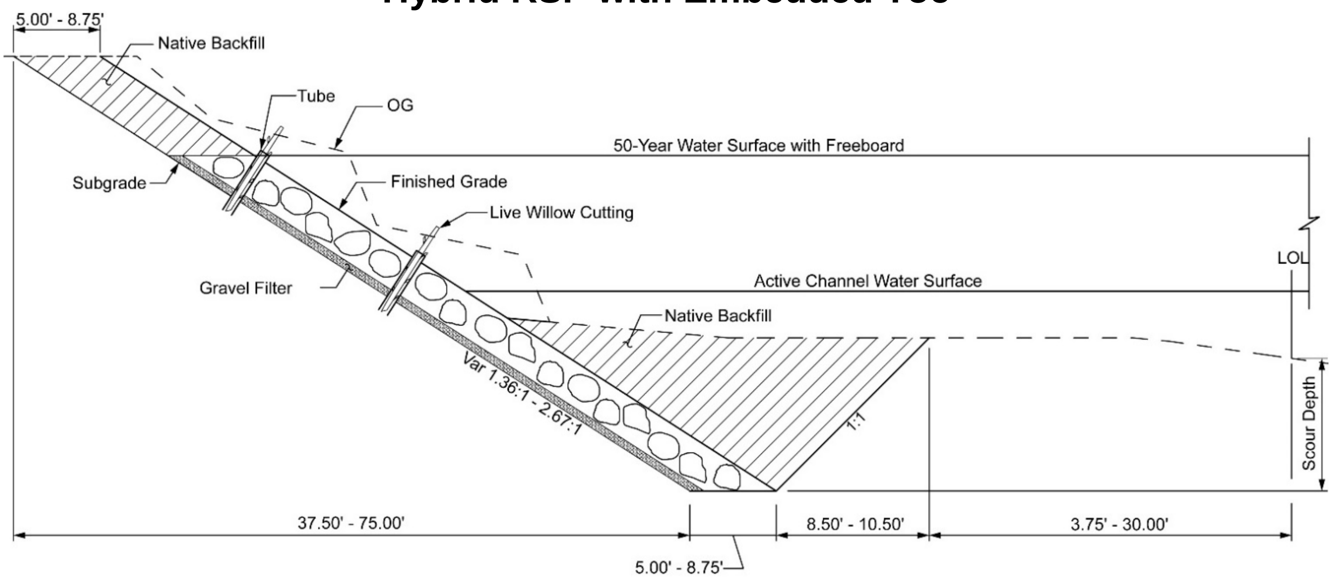
Pre-construction and post-construction hydraulic modelling and hybrid revetment design are discussed in more detail in Design Information Bulletin No. 87. For rock sizing, Index 7.1.1.2 should be substituted with Index 873.3(3)(a)(2)(b) of this manual.

(e) *Gabions*. Gabion revetments consist of rectangular wire mesh baskets filled with stone. See Standard Plan D100A and D100B for gabion basket details and the Standard Specifications for requirements.

Gabions are formed by filling commercially fabricated and preassembled wire baskets with rock. There are two types of gabions, wall type and mattress type. In wall type the empty cells are positioned and filled in place to form walls in a stepped fashion. Mattress type baskets are positioned on the slope and filled. See HEC 23, Volume II, Design Guideline 10 and Figure 873.3B. Wall type revetment is not fully self-adjusting but has some flexibility. The mattress type is very flexible and well suited for man-made roadside channels (with uniform flow) discussed in Chapter 860 and as overside drains that are constructed on steep, unstable slopes. For some stream locations, gabions may be more aesthetically acceptable than rock riprap or may be considered when larger stone sizes are not readily available and flows are nonabrasive. Due to abrasion, corrosion and vandalism concerns and difficulty of repairs, caution is advised regarding in-stream placement of gabions. In addition, the California Department of Fish and Wildlife recommends against using gabions as weirs in streams. If gabions are placed in-stream, some form of abrasion protection in the form of wooden planks or other facing will typically be necessary for wall type, see Figure 873.3C. Maintenance-free design service life in most

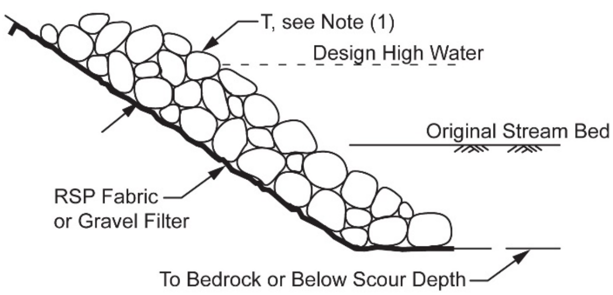
Figure 873.3D
Rock Slope Protection

Hybrid RSP with Embedded Toe

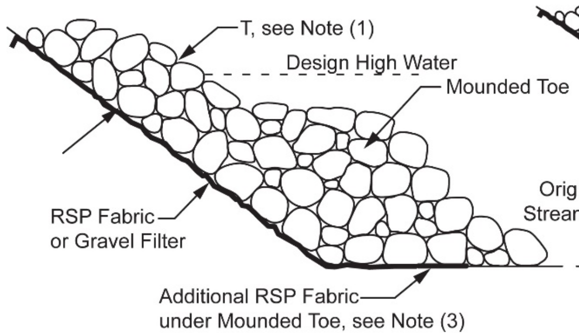


Rock Slope Protection

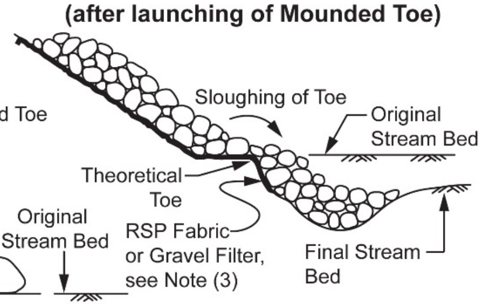
Embedded Toe RSP



Mounded Toe RSP
(as constructed)



Mounded Toe RSP
(after launching of Mounded Toe)



NOTES:

(1) Thickness "T" = 1.5 d_{50} or d_{100}, whichever is greater.

(2) Face stone size is determined from Index 873.3(2)(a)(2)(a).

(3) RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

environments is generally under 20 years.

Figure 873.3C
Gabion Lined Streambank

Gabion wall with timber facing to protect wires from abrasive flow.

(f) Articulated Precast Concrete. This type of revetment consists of pre-cast concrete blocks which interlock with each other, are attached to each other, or butted together to form a continuous blanket or mat. A number of block designs are commercially available. They differ in shape and method of articulation, but share common features of flexibility and rapid installation. Most provide for establishment of vegetation within the revetment.

The permeable nature of these revetments permits free draining of the embankment and their flexibility allows the mat to adjust to minor changes in bank geometry. Pre-cast concrete block revetments may be economically justified where suitable rock for slope protection is not readily available. They are generally more aesthetically pleasing than other types of revetment, particularly after vegetation has become established.

Individual blocks are commonly joined together with steel cable or synthetic rope, to form articulated block mattresses. Pre-assembled in sections to fit the site, the mattresses can be used on slopes up to 2:1. They are anchored at the top of the revetment to secure the system against slippage.

Pre-cast block revetments that are formed by butting individual blocks end to end, with no physical connection, should not be used on slopes steeper than 3:1. An engineering fabric is normally used on the slope to prevent the migration of the underlying embankment through the voids in the concrete blocks.

Refer to HEC 11, Design of Riprap Revetment, Section 6.2, and HEC 23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 4, for further discussion on the use of articulated concrete blocks.

(4) Rigid Revetments.

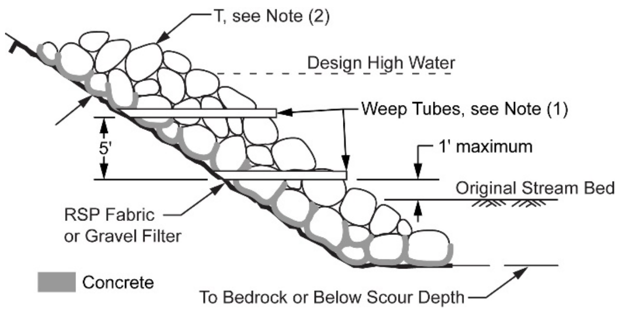
(a) Concreted-Rock Slope Protection.

(1) General Features. This type of revetment consists of rock slope protection with interior voids filled with PCC to form a monolithic armor. A typical section of this type of installation is shown in Figure 873.3E.

It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available.

(2) Design Concepts. Concreting of RSP is a common practice where availability of large stones is limited, or where there is a need to reduce the total thickness of a RSP revetment. Inclusion of the concrete, and the labor required to place it, makes concreted RSP installations more expensive per unit area than non-concreted installations.

Figure 873.3E
Concreted-Rock Slope Protection



NOTES:

(1) If needed to relieve hydrostatic pressure.

(2) $1.5d_{50}$ or d_{100}, whichever is greater from Table 873.3A for section thickness.

Dimensions and details should be modified as required.

Design procedures for concreted RSP revetments are similar to that of non-concreted RSP. Start by following the design example provided in Index 873.3(3)(a)(2)(c) to select a stable rock class for a non-concreted design based on the d_{50} and the next larger class in Table 873.3A. This non-concreted rock size is divided by a factor of roughly four or five to arrive at the appropriate d_{50} size rock for a concreted revetment. The factor is based on observations of previously constructed facilities and represents the typical sized pieces that stay together even after severe cracking (i.e., failed revetments will still usually have segments of four to five rocks holding together). As with the non-concreted design procedures, use the rock size derived from this calculation to enter Table 873.3A (i.e., round up to the next larger d_{50} rock to select the appropriate RSP Class.

As this type of protection is rigid without high strength, 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 1.5:1. Precautions to prevent undermining of embankment are particularly important, see Figure 873.3F. The concreted-rock must be founded on solid rock or below the depth of possible scour. Ends should be protected by tying into stable rock or forming smooth transitions with embankment subjected to lower velocities. As a precaution, cutoff stubs may be provided. If the embankment material is exposed at the top, freeboard is warranted to prevent overtopping.

Figure 873.3F
Toe Failure - Concreted RSP

Toe of concreted RSP that has been undermined.

The design intent is to place an adequate volume of concrete to tie the rock mass together, but leave the outer face roughened with enough rock projecting above the concrete to slow flow velocities to more closely approximate natural conditions.

The volume of concrete required is based on filling roughly two-thirds of the void space of the rock layer, as shown in Figure 873.3E. The concrete is rodded or vibrated into place leaving the outer stones partially exposed. Void space for the various RSP gradations ranges from approximately 30 percent to 35 percent for Method A placed rock to 40 percent to 45 percent for Method B placed rock of the total volume placed.

Specifications. Quality specifications for rock used in concreted-rock slope protection are usually the same as for rock used in ordinary rock slope protection. However, as the rocks are protected by the concrete which surrounds them, specifications for specific gravity and hardness may be lowered if necessary. The concrete used to fill the voids is normally 1 inch maximum size aggregate minor concrete. Except for freeze-thaw testing of aggregates, which may be waived in the contract special provisions, the concrete should conform to the provisions of Standard Specification Section 90.

Size and grading of stone and concrete penetration depth are provided in Standard Specification Section 72.

(b) Partially Grouted Rock Slope Protection. Partially grouted rock slope protection (PGRSP) is a viable alternative to larger rock or concreted rock slope protection where either the availability of large material is limited, or site limitations regarding placement of large material (e.g., no excavation below spread footing base) would lead the designer to consider using some form of smaller rock held together with a cementitious material. With partially grouted rock slope protection, there are no relationships per se for selecting the size of rock, other than the practical considerations of proper void size, gradation, and adequate stone-to-stone contact area. The intent of partial grouting is to "glue" stones together to create a conglomerate of particles. Each conglomerate is therefore significantly larger than the d_{50} stone size, and typically is larger than the d_{100} size of the individual stones in the matrix. The proposed gradation criteria are based on a nominal or "target" d_{50} and only stones with a d_{50} ranging from 9 inches to 15 inches may be used with the partial grouting technique. See rock classes II, III and IV in Table 873.3A. In HEC 23, PGRSP is presented as a pier scour countermeasure, but it may be also used for bridge abutment protection, as well as for bed/bank protection for short localized areas with high velocities and shear stresses that require a smaller rock footprint than a non-grouted design. Both Headquarters Office of Highway Drainage Design and District biologist staff should be consulted early on during the planning phase for subject matter expertise relative to design and obtaining project specific permits. For more guidance, see HEC 23, Volume 2, Design Guideline 12.

(c) Sacked-Concrete Slope Protection. This method of protection consists of facing the embankment with sacks filled with concrete. It is expensive, but historically was a much used type of revetment. Much hand labor is required but it is simple to construct and adaptable to almost any embankment contour. Use of this method of slope protection is generally limited to replacement or repair of existing sacked concrete facilities, or for small, unique situations that lend themselves to hand-placed materials.

Tensile strength is low and as there is no flexibility, the installation 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. Slopes steeper than 1:1 are rare; 1.5:1 is common. The flatter the slope, the less is the area of bond between sacks. From a construction standpoint it is not practical to increase the area of bond between sacks; therefore for slopes as flat as 2:1 all sacks should be laid as headers rather than stretchers.

Integrity of the revetment can be increased by embedding dowels in adjoining sacks to reinforce intersack bond. A No. 3 deformed bar driven through a top sack into the underlying sack while the concrete is still fresh is effective. At cold joints, the first course of sacks should be impaled on projecting bars that were driven into the last previously placed course. The extra strength may only be needed at the perimeter of the revetment.

Most failures of sacked concrete are a result of stream water eroding the embankment material from the bottom, the ends, or the top.

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 are to be provided and if the protection is long, cutoff stubs are built at 30-foot intervals, in order to prevent or retard a progressive failure.

Protection should be high enough to preclude overtopping. If the roadway grade is subject to flooding and the shoulder material does not contain sufficient rock to prevent erosion from the top, then pavement should be carried over the top of the slope protection in order to prevent water entering from this direction.

Class 8 RSP fabric as described in Standard Specification Section 96 should be placed behind all sacked concrete revetments. For revetments over 4 feet in height, weep tubes should also be placed, see Figure 873.3E.

For good appearance, it is essential that the sacks be placed in horizontal courses. If the foundation is irregular, corrective work such as placement of entrenched concrete or sacked concrete is necessary to level up the foundation. Refer to HDS No. 6, Section 6.6.5, for further discussion on the use of sacked concrete slope protection.

(5) Bulkheads. A bulkhead is a steep or vertical structure supporting a natural slope or constructed embankment. As bank protection structures, bulkheads serve to secure the bank against erosion as well as retaining it against sliding. As a retaining structure, conventional design methods for retaining walls, cribs and laterally loaded piles are used.

Bulkheads are usually expensive, but may be economically justified in special cases where valuable riparian property or improvements are involved and foundation conditions are not satisfactory for less expensive types of slope protection. They may be used for toe protection in combination with other revetment types of slope protection. Some other considerations that may justify the use of bulkheads include:

- Encroachment on a channel cannot be tolerated.
- Retreat of highway alignment is not viable.
- Right of Way is restricted.
- The force and direction of the stream can best be redirected by a vertical structure.

The foundation for bulkheads must be positive and all terminals secure against erosive forces. The length of the structure should be the minimum necessary, with transitions to other less expensive types of slope protection when possible. Eddy currents can be extremely damaging at the terminals and transitions. If overtopping of the bulkheads is anticipated, suitable protection should be provided.

Along a stream bank, using a bulkhead presumes a channel section so constricted as to prohibit use of a cheaper device on a natural slope. Velocity will be unnaturally high along the face of the bulkhead, which must have a fairly smooth surface to avoid compounding the restriction. The high velocity will increase the threat of scour at the toe and erosion at the downstream end. Allowance must be made for these threats in selecting the type of foundation, grade of footing, penetration of piling, transition, and anchorage at downstream end. Transitions at both ends may appropriately taper the width of channel and slope of the bank. Transition in roughness is desirable if attainable. Refer to HDS No. 6, Section 6.4.8, for further discussion on the use of bulkheads to prevent streambank erosion or failure.

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

(b) Crib walls. Timber and concrete cribs can be used for bulkheads in locations where some flexibility is desirable or permissible. Metal cribs are limited to support of embankment and are not recommended for use as protection because of vulnerability to corrosion and abrasion.

The design of crib walls is essentially a determination of line, foundation grade, and height with special attention given to potential scour and possible loss of backfill at the base and along the toe. Design details for concrete crib walls are shown on Standard Plans C7A through C7C. Concrete crib walls used as bulkheads and exposed to salt water require special provisions specifying the use of coated rebars and special high density concrete. Recommendations from METS Corrosion Technology Branch should be requested.

(c) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 15 feet below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(6) *Vegetation*. Vegetation is the most natural method for stabilization of embankments and channe bank protection. Vegetation can be relatively easy to maintain, visually attractive and environmentally desirable. The root system forms a binding network that helps hold the soil. Grass and woody plants above ground provide resistance to the near bank water flow causing it to lose some of its erosive energy.

Erosion control and revegetation mats are flexible three-dimensional mats or nets of natural or synthetic material that protect soil and seeds against water erosion prior to establishment of vegetation. They permit vegetation growth through the web of the mat material and have been used as temporary channel linings where ordinary seeding and mulching techniques will not withstand erosive flow velocities. The designer should recognize that flow velocity estimates and a particular soils resistance to erosion are parameters that must be based on specific site conditions. Using arbitrarily selected values for design of vegetative slope protection without consultation with the District Hydraulic Unit and/or the District Landscape Architect Unit is not recommended. However, a suggested starting point of reference is Table 865.2 in which the resistance of various unprotected soil classifications to flow velocities are given. Under near ideal conditions, ordinary seeding and mulching methods cannot reasonably be expected to withstand sustained flow velocities above 4 feet per second. If velocities are in excess of 4 feet per second, a lining maybe needed, see Table 865.2.

Temporary channel liners are used to establish vegetative growth in a drainage way or as slope protection prior to the placement of a permanent armoring. Some typical temporary channel liners presented in Table 865.2 are:

- Single net straw
- Double net coconut/straw blend
- Double net shredded wood

Vegetative and temporary channel liners are suitable for conditions of uniform flow and moderate shear stresses.

Permanent soil reinforcing mats and rock riprap may serve the dual purpose of temporary and permanent channel liner. Some typical permanent channel liners are:

- Small rock slope protection
- Geosynthetic mats
- Polyethelene cells or grids

- Gabion Mattresses (see Index 873.3(3)(a)(2)(e))

However, geosynthetics and plastic (polyethylene, polypropylene, polyamide, etc.) based mats with no enhanced UV resistance must be installed in a fashion where there will be no potential for long-term sunlight exposure, as these products will degrade due to UV radiation.

Composite designs are often used where there are sustained low flows of high to moderate velocities and intermediate high water flows of low to moderate velocities. Brush layering is a permanent type of erosion control technique that may also have application for channel protection, particularly as a composite design.

Additional design information on vegetation, and temporary and permanent channel liners is given in Chapter IV, HEC 15, Design of Roadside Channels and Flexible Linings and in Chapter 860 of this manual.

873.4 Training Systems

(1) General. Training systems are structures, usually within a channel, that act as countermeasures to control the direction, velocity, or depth of flowing water. When training systems are used, they generally straighten the channel, shorten the flow line, and increase the local velocity within the channel. Any such changes made in the system that cause an increase in the gradient may cause an increase in local velocities. The increase in velocity increases local and contraction scour with subsequent deposition downstream, where the channel takes on its normal characteristics. If significant lengths of the river are trained and straightened, there can be a noticeable decrease in the elevation of the water surface profile for a given discharge in the main channel. Tributaries emptying into the main channel in such reaches are significantly affected. Having a lower water level in the main channel for a given discharge means that the tributary streams entering in that vicinity are subjected to a steeper gradient and higher velocities which can cause degradation in the tributary streams. In extreme cases, degradation can be induced of such magnitude as to cause failure of structures such as bridges, culverts or other encroachments on the tributary systems. In general, any increase in transported materials from the tributaries to the main channel causes a reduction in the quality of the environment within the river.

(a) Bendway Weirs. Bendway weirs, also referred to as stream barbs, bank barbs, and reverse sills are low elevation stone sills used to improve lateral stream stability and flow alignment problems at river bends and highway crossings on streams and smaller rivers.

They are placed at an angle with the embankment in meandering streams for the purpose of directing or forcing the current away from the embankment, see Figure 873.4A. They also encourage deposition of bed material and growth of vegetation. When the purpose is to deposit material and promote growth, the weirs are considered to have fulfilled their function and are expendable when this occurs.

Figure 873.4A
Thalweg Redirection Using Bendway Weirs

Bendway weirs in conjunction with rock slope protection.

Bendway weirs are similar in appearance to stone spurs, but have significant functional differences. Spurs are typically visible above the flow line and are designed so that flow is either diverted around the structure, or flow along the bank line is reduced as it passes through the structure. Bendway weirs are normally not visible, especially at stages above low water, and are intended to redirect flow by utilizing weir hydraulics over the structure. Flow passing over the bendway weir is redirected such that it flows perpendicular to the axis of the weir and is directed towards the channel centerline. See Figure 873.4B for typical cross section and layout. Similar to stone spurs, bendway weirs reduce near bank velocities, reduce the concentration of currents on the outer bank, and can produce a better alignment of flow through the bend and downstream crossing. Experience with bendway weirs has indicated that the structures do not perform well in degrading or sediment deficient reaches.

Material sizing should be based on the Isbash equation plotted in Figure 873.4C. Riprap stone size is designed using the critical velocity near the boundary where the riprap is placed. Typically the size ranges between 1 and 3 ft and should be approximately 20% greater than that computed from the rock sizing formula presented in Index 873.3(3)(a)(2)(b). The minimum rock size should not be less than the D_{100} of the streambed material. See Tables 873.3A and 873.3B to determine rock class.

See HEC 23 Volume 2, Design Guideline 1 for detailed guidance on weir height, length, angle, location and spacing,

(b) Spurs. A spur can be a pervious or impervious structure projecting from the streambank into the channel. Similar to bendway weirs, spurs are used to halt meander migration at a bend and channelize wide, poorly defined streams into well-defined channels by reducing flow velocities in critical zones near the streambank to prevent erosion and establish a more desirable channel alignment or width. The main function of spurs is to reduce flow velocities near the bank, which in turn, encourages sediment deposition due to these reduced velocities. Increased protection of banks can be achieved over time, as more sediment is deposited behind the spurs. Because of this, spurs may protect a streambank more effectively and at less cost than revetments. Furthermore, by moving the location of any scour away from the bank, partial failure of the spur can often be repaired before damage is done to structures along and across the stream.

In braided streams, the use of spurs to establish and maintain a well-defined channel location, cross section, and alignment can decrease the required bridge length, thus decreasing the cost of bridge construction and maintenance.

Spur types are classified based upon their permeability as retarder spurs, retarder/deflector spurs, and deflector spurs. The permeability of spurs is defined simply as the percentage of the spur surface area facing the streamflow that is open. Deflector spurs are impermeable spurs which function by diverting the primary flow currents away from the bank. Retarder/deflector spurs are more permeable and function by retarding flow velocities at the bank and diverting flow away from the bank. Retarder spurs are highly permeable and function by retarding flow velocities near the bank.

These structures should be designed not to overtop. Therefore, for permeable spurs, the rock sizing formula presented in Index 873.3(3)(a)(2)(b) may be used and a Cv value of 1.25 is recommended. Where overtopping the spur is unavoidable, the riprap size may be determined by equations 5.2 (for slopes > 25%) or 5.3 (for slopes < 25%) in HEC 23 Volume 2, Design Guideline 5. Since these equations are for free flow down the slope, always check to see if the structure is actually drowned (submerged) by high tailwater. If that is the case, then use the rock sizing formula presented in Index 873.3(3)(a)(2)(b) for sizing riprap on a stream bank should be used. See Tables 873.3A and 873.3B to determine rock class.

In general a top width equal to the width of a dump truck can be used. The side slopes of the spur should be 2H:1V or flatter. Rock riprap should be placed on the upstream and downstream faces as well as on the nose of the spur to inhibit erosion of the spur.

Figure 873.4B

Bendway Weir Typical Cross Section and Layout

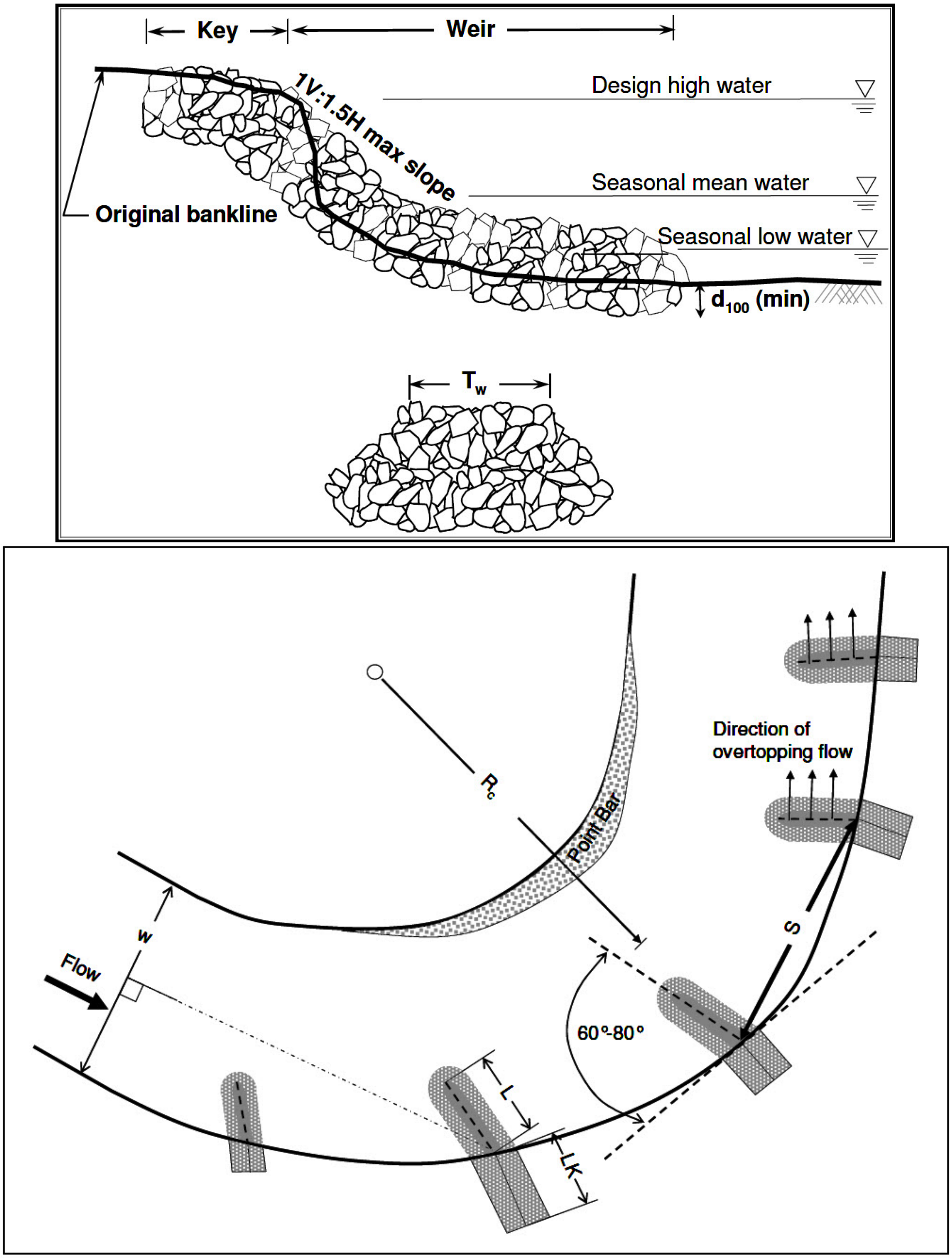
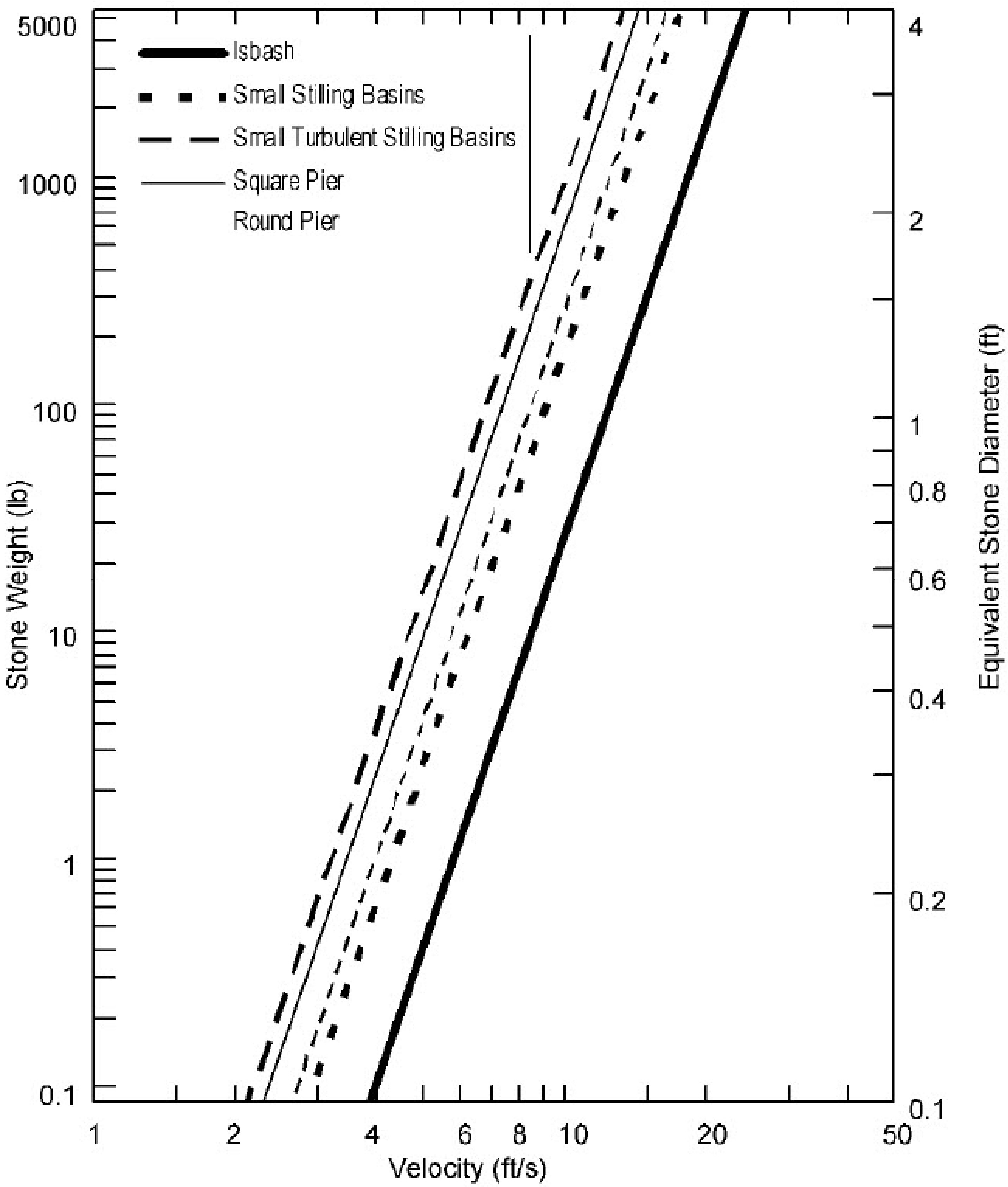


Figure 873.4C

Bendway Weir Rock Size Chart



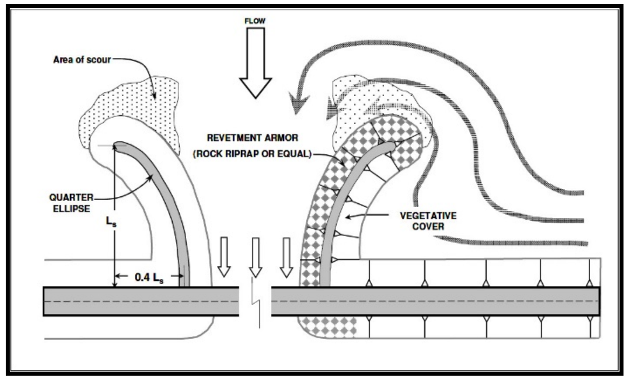
Depending on the embankment material being used, a gravel, sand, or geotextile filter may be required. It is recommended that riprap be extended below the bed elevation to the combined long-term degradation and contraction scour depth. Riprap should also extend to the crest of the spur, in cases where the spur would be submerged at design flow, or to 2 feet above the design flow, if the spur crest is higher than the design flow depth. Additional riprap should be placed around the nose of the spur, so that spur will be protected from scour.

See Figure 873.4D for example of spur design and HEC 23 Volume 2, Design Guideline 2, for detailed guidance on spur height, length, shape, angle, permeability, location and spacing.

(c) Guide Dikes/Banks. Guide banks are appendages to the highway embankment at bridge abutments, see Figure 873.4E. They are smooth extensions of the fill slope on the upstream side. When embankments encroach on wide floodplains to attain an economic length of bridge, the flows from these areas must flow parallel to the approach embankment to the bridge opening. These flows can cause a severe flow contraction at the abutment with damaging eddy currents that can scour away abutment and pier foundations, erode the approach embankment, and reduce the effective bridge opening.

Guide banks can be used in these cases to prevent erosion of the approach embankments by cutting off the flow adjacent to the embankment, guiding streamflow through a bridge opening, and transferring scour away from abutments to prevent damage caused by abutment scour. The two major enhancements guide banks bring to bridge design are (1) reduce the separation of flow at the upstream abutment face and thereby maximize the use of the total bridge waterway area, and (2) reduce the abutment scour due to lessening turbulence at the abutment face. Guide banks can be used on both sand and gravel-bed streams.

Figure 873.4E
Bridge Abutment Guide Banks



Guide banks are usually earthen embankment faced with rock slope protection. Optimum shape and length of guide dikes will be different for each site. Field experience has shown that an elliptical shape with a major to minor axis ratio of 2.5:1 is effective in reducing turbulence. The length is dependent on the ratio of flow diverted from the floodplain to flow in the first 100 feet of waterway under the bridge. If the use of another shape dike, such as a straight dike, is required for practical reasons more scour should be expected at the upstream end of the dike. The bridge end will generally not be immediately threatened should a failure occur at the upstream end of a guide dike.

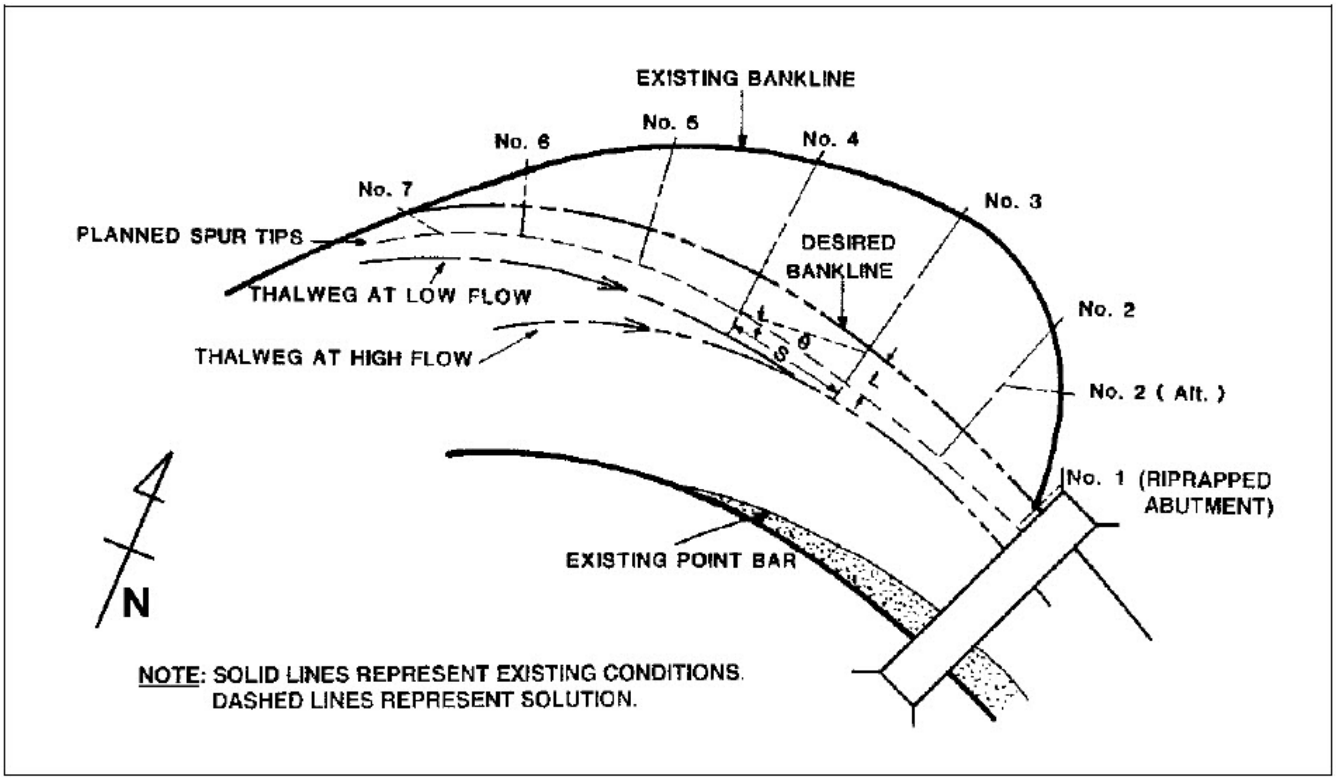
Toe dikes are sometimes needed downstream of the bridge end to guide flow away from the structure so that redistribution in the floodplain will not cause erosion damage to the embankment due to eddy currents. The shape of toe dikes is of less importance than it is with upstream guide banks.

Principal factors to be considered when designing guide banks, are their orientation to the bridge opening, plan shape, upstream and downstream length, cross-sectional shape, and crest elevation.

It is apparent from the Figure 873.4E that without this guide bank, overbank flows

Figure 873.4D

Example of Spur Design



would return to the channel at the bridge opening, which can increase the severity of contraction and scour at the abutment. With installation of guide banks the scour holes which normally would occur at the abutments of the bridge are moved upstream away from the abutments. Guide banks may be designed at each abutment, as shown, or singly, depending on the amount of overbank or floodplain flow directed to the bridge by each approach embankment.

The goal in the design of guide banks is to provide a smooth transition and contraction of the streamflow through the bridge opening. Ideally, the flow lines through the bridge opening should be straight and parallel. As in the case with other countermeasures, the designer should consider the principles of river hydraulics and morphology, and exercise sound engineering judgment.

The Division of Engineering Services (DES) and Structures Maintenance and Investigations (SMI) Hydraulics Branches are responsible for the hydraulic design of bridges, therefore, for protection at bridge abutments and approaches, the District is responsible for consulting with them to verify the design parameters and also obtaining the bridge hydraulic model. See Index 873.6 “Coordination with the Division of Engineering Services and Structures Maintenance and Investigations.”

For further detailed information on guide bank design procedures, refer to HEC 23, Volume 2, Design Guidelines 14 and 15. See Tables 873.3A and 873.3B to determine rock class.

(d) Further Information and Other Countermeasures for Lateral Stream Instability. General design considerations and guidance for evaluating scour and stream stability at highway bridges is contained in HEC 18, HEC 20, and HEC 23.

For further information on other countermeasures such as retarder structures, longitudinal dikes and bulkheads, see HEC 23 Volume 1, Chapter 8.

(e) Check Dams and Drop Structures. Drop structures or check dams are an effective means of gradient control. They may be constructed of rock, gabions, concrete, treated timber, sheet piling or combinations of any of the above. They are most suited to locations where bed materials are relatively impervious otherwise underflow must be prevented by cutoffs. Rock riprap and timber pile construction have been most successful on channels having small drops and widths less than 100 ft. Sheet piles, gabions, and concrete structures are generally used for larger drops on channels with widths ranging up to 300 ft. Check dams can initiate erosion of banks and the channel bed downstream of the structure as a result of energy dissipation and turbulence at the drop. This local scour can undermine the check dam and cause failure. The use of energy dissipators downstream of check dams can reduce the energy available to erode the channel bed and banks. In some cases it may be better to construct several consecutive drops of shorter height to minimize erosion. Lateral erosion of channel banks just downstream of drop structures is another adverse result of check dams and is caused by turbulence produced by energy dissipation at the drop, bank slumping from local channel bed erosion, or eddy action at the banks. The usual solution to these problems is to place rock slope protection on the streambank adjacent to the drop structure or check dam. Erosion of the streambed can also be reduced by placing rock riprap in a preformed scour hole downstream of the drop structure. A row of sheet piling with top set at or below streambed elevation can keep the riprap from moving downstream. Because of the problems associated with check dams, the design of these countermeasures requires designing the check dams to resist scour by providing for dissipation of excess energy and protection of areas of the bed and the bank which are susceptible to erosive forces. Refer to HEC 23 Volume 2, Design

Guideline 3 and HDS No. 6, Section 6.4.11, for further discussion on the use of check dams and drop structures.

873.5 Summary and Design Check List

The designer should anticipate the more significant problems that are likely to occur during the construction and maintenance of channel protection facilities. So far as possible, the design should be adjusted to eliminate or minimize those potential problems.

The logistics of the construction activity such as access to the site, on-site storage of construction materials, time of year restrictions, environmental concerns, project specific permits and sequence of construction should be carefully considered during the project design. See Index 872.1, Planning, Index 872.3(6), Construction, Easements, Access and Staging, and Index 872.3(7), Biodiversity. The stream morphology and its response to construction activities is an integral part of the planning process. Communication between the designer and those responsible for construction administration as well as maintenance are important.

Channel protection facilities require periodic maintenance inspection and repair. Where practicable, provisions should be made in the facility design to provide access for inspection and maintenance.

The following check list has been prepared for both the designer and reviewer. It will help assure that all necessary information is included in the plans and specifications. It is a comprehensive list for all types of protection. Items pertinent to any particular type can be selected readily and the rest ignored.

1. Location and staging of the planned work with respect to:
   - The highway.
   - The stream, its morphology, biodiversity and project specific permits.
   - Right of way. See Index 872.1 and 872.3 for construction easements and examination of stream behavior far upstream and downstream.
2. Datum control of the work, and relation of that datum to gage datum on streams.
3. A typical cross section indicating dimensions, slopes, arrangement and connections.
4. Quantity of materials (per foot, per protection unit, or per job).
5. Relation of the foundation treatment with respect to the existing ground.
6. Relation of the top of the proposed protection to design high water (historic, with date; or predicted, with frequency).
7. The limits of excavation and backfill as they may affect measurement and payment.
8. Construction details such as weep holes, rock slope protection fabrics, geocomposite drains and associated materials.
9. Location and details of construction joints, cut-off stubs and end returns.
10. Restrictions to the placement of reinforcement.
11. Connections and bracing for framing of timber or steel.
12. Splicing details for timber, pipe, rails and structural shapes.
13. Anchorage details, particularly size, type, location, and method of connection.
14. Size, shape, and special requirements of units such as precast concrete shapes and other manufactured items.
15. Number and arrangement of cables and details of fastening devices.
16. Size, mass per unit area, mesh spacing and fastening details for wire-fabric or geosynthetic materials.
17. On timber pile construction the number of piles per bent, number of bents, length of piling, driving requirements, cut-off elevations, and framing details.
18. The details of gabions and the filling material. See Standard Plan D100A and D100B and the Standard Specifications.
19. The size of articulated blocks, the placement of steel, and construction details relating to fabrication.
20. The corrosion considerations that may dictate specialty concretes, coated reinforcing, or other special requirements.

873.6 Coordination with the Division of Engineering Services and Structures Maintenance and Investigations

(1) *The Division of Engineering Services and Structures Maintenance and Investigations Hydraulics Branches.* The Division of Engineering Services (DES) and Structures Maintenance and Investigations (SMI) Hydraulics Branches are responsible for the hydraulic design of bridges. Therefore, for protection at bridge piers, abutments and approaches, the District is responsible for consulting with them to verify the design parameters (i.e., water surface elevations, freeboard requirements, water velocities, scour recommendations etc.) used and also obtaining the bridge hydraulic model.

Figure 873.6A
Bridge Abutment Failure Example

Bridge Abutment Failure at Tex Wash on I-10 after a Flood Larger Than the Design Flood

The DES Hydraulics branch performs all hydraulic designs for new bridges or replacement bridges that meet the National Bridge Inventory (NBI) bridge definition. Modifications to an existing bridge or constructing a new bridge require obtaining permits from the regulatory agencies. The DES Hydraulics branch should coordinate with the District to perform conceptual designs for permit approval. The DES Hydraulics branch is essentially a consultant/designer to the District Design Offices.

The SMI Hydraulics branch within the Division of Maintenance is responsible for the hydraulic analyses, repair and monitoring of in-service bridges. Typical maintenance challenges include scour, flooding, and lateral migration. Maintenance related impacts to a bridge will trigger a hydraulic report for that specific bridge. The hydraulic report recommendations are used by the District in determining the scope of hydraulic improvements to the bridge projects. For countermeasure design at bridge abutments and piers (e.g., rock slope protection, guide banks, check dams, structural repairs etc.) the magnitude of the discharge used is the 100-year flood. This standard is independent of the design flood used by the District for protecting the channel bank or the bridge approach embankment (see Index 873.2).

Since the mid 1990's, new bridges have been designed so that the top of the pile cap is at the bottom of anticipated scour (long-term degradation, contraction and local scour) for the 100-year flood using the hypothesis that by designing the foundations lower than recommended in HEC 18 for the 100-year flood, there would be ample safety factor inherent to withstand the 200-year scour check flood. Bridges that were designed prior to the first edition of HEC 18 in 1991 may be more vulnerable to the possible effects of climate change or floods larger than the 100-year flood. See Figure 873.6A.

Depending on location, site considerations may include constructability and biodiversity, see Index 872.3(4) and Index 872.3(5). During the planning and environmental phases on environmentally sensitive projects (e.g., bridge structures that require permits for fish passage design under California Fish and Wildlife jurisdiction – see Figure 873.3B), the District should initiate contact with resource agencies early to propose conceptual design, identify impacts and any necessary mitigation as part of the permitting process. The overriding issue of concern is the difference in timing of detailed analyses (e.g., hydraulics, geotechnical, foundation) that takes place on the District side of the project development process verses what takes place in DES prior to project approval during the environmental phase.

Prematurely approved projects and environmental documents prior to permit approval can, and do, result in costly major re-work during the design phase. On environmentally sensitive projects the District should consider the need to shift resources to the environmental phase so that a more advanced bridge foundation design can be incorporated into the Advanced Planning Study (APS) and the Environmental Document (ED) to facilitate permit approval consistent with the Project Approval (PA) and to minimize rework.

Figure 873.6B
Habitat Enhancement Example

Longitudinal Peaked Stone Toe Protection (LPSTP) with Rock Vanes for Chinook Salmon Habitat Enhancement, Route 128, Russian River Bridge in Geyserville

(2) *Geotechnical Design and Geology.* The Project Engineer must review the Project Initiation Document and Preliminary Geotechnical Design Report, if any, to ascertain the scope of geotechnical involvement for a project.

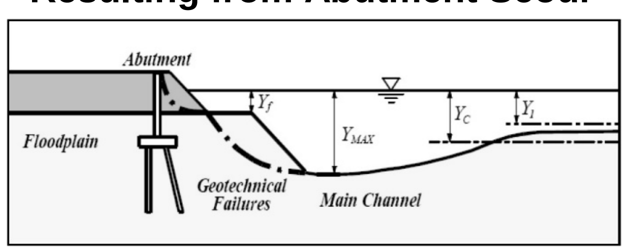
For all projects that involve designs for cut slopes, embankments, earthwork, landslide remediation, retaining walls, groundwater studies, erosion control features, subexcavation and any other studies involving geotechnical investigations and engineering geology, a Geotechnical Design Report (GDR) is to be prepared by the Roadway Geotechnical Engineering Branches of the Division of Engineering Services, Geotechnical Services (DES-GS).

Coordination with Geotechnical Design and Geology within DES may be initiated by the designer when any of the following determinations need to be made:

- Scour potential of channel material.
- Natural erosion potential of stream banks that may affect project features. See Figure 873.6C.
- The performance of existing cut, fill and natural slopes including the slope soil/rock composition.
- Slope stability analysis and need for earth retaining systems including crib walls and gabion walls.
- Embankment constructability and impact to nearby structures or bridge abutments. See following link to the Geotechnical Manual and Figure 873.6D: http://www.dot.ca.gov/hq/esc/geotech/geo_manual/page/Embankments_Dec2014.pdf

Figure 873.6C
Lateral Stream Migration Within a Canyon Setting Example

Figure 873.6D
Conceptual Geotechnical Failures Resulting from Abutment Scour



CHAPTER 880
SHORE PROTECTION

Topic 881 - General

Index 881.1 - Introduction

Highways, bikeways, pedestrian facilities and appurtenant installations are often attracted to parallel locations along lakes and coastal zones. These locations are under attack from the action of waves and may require protective measures.

Shore protection along coastal zones and lake shores that are subjected to wave attack can be a major element in the design, construction, and maintenance of highways. Chapter 880 deals with procedures, methods, devices, and materials commonly used to mitigate the damaging effects of wave action on transportation facilities and adjacent properties. The primary focus is on quantifying exposure of these locations to sea level rise, storm surge, and wave action. The practice of coastal engineering is still much of an art. This is for a variety of reasons including that the physical processes are so complex, often too complex for adequate theoretical description, and the design level of risk is often high.

Refer to Index 806.2 for definitions of drainage terms.

881.2 Design Philosophy

In each district there should be a designer or advisor, usually the District Hydraulic Engineer, knowledgeable in the application of shore protection principles and the performance of existing works at coastal and lake shore locations vulnerable to wave attack.

Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from involvement with Design, Environmental, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802). For habitat characterization and assessment relative to design and obtaining project specific permits, the designer may also require input from biologists. The District Hydraulic Engineer will typically be able to assist with selecting storm scenarios for design wave heights, the design of high water level (including sea or lake level change estimates) using coastal surge and wave models, flood analysis, water surface elevations/profiles, shear stress computations, scour analysis and hydraulic analysis for placement of coastal structures.

There are a number of ways to deal with the problem of wave action and shore erosion.

- Where avoidance is not feasible, the simplest way and generally the surest of success and permanence, is to locate the facility away from the erosive forces. This is not always feasible or economical, but should be the first consideration. Locating the facility to higher ground or solid support should never be overlooked, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for protection at other points of attack.
- The most commonly used method is to armor the shore with a more resistant material like rock slope protection. FHWA Hydraulic Engineering Circular 25 (HEC 25), Volume 1, presents general issues and approaches in coastal highway design. Types of revetments for wave attack and coastal structures are covered in Indexes 6.1 and 7.6.
- *Rock Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity of stone is a major factor in shore protection and the specified minimum should not be lowered without increasing the mass of stones. See Index 873.3(3)(a)(2)(b) for equations to estimate rock size.

881.3 Selected References

Hydraulic and drainage related publications are listed by source under Topic 807. References specifically related to shore protection measures are listed here for convenience.

(a) FHWA Hydraulic Engineering Circulars (HEC) -- The following circulars were developed to assist the designer in using various types of slope protection and channel linings:

- HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006)
- HEC 18, Evaluating Scour at Bridges (2012)
- HEC 20, Stream Stability at Highway Structures (2012)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
- HEC 25, Highways in the Coastal Environment (2008 with 2014 supplement – Assessing Extreme Events)

(c) AASHTO Highway Drainage Guidelines -- General guidelines for good erosion control practices are covered in Volume III - Erosion and Sediment Control in Highway Construction, and Volume XI - Guidelines for Highways Along Coastal Zones and Lakeshores.

(d) AASHTO Drainage Manual (2014) – Refer to Chapters; 11 – Energy Dissipators; 16 – Erosion and Sediment Control; 17 – Bank Protection; and 18 – Coastal Zone. The MDM provides guidance on engineering practice in conformance with FHWA's HEC and HDS publications and other nationally recognized engineering policy and procedural documents.

(e) Shore Protection Manual (SPM) (1984) – Comprehensive two volume guidance on wave and shore processes and methods for shore protection. No longer in publication but still referenced pending completion of the Coastal Engineering Manual.

(f) Design of Coastal Revetments, Seawalls, and Bulkheads. Engineering Manual 1110-2-1614 (1995) – Supersedes portions of Volume 2 of the Shore Protection Manual (SPM).

(g) Coastal Engineering Manual. Engineer Manual (EM) 1110-2-1100 (2002) – Published in six parts plus an appendix, this set of documents supersedes the SPM and EM 1110-2-1614.

Topic 882 - Planning and Location Studies

882.1 Planning

The development of sustainable, cost effective and environmentally friendly protective works requires careful planning and a good understanding of both the site location and habitat within the shore or coastal zone subject to wave attack. Planning begins with an office review followed by a site investigation.

Google Earth can be a useful tool for determining site location and recent changes to the coastal zone.

Nearby bridges should be reviewed for site history and changes in stream cross-section. All bridge files belong to Structure Maintenance within the Division of Maintenance.

Coastal highways traverse bays, estuaries, beaches, dunes and bluffs which are some of the most unique and treasured habitats for humans as well as the habitats of a variety of plants and animals. The list of endangered species requiring these coastal habitats for survival includes numerous sea turtles, birds, mammals, rodents, amphibians and fish. District biologist staff should be consulted early on during the project planning phase for subject matter expertise to perform an initial habitat assessment. Contact information for Department biologists can be accessed through the CalBioRoster.

For habitat characterization and preliminary assessment relative to design and obtaining project specific permits, the initial site investigation team should include the project engineer, the district hydraulic engineer, and a biologist.

The selection of the type of protection can be determined during or following site investigation. For some sites the choice is obvious; at other sites several alternatives or combinations may be applicable.

Considerations at this stage are:

- Design life and whether the protection need be permanent or temporary.
- The severity of wave attack.
- The coastal water level and future sea level.

- Littoral drift of the beach sands.
- Seasonal shifts of the shore.
- The ratio of cost of highway replacement versus cost of protection.
- Analysis of foundation and materials explorations.
- Access for construction
- Slope (H:V)
- Vegetation type and location
- Physical habitat
- Failure mode (see Table 872.2)
- Total length of protection needed

The second step is the selection and layout of protective elements in relation to the highway facility.

882.2 Class and Type of Protection

Protective devices are classified according to their function. They are further categorized as to the type of material from which they are constructed or shape of the device.

882.3 Site Consideration

The determination of the lengths, heights, alignment, and positioning of the protection is affected to a large extent by the facility location environment.

An evaluation is required for any proposed highway construction or improvement that encroaches on a floodplain. See Topic 804, Floodplain Encroachments for detailed procedures and guidelines.

(1) *Lakes and Tidal Basins.* Highways adjacent to lakes or basins may be at risk from wave generated erosion. All bodies of water generate waves. Height of waves is a function of fetch and depth. Erosion along embankments behind shallow coves is reduced because the higher waves break upon reaching a shoal in shallow water. The threat of erosion in deep water at headlands or along causeways is increased. Constant exposure to even the rippling of tiny waves may cause severe erosion of some soils.

Older lakes normally have thick beds of precipitated silt and organic matter. Bank protection along or across such lakes must be designed to suit the available foundation. It is usually more practical to use lightweight or self-adjusting armor types supported by the soft bed materials than to excavate the mud to stiffer underlying soils. See Index 883.3 for further information on armor protection.

In fresh waters, effective protection can often be provided by the establishment of vegetation, but planners should not overlook the possibility of moderate erosion before the vegetative cover becomes established. A light armor treatment should be adequate for this transitional period.

(2) *Ocean Front Locations.* Wave action is the erosive force affecting the reliability of highway locations along the coast. The corrosive effect of salt water is also a major concern for hydraulic structures located along the coastline. Headlands and rocks that have historically withstood the relentless pounding of tide and waves can usually be relied on to continue to protect adjacent highway locations founded upon them. The need for shore protection structures is, therefore, generally limited to highway locations along the top or bottom of bluffs having a history of sloughing and along beach fronts.

Beach protection considerations include:

- Attack by waves.
- Littoral drift of the beach sands.
- Seasonal shifts of the shore.
- Foundation for protective structures.

Wave attack on a beach is less severe than on a headland, due to the gradual shoaling of the bed which trips incoming waves into a series of breakers called a surf.

Littoral drift of beach sands may either be an asset or a liability. If sand is plentiful, a new beach will be built in front of the highway embankment, reducing the depth of water at its toe and the corresponding height of the waves attacking it. If sand supply is less plentiful or subject to seasonal variations, the new beach can be induced or retained by groins.

If sand is in scant supply, backwash from a revetment tends to degrade the beach or bed even more than the seasonal variation, and an allowance should be made for this scour when designing the revetment, both as to weight of stones and depth of foundation. Groins may be ineffective for such locations; if they succeeded in trapping some littoral drift, downcoast beaches would recede from undernourishment.

Seasonal shifts of the shore line result from combinations of:

- Ranges of tide.
- Reversal of littoral currents.
- Changed direction of prevailing onshore winds.
- Attack by swell.

Generally the shift is a recession, increasing the exposure of beach locations to the hazard of damage by wave action. On strands or along extensive embayments, recession at one end may result in deposition at the other. Observations made during location assessment should include investigation of this phenomenon. For strands, the hazard may be avoided by locating the highway on the backshore facing the lagoon.

Foundation conditions vary widely for beach locations. On a receding shore, good bearing may be found on soft but substantial rock underlying a thin mantle of sand. Bed stones and even gravity walls have been founded successfully on such foundations. Spits and strands, however, are radically different, often with softer clays or organic materials underlying the sand. Sand is usually plentiful at such locations, subsidence is a greater hazard than scour, and location should anticipate a "floating" foundation for flexible, self-adjusting types of protection.

In planning ocean-front locations, the primary decision is a choice of (1) alignment far enough inshore to avoid wave attack, (2) armor on the embankment face, or (3) off shore devices like groins to aggrade the beach at embankment toe.

Topic 883 - Design

883.1 Introduction

A set of plans and specifications must be prepared to define and describe the protection that the design engineer has in mind. See Index 873.1.

Recommendations on slope protection, and erosion control materials can be requested from the District Hydraulic Engineer, the District Materials Branch and the Office of State Highway Drainage and Water Quality Design in Headquarters. The District Landscape Architect will provide recommendations for temporary and permanent erosion and sediment control measures.

The Caltrans Bank and Shore Protection Committee is available on request to provide advice on extraordinary situations or problems and to provide evaluation and formal approvals for acceptable non-standard designs. See Index 802.3 for further information on the organization and functions of the Committee.

883.2 Design High Water and Design Wave Height

Information needed to design shore protection is:

- Design High Water Level
- Design Wave Height

(1) Design High Water Level

Designs should not be based on an arbitrary storm, high tide or flood frequency.

Per Index 873.2, a suggested starting point of reference for the determination of the design high water level is that the protection withstands high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. Depending on the type of facility, it may be appropriate to base the preliminary evaluation on a high water elevation resulting from a storm or flood with a 2 percent probability of exceedance (50 year frequency of recurrence). The first evaluation may have to be adjusted to conform with a subsequent analysis which considers the level of related risks, local historic high water marks, sea level rise and climate change. Scour countermeasures protecting structures designed by the Division of

Engineering Services (DES) may include consideration of floods greater than a 1 percent probability of exceedance (100 year frequency of recurrence). See Index 873.6.

There is always some risk associated with the design of protection features. Significant risks are classified as those having probability of:

- Catastrophic failure with loss of life.
- Disruption of fire and ambulance services or closing of the only evacuation route available to a community.

Refer to Topic 804, Floodplain Encroachments, for further discussion on evaluation of risks and impacts.

(a) Lake Shore Locations. The flood stage elevation on a lake or reservoir is usually the result of inflow from upland runoff. If the water stored in a reservoir is used for power generation, flood control, or irrigation, the design high water elevation should be based on the owner's schedule of operation.

(b) Coastal Locations.

Except for inland tidal basins effected by wind tides, floods and seiches, the static or still-water level used for design of shore protection is the highest tide. In tide tables, this is the stage of the highest tide above "tide-table datum" at MLLW. To convert this to MSL datum there must be subtracted a datum equation (2.5 feet to 3.9 feet) factor. If datum differs from MSL datum, a further correction is necessary. These steps should be undertaken with care and independently checked. Common errors are:

- Ignoring the datum equation.
- Adding the factor instead of subtracting it.
- Using half the diurnal range as the stage of high water.

To clarify the determination of design high-water, Fig. 883.2A shows the *Highest Tide* in its relation to an extreme-tide cycle and to a hypothetical average-tide cycle, together with nomenclature pertinent to three definitions of tidal range. Note that the cycles have two highs and two lows. The average of all the higher highs for a long period (preferably in multiples of the 19-yr. metonic cycle) is MHHW, and of all the *lower* lows, MLLW. The vertical difference between them is the *diurnal range*.

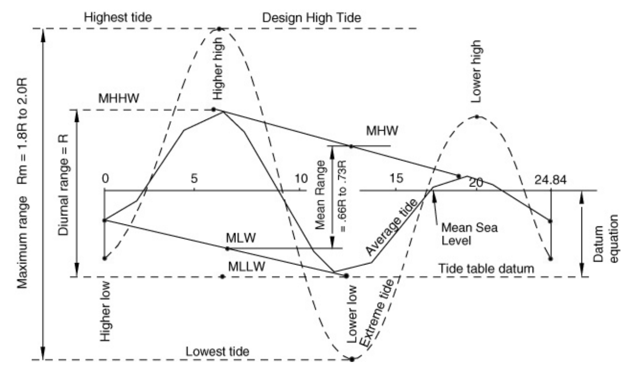
Particularly on the Pacific coast where MLLW is datum for tide tables, the stage of MHHW is numerically equal to diurnal range.

The average of all highs (indicated graphically as the mean of higher high and lower high) is the MHW, and of all the lows, MLW. Vertical difference between these two stages is the *mean range*.

See Index 814.5, Tides and Waves, for information on where tide and wave data may be obtained. See HEC 25, Volume 1, for a discussion on tidal and survey datums.

Figure 883.2A

Nomenclature of Tidal Ranges



NOTES:

(1) Because of the great variation of tidal elements, Figure 883.2A was not drawn to scale.

(2) The elevation of the design high tide may be taken as mean sea level (MSL) plus one-half the maximum tidal range (Rm).

(2) Design Wave Heights.

(a) General. Even for the simplest of cases, the estimation of water levels caused by meteorological conditions is complex. Elaborate numerical models requiring the use of a computer are available. See HEC 25, Volume 2, Index 2.4.2. Simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of embayments, inland lakes, and reservoirs. The Coastal Engineering Manual provides a simplified wave prediction method which is suitable for most riprap sizing applications. The method is described in HEC 23, Volume 2, Index 17.2.2 of Design Guideline 17. It is recommended that for ocean shore protection designs the assistance of the U.S. Army Corp of Engineers be requested.

Shore protection structures are generally designed to withstand the wave that induces the highest forces on the structure over its economic service life. The design wave is analogous to the design storm considerations for determining return frequency. A starting point of reference for shore protection design is the maximum significant wave height that can occur once in about 20-years. Economic and risk considerations involved in selecting the design wave for a specific project are basically the same as those used in the analysis of other highway drainage structures.

(b) Wave Distribution Predictions. Wave prediction is called hindcasting when based on past meteorological conditions and forecasting when based on predicted conditions. The same procedures are used for hindcasting and forecasting. The only difference is the source of the meteorological data. Reference is made to the Army Corps of Engineers, Coastal Engineering Manual – Part II, for more complete information on the theory of wave generation and predicting techniques.

The prediction of wave heights from boat generated waves must be estimated from observations.

The surface of any large body of water will contain many waves differing in height, period, and direction of propagation. A representative wave height used in the design of bank and shore protection is the significant wave height, H_S. The significant wave height is the average height of the highest one-third of all the waves in a wave train for the time interval (return frequency) under consideration. Thus, the design wave height generally used is the significant wave height, H_s, for a 20-year return period.

Other design wave heights can also be designated, such as H_{10} and H_1. The H_{10} design wave is the average of the highest 10 percent of all waves, and the H_1 design wave is the average of the highest 1 percent of all waves. The relationship of H_{10} and H_1 to H_s can be approximated as follows:

$H_{10} = 1.27\ H_S$ and $H_1 = 1.67\ H_S$

Economics and risk of catastrophic failure are the primary considerations in designating the design wave average height.

(c) Wave Characteristics. Wave height estimates are based on wave characteristics that may be derived from an analysis of the following data:

- Wave gage records
- Visual observations
- Published wave hindcasts
- Wave forecasts
- Maximum breaking wave at the site

(d) Predicting Wind Generated Waves. The height of wind generated waves is a function of fetch length, windspeed, wind duration, and the depth of the water.

(1) Hindcasting -- The U.S. Army Corp of Engineers has historical records of onshore and offshore weather and wave observations for most of the California coastline. Design wave height predictions for coastal shore protection facilities should be made using this information and hindcasting methods. Deep-water ocean wave characteristics derived from offshore data analysis may need to be transformed to the project site by refraction and diffraction techniques. As mentioned previously, it is strongly advised that the Corps technical expertise be obtained so that the data are properly interpreted and used.

(2) Forecasting -- Simplified wind wave prediction techniques may be used to establish probable wave conditions for the design of highway protection on bays, lakes and other inland bodies of water. Wind data for use in determining design wind velocities and durations is usually available from weather stations, airports, and major dams and reservoirs.

The following assumptions pertain to these simplified methods:

- The fetch is short, 75 miles or less
- The wind is uniform and constant over the fetch.

It should be recognized that these conditions are rarely met and wind fields are not usually estimated accurately. The designer should therefore not assume that the results are more accurate than warranted by the accuracy of the input and simplicity of the method. Good, unbiased estimates of all wind generated wave parameters should be sought and the cumulative results conservatively interpreted. The individual input parameters should not each be estimated conservatively, since this may bias the result.

The applicability of a wave forecasting method depends on the available wind data, water depth, and overland topography. Water depth affects wave generation and for a given set of wind and fetch conditions, wave heights will be smaller and wave periods shorter if the wave generation takes place in transitional or shallow water rather than in deep water.

The height of wind generated waves may also be fetch-limited or duration-limited. Selection of an appropriate design wave may require a maximization procedure considering depth of water, wind direction, wind duration, wind speed, and fetch length.

Procedures for predicting wind generated waves are complex and our understanding and ability to describe wave phenomena, especially in the region of the coastal zone, is limited. Many aspects of physics and fluid mechanics of wave energy have only minor influence on the design of shore protection for highway purposes. Designers interested in a more complete discussion on the rudiments of wave mechanics should consult the U.S. Army Corps of Engineers' Coastal Engineering Manual – Part II.

An initial estimate of wind generated significant wave heights can be made by using Figure 883.2B. If the estimated wave height from the nomogram is greater than 2 feet, the procedure may need to be refined. It is recommended that advice from the Army Corps of Engineers be obtained to refine significant wave heights, Hs, greater than 2 feet.

(e) Breaking Waves. Wave heights derived from hindcasts or any forecasting method should be checked against the maximum breaking wave that the design stillwater level depth and nearshore bottom slope can support. The design wave height will be the smaller of either the maximum breaker height or the forecasted or hindcasted wave height.

The relationship of the maximum height of breaker which will expend its energy upon the protection, Hb, and the depth of water at the slope protection, ds, which the wave must pass over are illustrated in Figure 883.2C.

The following diagram, with some specific references to the SPM, summarizes an overly simplified procedure that may be used for highway purposes to estimate wind generated waves and establish a design wave height for shore protection.

(f) Wave Run-up. Run-up is the extent, measured vertically, that an incoming wave will rise on a structure. An estimate of wave run-up, in addition to design wave height, will typically be needed and is required by

Figure 883.2B

Significant Wave Height Prediction Nomograph

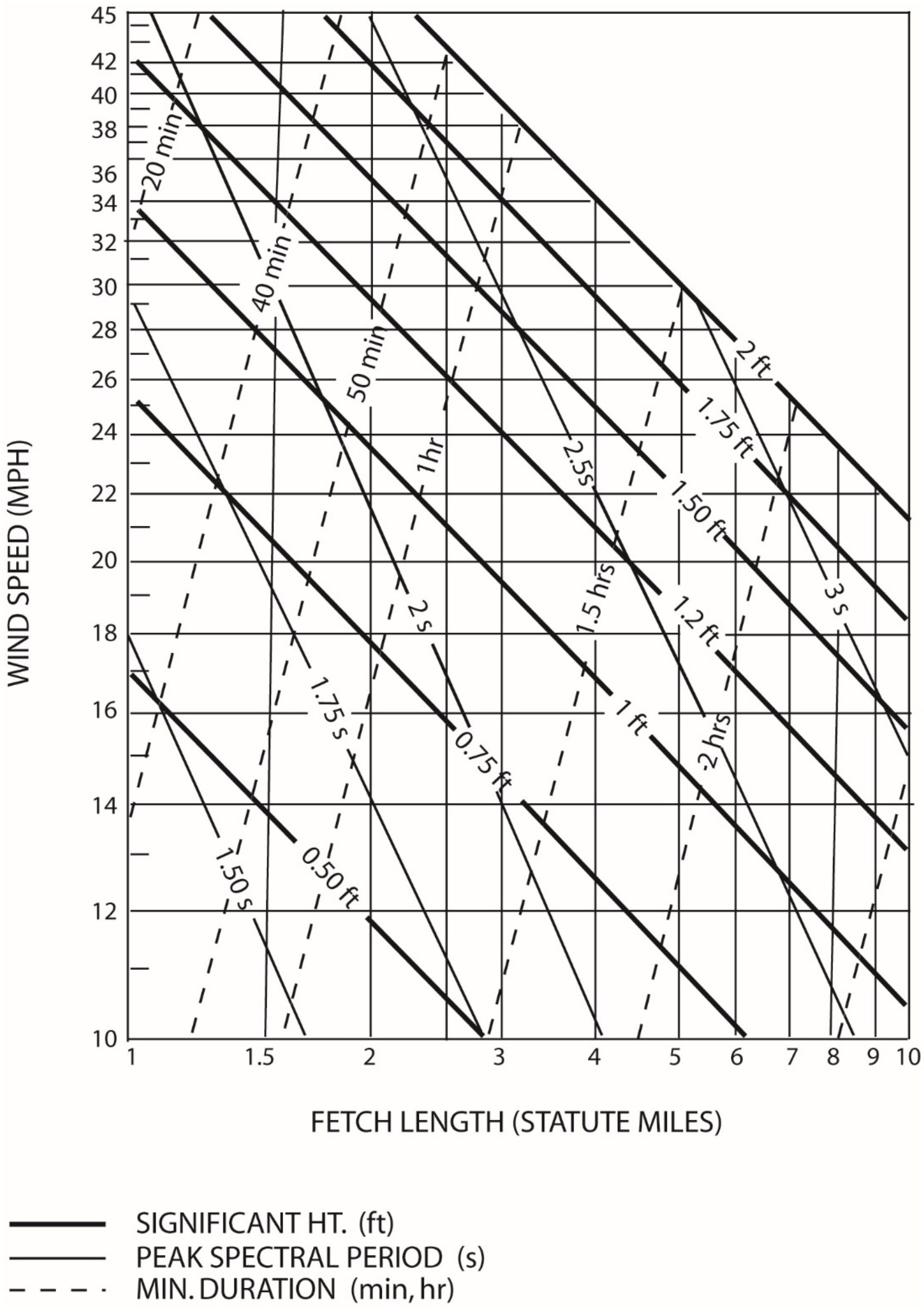
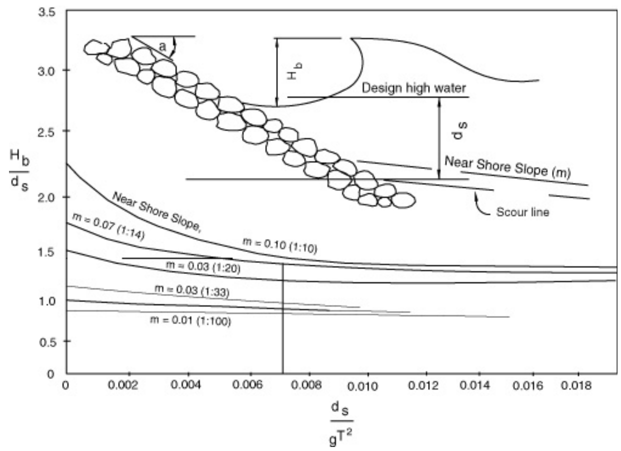


Figure 883.2C

Design Breaker Wave



Example:

By using hindcast methods, the significant wave height (H_s) has been estimated at 4 feet with a 3 second period. Find the design wave height (H_d) for the slope protection if the depth of water (d) is only 2 feet and the nearshore slope (m) is 1:10.

Solution:

$$\frac{d_s}{gT^2} = \frac{2\ ft}{(32.2\ ft/sec^2) \times (3\ sec)^2} = 0.007$$

From Graph) - $H_b/d_s = 1.4$

$H_b = 2 \times 1.4 = 2.8$ ft

Answer:

Since the maximum breaker wave height, H_b, is smaller than the significant deepwater wave height, H_s, the design wave height H_d is 2.8 feet.

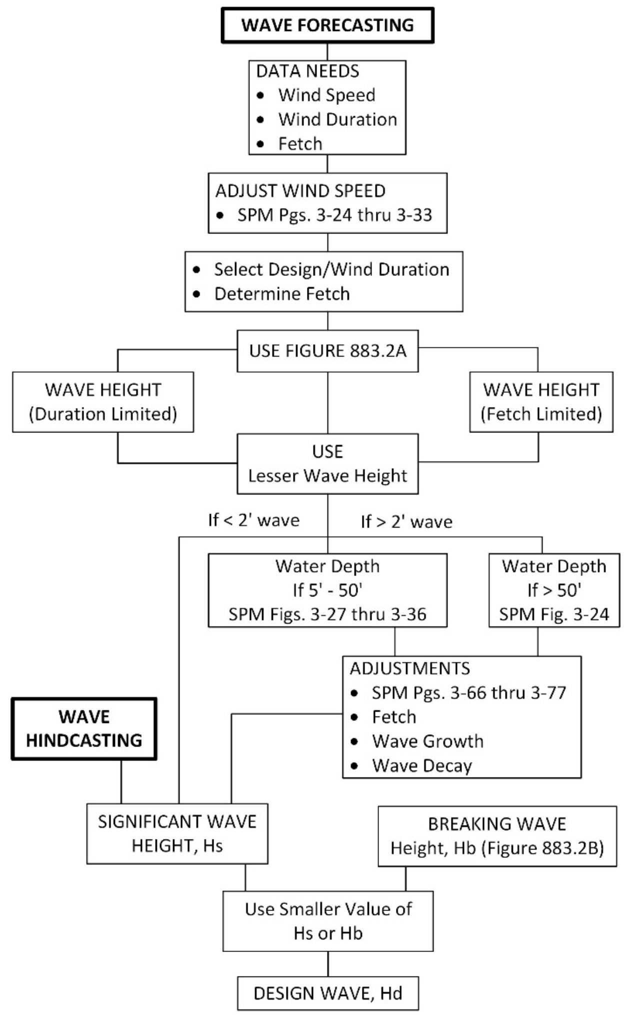
T = Wave Period (SPM)

policy for projects subject to California Coastal Commission (CCC) jurisdiction (see CCC guidance document "Beach Erosion and Response," December 1999). Procedures for estimating wave run-up for rough surfaces (e.g., RSP) are contained in the U.S. Army Corps of Engineers manual, Design of Coastal Revetments, Seawalls, and Bulkheads, (EM 1110-2-1614) published in 1995.

Procedures for estimating wave run-up for smooth surfaces (e.g., concrete paved slopes) and for vertical and curved face walls are contained in the U.S. Army Corps of Engineers, Shore Protection Manual, 1984. See Figure 873.2D for estimating wave run-up on smooth slopes for wave heights of 2 feet or less.

In protected bays and estuaries, waves generated by recreational or commercial boat traffic and other watercraft may dominate the design over wind generated waves. Direct observation and measurements during high tidal cycles may provide the designer the most useful tool for establishing wave run-up for these situations.

Determining Design Wave



(g) Littoral Processes. See Index 882.3(2). Littoral processes result from the interaction of winds, waves, currents, tides, and the availability of sediment. The rates at which sediment is supplied to and removed from the shore may cause excessive accretion or erosion that can affect the structural integrity of shore protection structures or functional usefulness of a beach. The aim of good shore protection design is to maintain a stable shoreline where the volume of sediment supplied to the shore balances that which is removed.

Designers interested in a more complete discussion on littoral processes should consult the U.S. Army Corps of Engineers' Coastal Engineering Manual (CEM) – Part III.

(3) *Sea Level Rise.* Sea levels have been rising not only in the past century, but for the past several millennia. The average rate of relative sea level change, which accounts for changes in both land and ocean elevation, varies with location. Tide and current data is available at the following NOAA web site: http://tidesandcurrents.noaa.gov

The National Research Council developed a report on sea level changes with specific commissioning from the three western states of California, Oregon, and Washington (NRC 2012). Their projection is for global eustatic (the rate with landmass elevation effects removed) sea level to rise 2.7 ft by the year 2100. The projected rate of rise is summarized in Figure 883.2D. A range of estimates is shown to account for the uncertainties in the best-available science. The range is from 1.7 ft to 4.6 ft by 2100.

The timeframe identified for a project is important for sea level rise (SLR) assessments and will affect the approach for assessing impacts. Until 2050, there is strong agreement among the various climate models for the amount of SLR that is likely to occur. After mid-century, projections of SLR are uncertain, because modeling results diverge and SLR projections vary depending upon how quickly the international community reduces greenhouse gas emissions. Therefore, for projects with timeframes beyond 2050, it is especially important to consider adaptive capacity, impacts, and risk tolerance to guide decisions of whether to use low, medium, or high SLR projections. Projects that have a long design life of 20+ years should include further SLR analysis. These projects have a high likelihood of being impacted by SLR at some point during their lifespan. The shorter lifespan projects may be less likely to face SLR impacts, and as a result be less inclined to incorporate SLR, depending on their proximity to the coast line.

HEC 25, Volume 2, Index 2.3.1.2 provides discussion on projections for SLR and and example calculation is presented in Index 2.3.1.3.

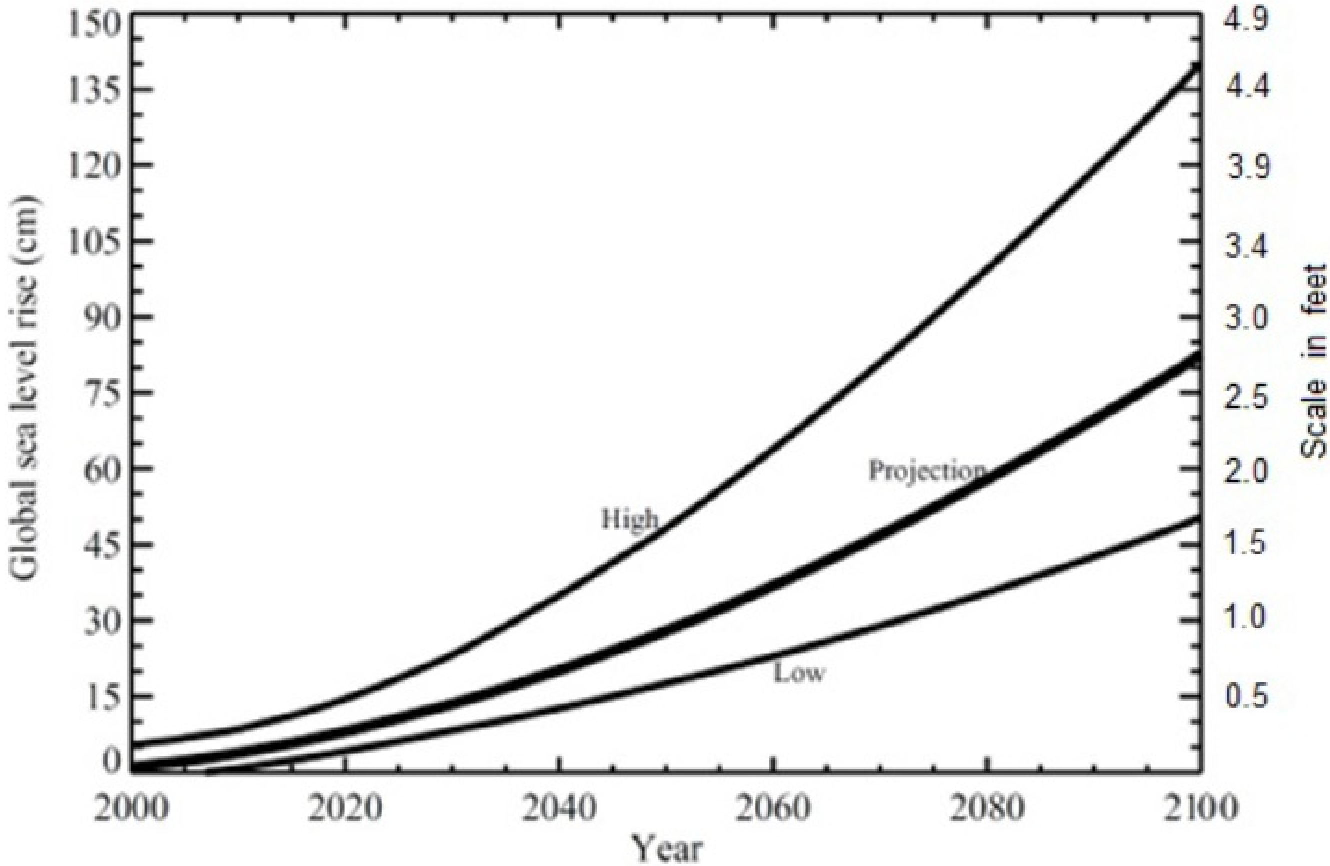
(4) *Assessing Extreme Events and Climate Change.* Chapter 4 of HEC 25, Volume 2 presents guidance on specific methodologies for assessing exposure of coastal transportation infrastructure to extreme events and climate change. For all projects, as a minimum, the use of existing data and resources should be utilized through the use of existing inundation (FEMA) or tsunami hazard maps to determine the exposure of infrastructure under selected sea (lake) level change scenarios, and sensitivity to depth-limited wave or wave runup processes. See HEC 25, Volume 2, Indexes 4.1.1 and 4.5.1 Level of Effort 1: Pacific Coast – Storms.

883.3 Armor Protection

(1) *General.* Armor is the artificial surfacing of shore or embankment to resist erosion or scour. Armor devices can be flexible (self-adjusting) or rigid. The distinction between revetments (layers of rock or concrete), seawalls, and bulkheads is one of functional purpose. Revetments usually consist of rock slope protection on the top of a sloped surface to protect the underlying soil. Seawalls are walls designed to protect against large wave forces. Bulkheads are designed primarily to retain the soil behind a vertical wall in locations with less wave action. Design issues such as tie-backs, depth of sheets are primarily controlled by geotechnical issues. The use of each one of the three types of coastal protection depends on the relationship between wave height and fetch (distance across the water body). Bulkheads are most common where fetches and wave heights are small. Seawalls are most common where fetches and wave heights are large. Revetments are often common in intermediate situations such as on bay or lake shorelines.

Figure 883.2D

Global Sea Level Rise Projections[1]



NOTES:

(1) Adapted from NRC 2012.

(2) Revetments.

(a) Rock Slope Protection (RSP). Hard armoring of shorelines, primarily with RSP, has been the most common means of providing long-term protection for transportation facilities, and most importantly, the traveling public. With many years of use, dozens of formal studies and thousands of constructed sites, RSP is the armor type for which there exists the most quantifiable data on performance, constructability, maintainability and durability, and for which there exist several nationally recognized design methods.

Due to the above factors, RSP is the general standard against which other forms of armoring are compared.

The results of internal research led to the publication of Report No. FHWA-CA-TL-95-10, "California Bank and Shore Rock Slope Protection Design". Within that report, the methodology for RSP design adopted as the Departmental standard for many years, was the California Bank and Shore, (CABS), layered design. The CABS layered design methodology and its associated gradations are now obsolete. For reference only, the full report is available at the following website:

http://www.dot.ca.gov/hq/oppd/hydrology/hydroidx.htm.

For RSP designs along coastal and lake shores, for wave heights five feet or less, the methodology presented in HEC 23, Volume 2, Design Guideline 17- Riprap Design for Wave Attack has been formally adopted by the Caltrans Bank and Shore Protection Committee. Section 72 of the Standard Specifications provides all construction and material specifications.

Rock is usually the most economical type of revetment where stones of sufficient size and quality are available. It also has the following advantages:

- Wave run-up is less than with smooth types (See Figure 883.2E).
- It is salvageable, may be stockpiled and reused if necessary.

Figure 883.2E

Wave Run-up on Smooth Impermeable Slope

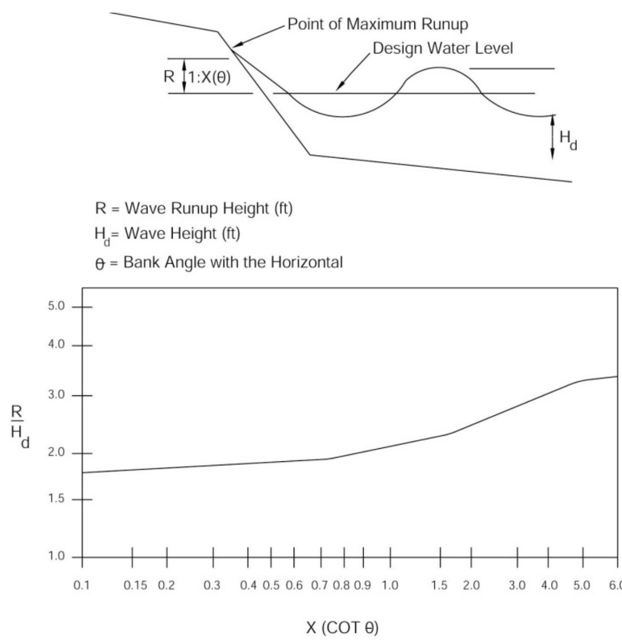


Figure 883.2F

RSP Lined Ocean Shore

In designing the rock slope protection for a shore location, the following determinations are to be made for the typical section.

- Depth at which the stones are founded (bottom of toe trench. See Figure 883.2G and Figure 17.2 in HEC 23, Volume 2, Design Guideline 17).

- Elevation at the top of protection.
- Rock size, specific gravity and section thickness.
- Need for geotextile or rock filter material.
- Face slope.

Well designed coastal rock slope protection should:

- Assure stability and compatibility of the protected shore as an integral part of the shoreline as a whole.
- Not be placed on a slope steeper than 1.5H:1V.
- Use stone of adequate weight to resist erosion, derived from Index 883.3(2)(a)(2)(1).
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric or a filter layer should be used.
- Rest on a good foundation on bedrock or extend below the depth of probable scour. If questionable, use heavy bed stones and provide a wide base section with a reserve of material to slough into local scour holes (i.e., mounded toe).
- Be constructed of rock of such shape as to form a stable protection structure of the required section. See Index 873.3(3)(a)(2)(a).

(1) General Features -- See Index 873.3(3)(a)(1)(a) through (e) for discussions on methods of placement, foundation treatment, rock slope protection fabrics and gravel filters.

(2) Stone Size -- Two methods for determining riprap size for stability under wave action are presented in HEC 23, Volume 2, Design Guideline 17: (1) the Hudson method, and (2) the Pilarczyk method.

(a) The Hudson Method. Applications of Hudson's equation in situations with a design significant wave height of H=5 feet or less have performed well. This range of design wave heights encompasses many coastal revetments along highway embankments. When design wave heights get large and the design water depths get large, problems with the performance of rubble-mound structures can occur. A more conservative design approach should use a more conservative H statistic. The proper input wave height statistic is required and discussed in Section 6.3 of HEC 25, Volume 1. RSP with design wave heights much greater than H=5 feet require more judgment and more experience and input from a trained, experienced coastal engineer. Therefore, when design wave heights are much greater than H=5 feet, contact the District Hydraulic Engineer. The Hudson method considers wave height, riprap density, and slope of the bank or shoreline to compute a required weight of a median-size riprap particle.

$$W_{50} = \frac{\gamma_r H^3 (\tan\theta)}{K_d (S_r - S_w)^3}$$

Where:

W_{50} = weight of median riprap particle size, (lb)

γ_r = unit weight of riprap, (lb/ft^3)

H = design wave height, (ft) (Note: Minimum recognized value for use with the Hudson equation is the 10 percent wave, $H_{0.10} = 1.27H_s$)

K_d = empirical coefficient equal to 2.2 for riprap

S_r = specific gravity for riprap

S_w = specific gravity for water (1.0 for fresh water, 1.3 for sea water)

θ = angle of slope inclination

The median weight W_{50} can be converted to an equivalent particle size d_{50} by the following relationship:

$$d_{50} = \sqrt[3]{\frac{W_{50}}{0.85\gamma_r}}$$

(b) The Pilarczyk Method. Compared to the Hudson method, the Pilarczyk method considers additional variables associated with particle stability in different wave environments, and therefore should more thoroughly characterize the rock stability threshold. The hydraulic processes that influence rock revetment stability are directly related to the type of wave that impacts the slope, as characterized by the breaker parameter. The breaker parameter is a dimensionless quantity that relates the bank slope, wave period, wave height, and wave length to distinguish between the types of breaking waves. This parameter is defined as:

$$\xi = \frac{\tan\theta}{\sqrt{H_s/L_o}} = \tan\theta\,\frac{K_u T}{\sqrt{H_s}}$$

Where:

ξ = dimensionless breaker parameter

θ = angle of slope inclination

L_o = wave height, (ft)

H_s = significant wave height, (ft)

T = wave period, (sec)

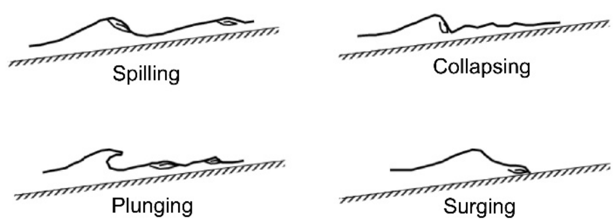
K_u = coefficient equal to 2.25 for wave height, (ft)

The wave types corresponding to the breaker parameter are listed in Table 883.2 and illustrated schematically below.

Table 883.2

Dimensionless Breaker Parameter and Wave Types

Value of the Dimensionless Breaker Parameter ξ	Type of Wave
$\xi < 0.5$	Spilling
$0.5 < \xi < 2.5$	Plunging
$2.5 < \xi < 3.5$	Collapsing
$\xi < 3.5$	Surging



The Pilarczyk method, like the Hudson method, uses a general empirical relationship for particle stability under wave action. When design wave heights are much greater than H=5 feet, contact the District Hydraulic Engineer. The Pilarczyk equation is:

$$\frac{H_s}{\Delta D} \leq \psi_u \phi \frac{\cos\theta}{\xi^b}$$

Where:

H_s = significant wave height, (ft)

Δ = relative unit weight of riprap, $\Delta = (\gamma_r - \gamma_w)/\gamma_w$

D = armor size thickness, (ft)

ψ_u = stability upgrade factor (1.0 for good riprap)

ϕ = stability factor (1.5 for good quality, angular riprap)

θ = angle of slope inclination

ξ = dimensionless breaker parameter

b = exponent (0.5 for riprap)

Rearranging the Pilarczyk equation to solve for the required stone size, and inserting the recommended values for riprap with a specific gravity of 2.65 and a fresh water specific gravity of 1.0 yields the following equation for sizing rock riprap for wave attack:

$$d_{50} \geq \frac{2}{3}\left(\frac{H_s \xi^{0.5}}{1.64 \cos\theta}\right)$$

For salt water locations (specific gravity = 1.03), substitute 1.57 for 1.64 into the denominator of the above equation.

Using standard sizes the appropriate gradation can be achieved by selecting the next larger size class, thereby creating a slightly over-designed structure, but economically a less expensive one. For example, if a riprap sizing calculation results in a required d_{50} of 16.8 inches, Class V riprap should be specified because it has a nominal d_{50} of 18 inches. See Table 873.3A.

Worked examples of the Pilarczyk and the Hudson method are presented in HEC 23, Design Guideline 17. Compared with the Hudson method, the Pilarczyk method is more complicated and includes the consideration of wave period, storm duration, clearly-defined damage level and permeability of structure. The choice of the appropriate formula is dependent on the design purpose (i.e. preliminary design or detailed design).

(3) Design Height -- The recommended vertical extent of riprap for wave attack includes consideration of high tide elevation, storm surge, wind setup, wave height, and wave runup. Details can be found in HEC 25, Volume 1, and HEC 23, Volume 2, Index 17.3.2.

(3) Bulkheads. The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

- Gravity or pile supported concrete or masonry walls.
- Crib walls
- Sheet piling

(a) Concrete or Masonry Walls. The expertise and coordination of several engineering disciplines is required to accomplish the development of PS&E for concrete walls serving the dual purpose of slope protection and support. The Division of Structures is responsible for the structural integrity of all retaining walls, including bulkheads.

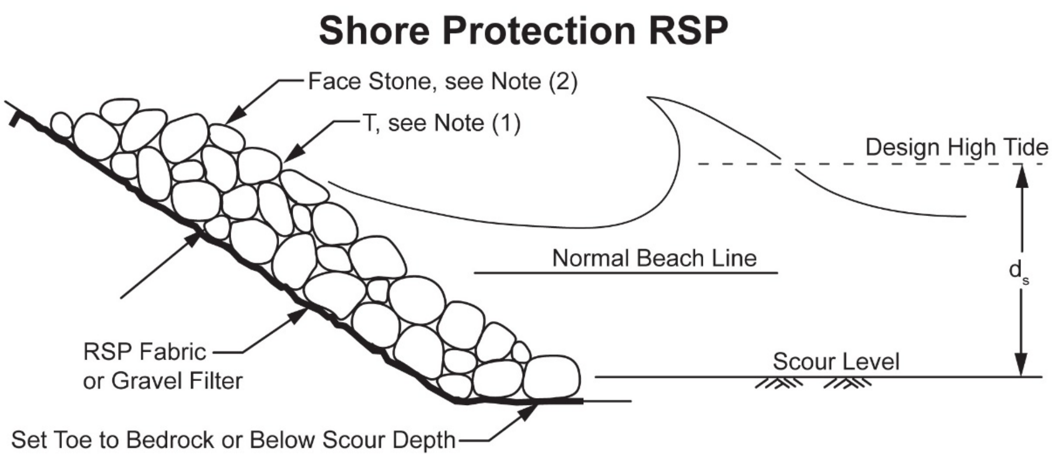
(b) Crib walls. Timber and concrete cribs can be used for bulkheads in locations where some flexibility is desirable or permissible. Metal cribs are limited to support of embankment and are not recommended for use as protection because of vulnerability to corrosion and abrasion.

The design of crib walls is essentially a determination of line, foundation grade, and height with special attention given to potential scour and possible loss of backfill at the base and along the toe. Design details for concrete crib walls are shown on Standard Plans C7A through C7C. Concrete crib walls used as bulkheads and exposed to salt water require special provisions specifying the use of coated rebars and special high density concrete. Recommendations from METS Corrosion Technology Branch should be requested.

(c) Sheet Piling. Timber, concrete and steel sheet piling are used for bulkheads that depend on deep penetration of foundation materials for all or part of their stability. High bulkheads are usually counterforted at upper levels with batter piles or tie back systems to deadmen. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles.

Figure 883.2G

Rock Slope Protection



NOTES:

(1) Thickness "T" = 1.5 d_{50}

(2) Face stone size is determined from Index 883.3(2)(b).

(3) RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

Excluding structural requirements, design of pile bulkheads is essentially as follows:

- Recognition of foundation conditions suitable to or demanding deep penetration. Penetration of at least 15 feet below scour level, or into soft rock, should be assured.
- Choice of material. Timber is suitable for very dry or very wet climates, for other situations economic comparison of preliminary designs and alternative materials should be made.
- Determination of line and grade. Fairly smooth transitions with protection to high-water level should be provided.

(4) *Sea Walls.* Sea walls are structures, often concrete or stone, built along a portion of a coast to prevent erosion and other damage by wave action. Seawalls can be rigid structures or rubble-mound structures specifically designed to withstand large waves. Often they retain earth against the shoreward face. A seawall is typically more massive and capable of resisting greater wave forces than a bulkhead. Index 6.1 of HEC 25, Volume 1 provides several examples of seawall designs.

(5) *Groins.* A groin is a relatively slender barrier structure usually aligned to the primary motion of water designed to trap littoral drift, retard bank or shore erosion, or control movement of bed load.

These devices are usually solid; however, upon occasion to control the elevation of sediments they may be constructed with openings. Groins typically take the following forms of construction:

- Rock mound.
- Concreted-rock dike.
- Sand filled plastic coated nylon bags.
- Single or double lines of sheet piling.

The primary use of groins is for ocean shore protection. When used as stream channel protection to retard bank erosion and to control the movement of streambed material they are normally of lighter construction than that required for shore installation.

In its simplest or basic form, a groin is a spur structure extending outward from the shore over beach and shoal. A typical layout of a shore protection groin installation is shown in Figure 883.2H.

Assistance from the U.S. Army Corp of Engineers is necessary to adequately design a slope protection groin installation. For a more complete discussion on groins, designers should consult Volume II, Chapter 6, Section VI, of the Corps' Shore

Protection Manual until Part VI of the Coastal Engineering Manual is published. Preliminary studies can be made by using basic information and data available from USGS quadrangle sheets, USC & GS navigation charts, hydrographic charts on currents for the Northeast Pacific Ocean and aerial photos of the area.

Factors pertinent to design include:

(a) Alignment. Factors which influence alignment are effectiveness in detaining littoral drift, and self-protection of the groin against damage by wave action.

A field of groins acts as a series of headlands, with beaches between each pair aligned in echelon, that is, extending from outer end of the downdrift groin to an intermediate point on the updrift groin, see Figure 883.2I. The offset in beach line at each groin is a function of spacing of groins, volume of littoral drift, slope of sea bed and strength of the sea, varying measurably with the season. Length and spacing must be complementary to assure continuity of beach in front of a highway embankment.

A series of parallel spurs normal to the beach extending seaward would be correct for a littoral drift alternating upcoast and downcoast in equal measure. However, if drift is predominantly in one direction the median attack by waves contributes materially to the longshore current because of oblique approach. In that case the groin should be more effective if built oblique to the same degree. Such an alignment will warrant shortening of the groin in proportion to the cosine of the obliquity, see Figure 883.2I.

Conformity of groin to direction of approach of the median sea provides an optimum ratio of groin length to spacing, and the groin is least vulnerable to storm damage. Attack on the groin will be longitudinal during a median sea and oblique on either side in other seas.

(b) Grade. The top of groins should be parallel to the existing beach grade. Sand may pass over a low barrier. The top of the groin should be established higher than the existing beach, say 2 feet as a minimum for moderate exposure combined with an abundance of littoral drift, to 5 feet for severe exposure and deficiency of littoral drift.

The shore end should be tapered upward to prevent attack of highway embankment by rip currents, and the seaward end should be tapered downward to match the side slope of the groin in order to diffuse the direct attack of the sea on the end of the groin.

(c) Length and Spacing. The length of groin should equal or exceed the sum of the offset in shoreline at each groin plus the width of the beach from low water (LW) to high water (HW) line, see Figure 883.2I. The offset is approximately the product of the groin spacing and the obliquity (in radians) of the entrapped beach. The width of beach is the product of the slope factor and the range in stage. The relation can be formulated:

$$L = ab + rh$$

Where:

L = Length of groin, feet

a = obliquity of entrapped beach in radians

b = beach width between groins, feet

r = reciprocal of beach slope

h = range in stage, feet

For example, with groins 400 feet apart, obliquity up to 20 degrees, on a beach sloping 10:1 with a tidal range of 11 feet,

$$L = 0.35 \times 400 + 10 \times 11 = 250\ feet$$

The same formula would have required L = 390 feet for 800-foot spacing, reducing the aggregate length of groins but increasing the depth of water at the outer ends and the average cost per foot. For some combination of length and spacing the total cost will be a minimum, which should be sought for economical design.

If groins are too short, the attack of the sea will still reach the highway embankment with only some reduction of energy. Some sites may justify a combination of short groins with light revetment to accommodate this remaining energy.

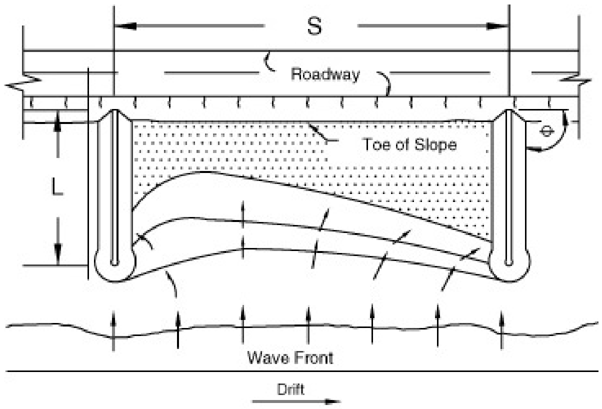
(d) Section. The typical section of a groin is shown in Figure 883.2J. The stone may be specified as a single class, or by designating classes to be used as bed, core, face and cap stones.

Face stone may be chosen one class below the requirement for revetment. Full mass stone should be specified for bed stones, for the front face at the outer end of the groin, and for cap stones exposed to overrun. Core stones in wide groins may be smaller.

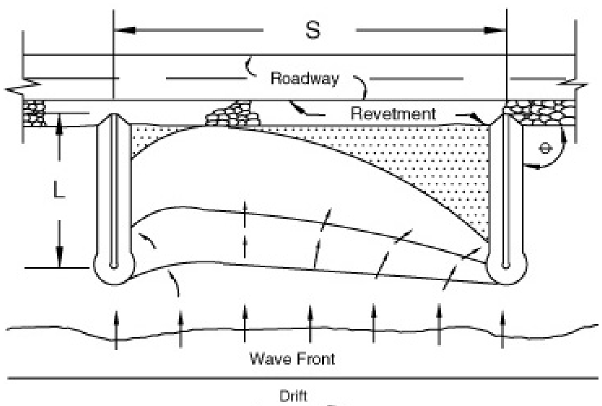
Width of groin at top should be at least 1.5 times the diameter of cap stones, or wider if necessary for operation of equipment. Side slopes should be 1.5:1 for optimum economy and ordinary stability. If this slope demands heavier stone than is available, side slope can be flattened or the cap and face stones bound together with concrete as shown in Figure 883.2J.

Figure 883.2H

Typical Groin Layout with Resultant Beach Configuration



Long Groins Without Revetment



Short Groins With Light Stone Revetment

NOTES:

"S", "L" and "θ" are determined by conditions at site.

Figure 883.2I

Alignment of Groins to an Oblique Sea Warrants Shortening Proportional to Cosine of Obliquity

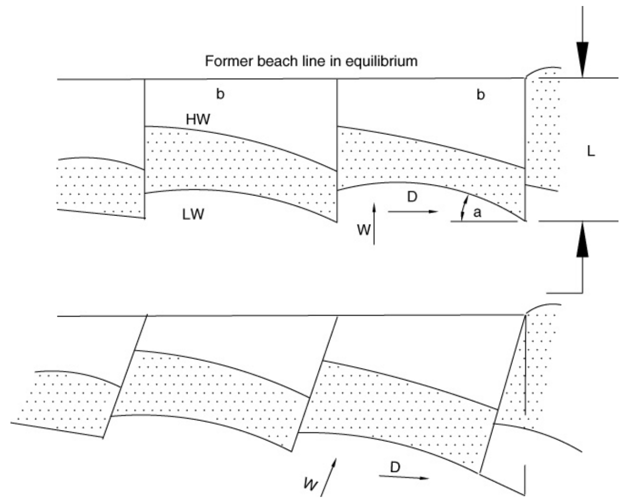
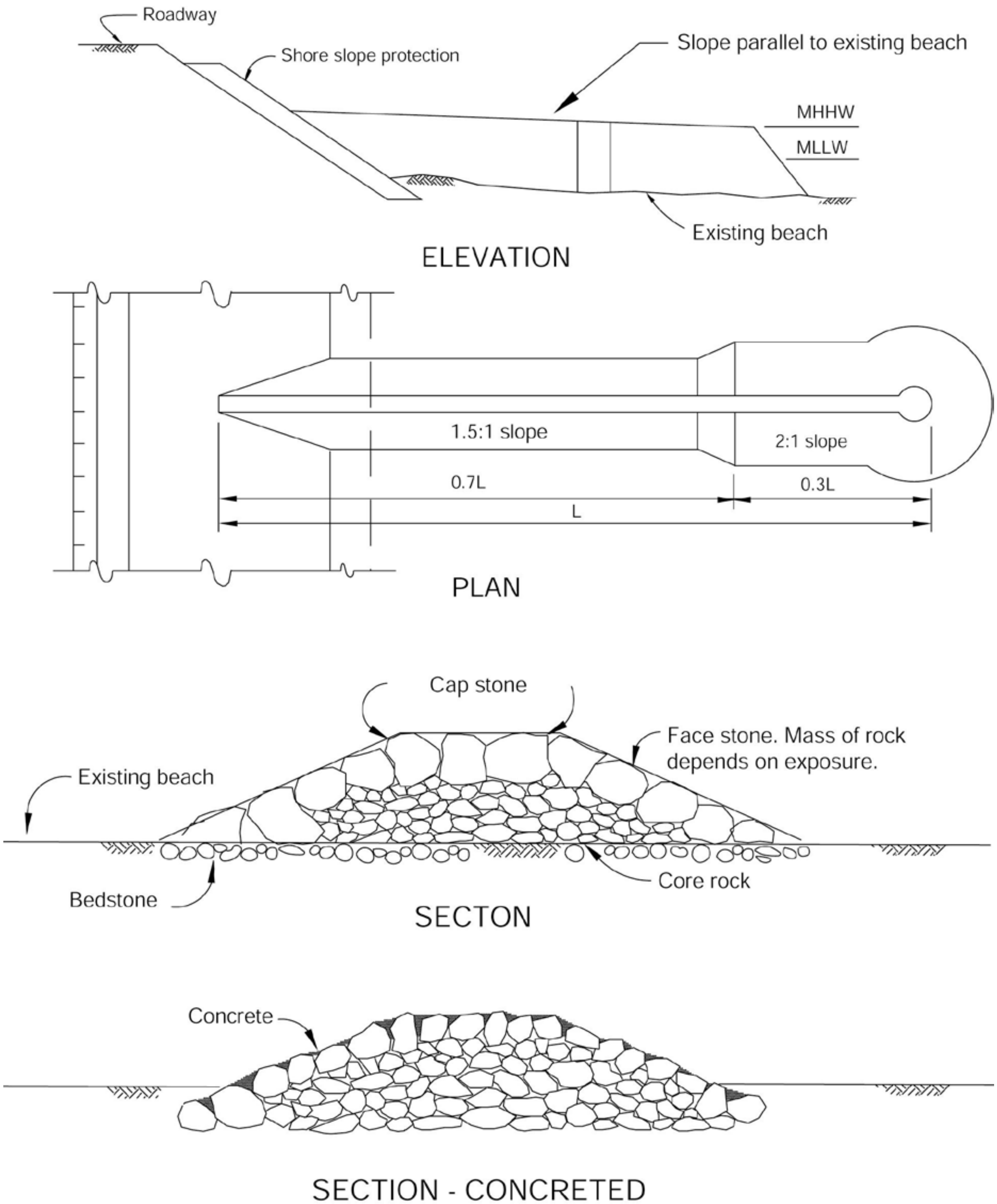


Figure 883.2J

Typical Stone Dike Groin Details



NOTES:

(1) This is not a standard design.

(2) Dimensions and details should be modified as required.

A

C

F

G

R

T