# CHAPTER 870 CHANNEL AND SHORE PROTECTION - EROSION CONTROL

## Topic 871 - General

### Index 871.1 - Introduction

Highways are often attracted to parallel locations along streams, coastal zones and lake shores. These locations are under attack from the action of waves and flowing water that may require protective measures.

Channel and shore 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 and wave action on highway 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.

Refer to Topic 874 for definition 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, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802).

There are a number of ways to deal with the problem of wave action and stream flow.

- The simplest way and generally the surest of success and permanence, is to locate the roadway away from the erosive forces. This is not always feasible or economical, but should be the first consideration. Locating the roadway 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 embankment with a more resistant material like rock slope protection. The type of material to be used for the protection is discussed under Topic 872.
- A third method is to reduce the force of the attacking water. This is often done by means of retards, permeable jetties and 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 direct the attacking water away from the embankment. In the case of wave attack, additional beach may be created between the embankment and the water by means of groins and sills which trap littoral drift or hold imported sand. 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 jetties, baffles, deflectors, groins 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 highways.

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. Retards, baffles and jetties can 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 objectives of the design are the safety of the traveling public and the security of the highway, 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 highways carry high traffic volumes, where no detour is available, or where roadway 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.
- *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. For example, 10 percent decrease in specific gravity requires a 55 percent increase in mass (say from a 9 ton stone to a 14 ton stone) for equivalent protection.
- *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 repeated here for convenience.

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

- HEC 11, Design of Riprap Revetment (2000)
- 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 (2001)
- HEC 20, Stream Stability at Highway Structures (2001)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
- HEC 25, Highways in the Coastal Environment (2008)

(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, and Volume XI - Guidelines for Highways Along Coastal Zones and Lakeshores.

(d) AASHTO Drainage Manual (MDM) (2003) – 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) U.S. Army Corps of Engineers Manuals. The following manuals are used throughout the U.S. as a primary resource for the design and analysis of coastal features:

- 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.
- Design of Coastal Revetments, Seawalls, and Bulkheads. Engineering Manual 1110-2-1614 (1995) – Supersedes portions of Volume 2 of the Shore Protection Manual (SPM).
- Coastal Engineering Manual. Engineer Manual (EM) 1110-2-1100 (2002) – Published in six parts plus an appendix, this set of documents, once complete, will supersede the SPM and EM 1110-2-1614. As of this writing Parts I thru V and the appendix are completed and available. Parts V and VI are considered "Engineering – Based" and present information on design process and selection of appropriate types of solutions to various coastal challenges.

# Topic 872 - Planning and Location Studies

## 872.1 Planning

The development of cost effective protective works requires careful planning. Planning begins with site investigation. The selection of the class 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. 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 class and 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.

Considerations at this stage are:

- The severity of 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 need be permanent or temporary.
- Analysis of foundation and materials explorations.

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 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) Young Valley.* Typically young valleys are narrow V-shaped valleys with streams on steep gradients. 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.

(a) 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 872.1 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.

**Figure 872.1**

**Slope Failure Due to Loss of Toe**

## Table 872.1

## Guide to Selection of Protection

| Location | Armor | | | | | | | | | | Training | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Flexible | | | | | Rigid | | | | | Guide Dikes Retards & Jetties | | | | Groins | | | | Baffles | | |
| | | | Mattresses | | | | | | | | | | | | | | | | | | |
| | Vegetation | Riprap | Gabions | Conc. Blocks | Fabric Filled | Grouted Rock | Stacked Conc. | Conc. Lined | Cribs | Bulk heads | Earth | Fencing | Piling | Other | Rock | Grouted Rock | Piling | Other | Drop Structure | Fencing | Rock Earth |
| Cross Channel | | | | | | | | | | | | | | | | | | | | | |
| Young Valley | | X | Ø | | | X | | | X | X | | | | | | | | | | | |
| Mature Valley | | X | Ø | Ø | Ø | X | | | X | X | X | X | X | X | X | X | | | X | X | X |
| Parallel Encroachment | | | | | | | | | | | | | | | | | | | | | |
| Young Valley | | X | Ø | | | X | | | X | X | | | | | | | | | | | |
| Mature Valley | X | X | Ø | Ø | Ø | X | Ø | | X | X | X | X | X | X | X | X | X | X | X | X | X |
| Lakes and Tidals Basins | X | X | Ø | Ø | Ø | X | Ø | | | X | | | | | | | | | | | |
| Ocean Front | | X | Ø | Ø | | | | | | X | | | | | X | X | X | X | | | |
| Desert-wash | | | | | | | | | | | | | | | | | | | | | |
| Top debris cone | | X | Ø | Ø | Ø | X | | | | X | | | | | | | | | | | |
| Center debris cone | | X | Ø | Ø | Ø | X | | | | | | | | | | | | | X | X | X |
| Bottomdebris cone | | X | Ø | Ø | Ø | X | | | | | | | | | | | | | X | X | X |
| Overflow and floodplain | X | X | Ø | | | X | Ø | | | | X | X | X | X | | | | | | | |
| Artificial channel | X | X | Ø | Ø | Ø | X | Ø | X | | | | | | | | | | | | | |
| Culvert | | | | | | | | | | | | | | | | | | | | | |
| Inlet | | X | | | | X | Ø | | | X | | | | | | | | | | | |
| Outlet | | X | | | | X | Ø | | | X | | | | | | | | | | | |
| Bridge | | | | | | | | | | | | | | | | | | | | | |
| Abutment | | X | | Ø | Ø | X | Ø | X | | | | | | | | | | | | | |
| Upstream | | X | | | | X | | | | | X | X | X | X | | | | | | | |
| Downstream | | X | | | | X | | | | | X | X | X | X | | | | | X | | |
| Roadside ditch | X | X | | | | X | Ø | X | | | | | | | | | | | | | |

Ø Where large rock for riprap is not available

## 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<br>• Unprotected upper bank | • Reduce bank slope<br>• Use more angular or smaller rock<br>• 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<br>• Exposure of filter | • Increase rock size<br>• Modify rock gradation |
| Piping or erosion beneath armor (improper filter) | Displacement of armor layer | Progressive failure | • Scalloping of upper bank<br>• Bank cutting<br>• 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<br>• Unprotected upper bank | • Increase size, thickness, depth or extent of toe or key |

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.

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.

(b) 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.

(2) *Mature Valley*. Typically mature valleys are broad V-shaped valleys with associated flood plains. The gradient and velocity of the stream are low to moderate. In addition to the general information previously given, the following applies to mature valleys.

(a) 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.

(b) 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. For a channel much shorter than the natural channel, particularly for elimination of an oxbow, the corresponding increase in gradient may require transverse weirs as grade control structures to prevent undercutting. For unusual channel changes, preliminary plans and hydraulic data must be submitted to FHWA for approval (see Index 805.5).

(3) *Lakes and Tidal Basins.* Highways adjacent to lakes or basins may be at risk from wave generated erosion. All bodies of waters 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.

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.

(4) *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.

See Index 873.3(2) for further discussion on determining the size of rocks necessary in shore protection for various wave heights.

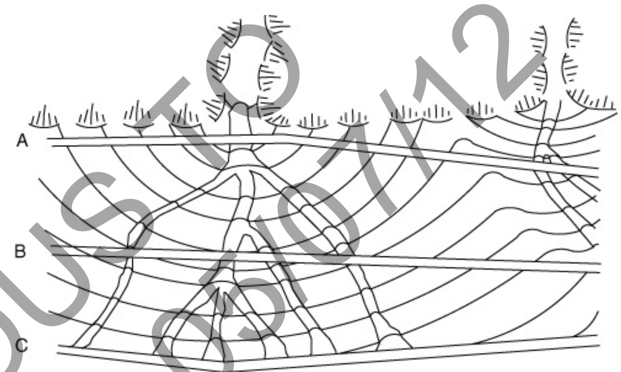
(5) *Desert Wash Locations.* Special consideration should be given to highway locations across the natural geographical features of desert washes, sand dunes, and other similar regions.

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 872.2. 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 872.3, which depicts a typical alluvial fan.

## Figure 872.2

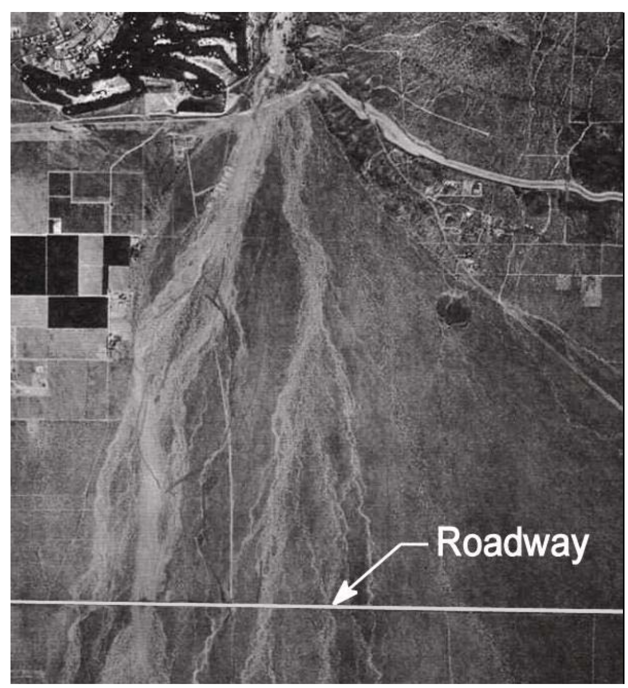
## Alternative Highway Locations Across Debris Cone




(A) crosses at a single definite channel,
(B) a series of unstable indefinite channels and
(C) a widely dispersed and diminished flow.

## Figure 872.3

## Alluvial Fan




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

Also common are roadway alignments which longitudinally encroach, or are fully within the desert wash floodplain, see Figure 872.4. 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 872.4

## Desert Wash Longitudinal Encroachment

Road washout due to longitudinal location in desert wash channel

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.

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.

### 872.4 Data Needs

The types and amount of data needed for planning and analysis of bank and shore protection varies from project to project depending upon the class and extent of the proposed protection, site location environment, and geographic area. 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.

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.

## 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 economical 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 State Highway Drainage 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 expert 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.

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.* Information needed to design shore protection is:

- Design High Water Level
- Design Wave Height

(a) Design High Water Level. 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 owners schedule of operation.

Except for inland tidal basins affected 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. 873.2A 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*.

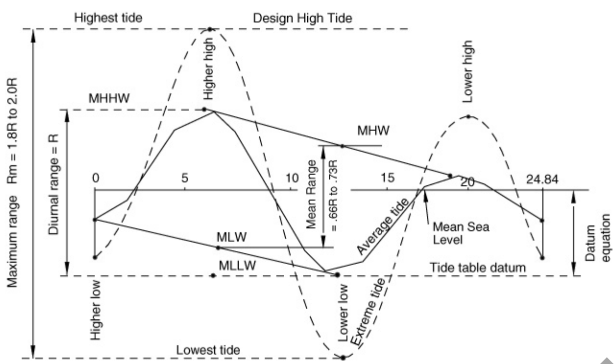
Particularly on the Pacific coast where MLLW is datum for tide tables, the stage of MHHW is numerically equal to diurnal range.

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.

## Figure 873.2A

## Nomenclature of Tidal Ranges




Because of the great variation of tidal elements, Figure 873.2A was not drawn to scale.

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

(b) Design Wave Heights.

(1) 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, but 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. 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.

(2) 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.

(3) 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

(4) 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.

(a) 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.

(b) 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 873.2B. 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, $H_s$, greater than 2 feet.

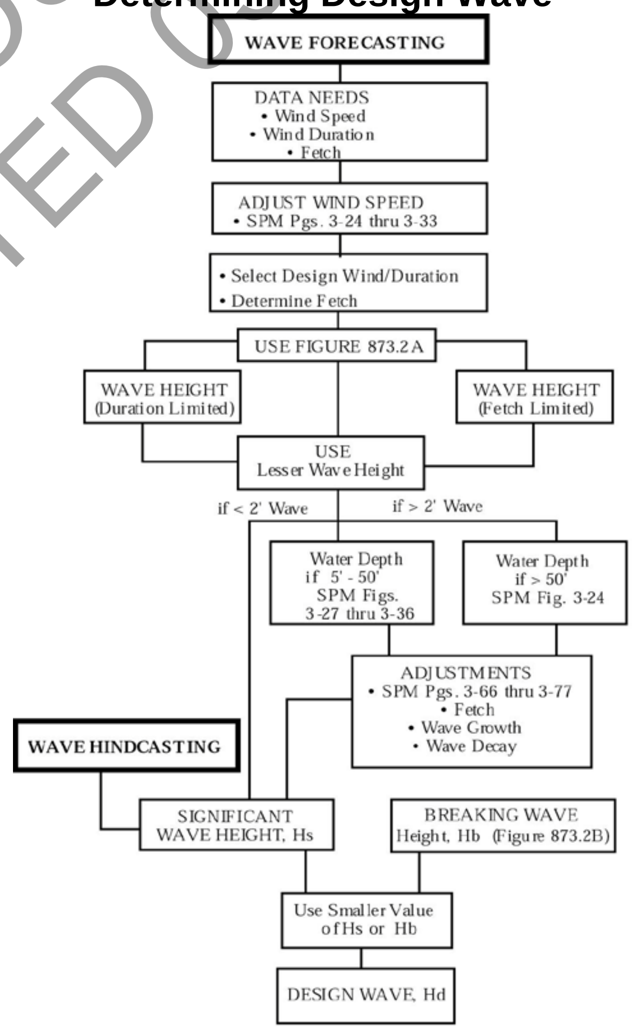
(5) Breaking Waves. Wave heights derived from hindcasts or any forecasting method should be checked against the maximum breaking wave that the design stillwater level depth and nearshore bottom slope can support. The design wave height will be the smaller of either the maximum breaker height or the forecasted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

The relationship of the maximum height of breaker which will expend its energy upon the protection, $H_b$, and the depth of water at the slope protection, $d_s$, which the wave must pass over are illustrated in Figure 873.2C.

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.

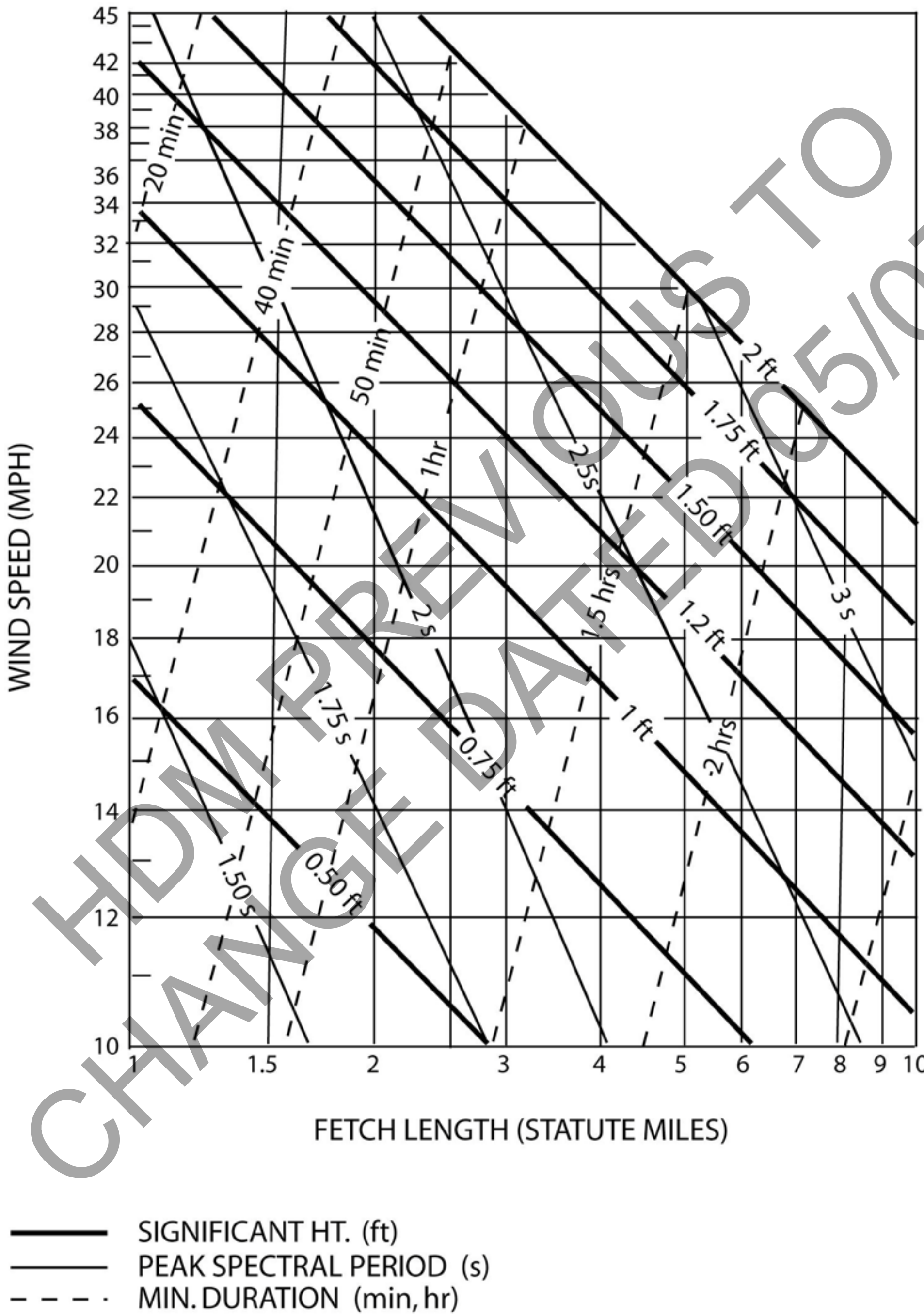
(6) Wave Run-up. Run-up is the extent, measured vertically, that an incoming wave will rise on a structure. An estimate of wave run-up, in addition to design wave height, will typically be needed and is required by policy for projects subject to California Coastal Commission (CCC) jurisdiction (see CCC 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.

**Determining Design Wave**

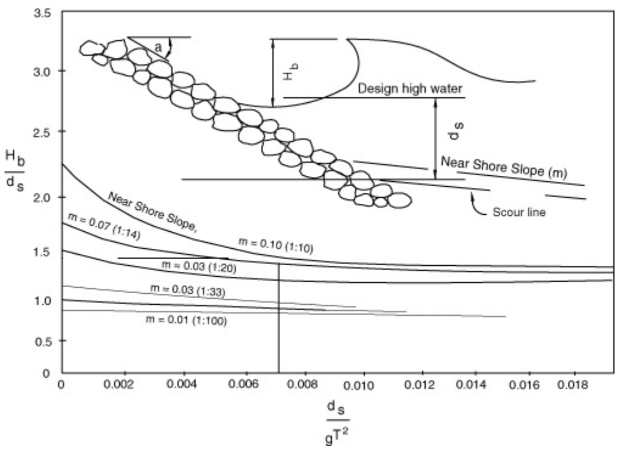


## Figure 873.2B

## Significant Wave Height Prediction Nomograph



## Figure 873.2C
## 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

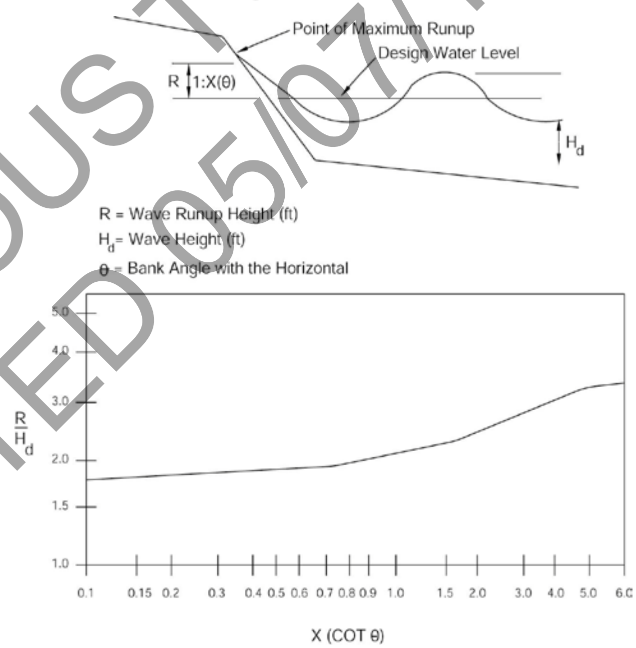
$$\frac{d_s}{gT^2} = \frac{2\text{ ft}}{(32.2\text{ ft/s}^2) \times (3\text{ sec})^2} = 0.007$$

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)

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.

## Figure 873.2D
## Wave Run-up on Smooth Impermeable Slope




(c) Littoral Processes. 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 effect 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.

## 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 and shorelines, 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, is the California Bank and Shore, (CABS), layered design. The full report is available at the following website:

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

This design method, which is applied with slight variation to ocean and lake shores vs. stream banks, and is also followed for concreted RSP designs, is the only protection method as of this writing that has been formally adopted by the Caltrans Bank and Shore Protection Committee. 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).

Other armor types listed below and described throughout this Chapter are viable and may be used, upon approval of the Headquarters Hydraulic Engineer or Caltrans Bank and Shore Protection Committee, where conditions warrant. Although the additional step of headquarters approval of these non-standard 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.
- Broken concrete slope protection.
- Broken concrete, uncoursed.
- Gabions, Standard Plan D100A and D100B.
- Precast concrete articulated blocks.
- Rock filled cellular mats.

(b) Rigid Types.

- Concreted-rock slope protection.
- Sacked concrete slope protection.
- Concrete slope protection.
- Concrete filled fabric slope protection.
- Air-blown mortar.
- Soil cement slope protection.

(c) Other Armor types:

(1) Channel Liners and Vegetation. Temporary channel lining can be used to promote vegetative growth in a drainage way or as protection prior to the placement of permanent armoring. This type of lining is used where an ordinary seeding and mulch application would not be expected to withstand the force of the channel flow. In addition to the following, other suitable products

of natural or synthetic materials are available that may be used as temporary or permanent channel liners.

- Excelsior
- Jute
- Paper mats
- Fiberglass roving
- Geosynthetic mats or cells
- Pre-cast concrete blocks with open cells
- Brush layering
- Rock riprap in sizes smaller than backing No. 3

(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
- Sea Walls

(d) 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.

*(2) 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.
- 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.
- Wave run-up is less than with smooth types (See Figure 873.2D).
- 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 and backing 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.

Under some circumstances the costs of placing rock slope protection with refinement are not justified and Method B placement can be specified. To compensate for a partial loss and assure stability and a reasonably secure protection, the thickness is increased over the more precise Method A by 25 percent.

(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 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/CHL.aspx?p=s&a=Software;3 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 and Inner Layers of Rock -- The layered method of designing RSP installations was developed prior to widespread availability of the rock slope protection fabrics which are described in Standard Specification Section 88. The RSP fabric and multiple layers of rock ensure 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 backing No. 3 or similar small, well graded materials. See Index 873.3(2)(a)(1)(e) "Gravel Filter."

Under special circumstances, the designer may consider allowing holes to be cut in the RSP fabric, generally to facilitate more rapid/extensive rooting of woody vegetation through the RSP revetment. This practice is only necessary for deeply rooted plant species. Holes in RSP fabric should not be cut below the stage of the 2-year return period event. The District Hydraulic Unit should be consulted for advice prior to any determination to cut or otherwise modify standard installation of RSP fabric.

Additionally, 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 the layered design is not feasible and 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 for establishment of vegetation that precludes the placement of fabric due to inadequate root penetration. Where RSP fabric cannot be placed, such as in stream environments where CA Fish & Game 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.

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. 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.

(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.

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 (see Figure 873.3A) 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 Figure 873.3A.
- Prevent loss of bank materials through interstitial spaces of the revetment. Rock slope protection fabric and multiple layers of backing 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 in two or more layers of rock sizes, with progressively smaller rock toward native bank to prohibit loss of soil fines.
- 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 Size -- Where stream velocity governs, rock size may be estimated by using the nomogragh, Figure 873.3A.

The nomograph is derived from the following formula:

$$W = \frac{0.00002V^6 sg_r \csc^3(\beta - \alpha)}{(sg_r - 1)^3}$$

Where:

$sg_r$ = specific gravity of stones

$\alpha$ = angle of face slope from the horizontal

$\beta$ = 70 for broken rock, a constant

W = weight of minimum stable stone in lbs

V = 2/3 average stream velocity, fps (flow parallel to bank) or 4/3 average stream velocity, fps (flow impinging on bank)

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

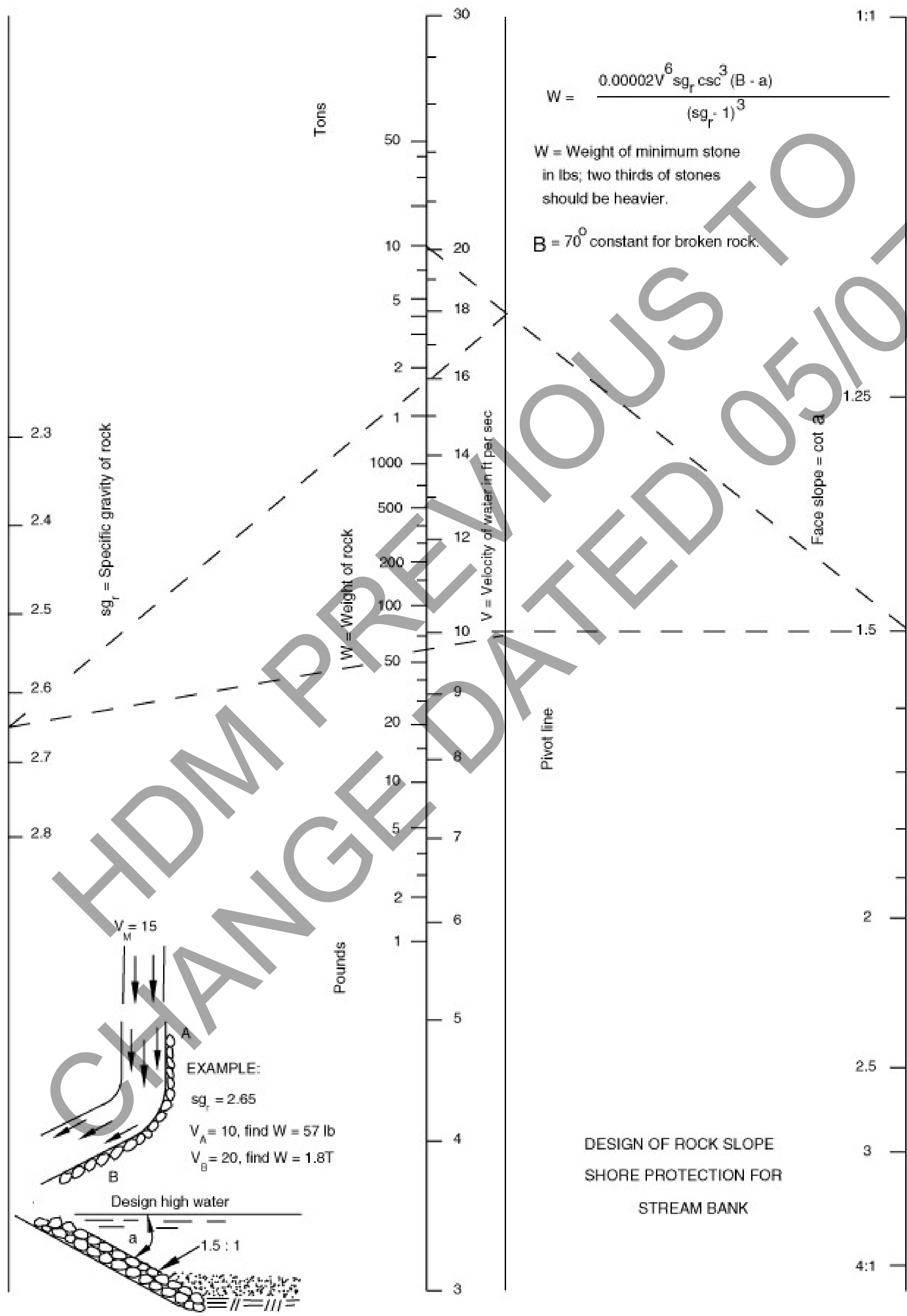
(b) Design Height -- The top of rock slope protection along a stream bank should be carried to the elevation of the design high water plus some allowance for freeboard. The flood stage elevation adopted for design may be based on an empirically derived frequency of recurrence (probability of exceedance) or historic high water marks. This stage may be exceeded during infrequent floods, usually with little or no damage to the upper slope.

Design high water should not be based on an arbitrary storm frequency alone, but should consider the cost of carrying the protection to this height, the probable duration and damage if overtopped, and the importance of the facility.

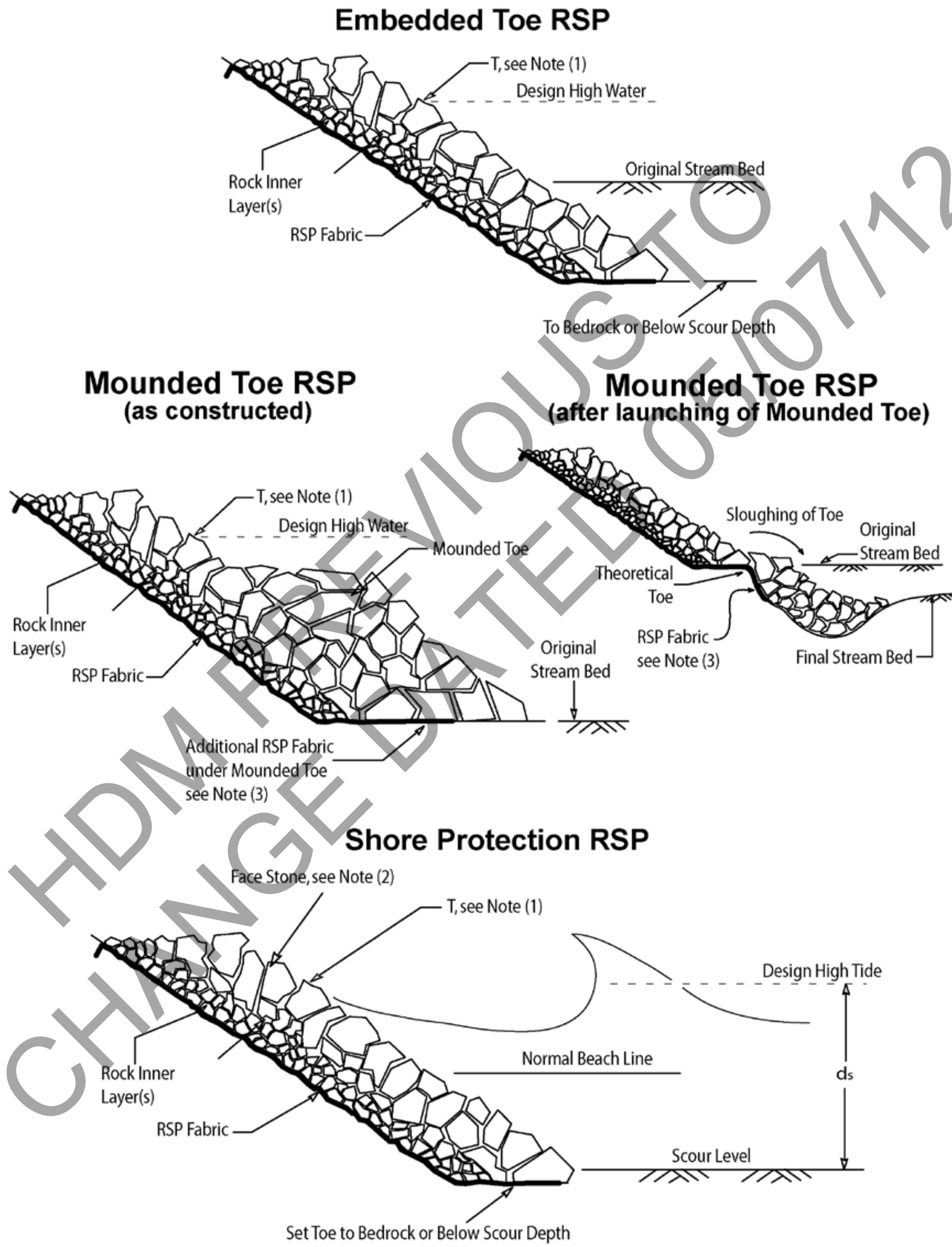
When determining freeboard, or the height above design high water from which the RSP is to extend, one should consider: the size and nature of debris in the flow; the

# Figure 873.3A

## Nomograph of Stream-Bank Rock Slope Protection



## Figure 873.3C
## Rock Slope Protection






Notes:

(1) Thickness "T" from Table 873.3 C.
(2) Face stone is determined from Figure 873.3G.
(3) RSP fabric not to extend more than 20 percent of the base width of the Mounded Toe past the Theoretical Toe.

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 along freeways, on bottleneck routes, on the outside bends of channels, or around critical bridges.

Design high water should be adjusted to the site based on sound engineering judgement.

Design Example -- The following example reflects the CABS method for designing RSP as described in Report No. FHWA – CA – TL – 95 – 10, as well as identify some of the considerations and technical principles that the designer must address to complete the installation design. These same considerations and principles apply to concreted RSP as well as RSP placed on beaches and shores (which are covered later), and therefore, separate examples for those designs are not provided. The designer is encouraged to review the entire report referenced above, available on the Division of Design website, for a comprehensive discussion of the basis of the CABS method and RSP design considerations.The following example assumes that the designer has conducted the appropriate site assessments and resulting calculations to establish average stream velocity, estimated depth of scour, stream alignment (i.e., parallel or impinging flow), length 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 – 16 feet per second
- Estimated scour depth – 5.5 feet
- Length of bank requiring protection – 550 feet
- Bank slope – 1.5:1
- Specific gravity of rock used for RSP – 2.65 (based on data from local quarry)
- Embankment is on outside of stream bend

1) Calculate minimum rock mass for outer layer:

$$W = \frac{(0.00002)\left(16 \times 4/3\right)^6 (2.65)}{(2.65-1)^3 \sin^3(70-33.69)}$$

W = 5,350 lb

W = 2.67 ton = 2.43 tonne

NOTES:

For ease of computation with hand held calculators, cosecant has been converted to 1/sine.)

2) Select gradation for outer layer.

   a) From minimum calculated rock weight of 2.67 tons in the example, select the rock weight from the left-side column tables in Standard Specification Section 72-2.02 that represents the standard rock weight just larger than the calculated weight. For ease, the Standard Specification tables are combined and reprinted in Table 873.3A.

# Table 873.3A
# Guide for Determining RSP-Class of Outside Layer

| Standard Rock Sizes | GRADING OF ROCK SLOPE PROTECTION PERCENTAGE LARGER THAN | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | RSP-Classes [1] | | | | | | | | | | | |
| | Method A Placement | | | | | Method B Placement | | | | | | |
| | RSP-Classes other than Backing | | | | | | | | | Backing No. | | |
| | 8 T | 4 T | 2 T | 1 T | 1/2 T | 1 T | 1/2 T | 1/4 T | Light | 1 | 2 | 3 |
| 16 ton | 0-5 | | | | | | | | | | | |
| 8 ton | 50-100 | 0-5 | | | | | | | | | | |
| 4 ton | 95-100 | 50-100 | 0-5 | | | | | | | | | |
| 2 ton | | 95-100 | 50-100 | 0-5 | | 0-5 | | | | | | |
| 1 ton | | | 95-100 | 50-100 | 0-5 | 50-100 | 0-5 | | | | | |
| 1/2 ton | | | | 95-100 | 50-100 | ----- | 50-100 | 0-5 | | | | |
| 1/4 ton | | | | | 95-100 | 95-100 | ----- | 50-100 | 0-5 | | | |
| 200 lb | | | | | | | 95-100 | ----- | 50-100 | 0-5 | | |
| 75 lb | | | | | | | | 95-100 | ----- | 50-100 | 0-5 | |
| 25 lb | | | | | | | | | 95-100 | 90-100 | 25-75 | 0-5 |
| 5 lb | | | | | | | | | | | 90-100 | 25-75 |
| 1 lb | | | | | | | | | | | | 90-100 |

[1] “Facing” has same gradation as “Backing No. 1”. To conserve space “Facing” is not shown.

The next larger rock mass above 2.67 ton is 4 ton. RSP this large is only to be installed using Method A placement techniques (i.e., individual rock placement, no end dumping). From this value, move horizontally across the gradation ranges to the "50-100" entry. From here, move vertically upward to select the design gradation, or RSP Class. In this instance the name of the RSP class is 4 T.

(b) Generally, this will represent the design outer RSP layer. However, the designer must assess this value against the site conditions observed during the field review and in conjunction with site history and projected future conditions prior to finalizing the selection. For the purposes of this example, we will assume this design gradation (i.e., 4 T RSP class) is appropriate.

3) Determine RSP Layers. As previously discussed, properly designed RSP revetments are comprised of multiple layers of progressively smaller rock gradations progressing from the large sized rocks of the outer layer to the native soil or constructed embankment. Where the outer layer is composed of relatively small rock only a single inner layer may be needed. For a large rock outer layer as many as three inner layers may be required.

For this example, the outer RSP layer is 4 T. From Table 873.3B, there are two options for the inner layers. The reason for multiple options for the larger RSP gradation classes is to allow the designer to better select RSP that is available from local quarry sources. Either set of layered designs is acceptable. The designer should contact rock producers in proximity to the project site to obtain price quotes for the different alternatives.

This information may also be available from the District Materials Engineer. For the purposes of this example, we will select the layered design of: 4 T, 1 T, ¼ T, Backing No. 2 and Class 10 RSP Fabric.

4) Determine Thickness of Revetment. RSP layers are composed of rock classes shown in Table 873.3A. Each layer is at least 1.5 times the diameter of the median sized rock ($D_{50}$) in the gradation in order to prevent the smaller rocks in the lower layers from migrating.

Table 873.3C provides the required thickness for the various RSP gradations and types of placement (Method A or Method B). Method B placement requires an increase in thickness to account for the looser rock contact and difficulty in controlling layer thickness inherent in end dumping of rock.

Based on the table values, the total thickness of the design in our example (measured normal to the slope) is:

## Table 873.3B

## California Layered RSP

| Outsider Layer RSP-Class * | Inner Layers RSP-Class * | Backing Class No. * | RSP-Fabric Class ** |
|---|---|---|---|
| 8 T | 2 T over ½ T | 1 | 10 |
| 8 T | 1 T over ¼ T | 1 or 2 | 10 |
| 4 T | ½ T | 1 | 10 |
| 4 T | 1 T over ¼ T | 1 or 2 | 10 |
| 2 T | ½ T | 1 | 10 |
| 2 T | ¼ T | 1 or 2 | 10 |
| 1 T | Light | None | 8 |
| 1 T | ¼ T | 1 or 2 | 8 |
| ½ T | None | 1 | 8 |
| ¼ T | None | 1 or 2 | 8 |
| Light | None | None | 8 |
| Backing No.1 *** | None | None | 8 |

* Rock grading and quality requirements per Standard Specifications.

** RSP-fabric Type of geotextile and quality requirements per Section 88 Rock Slope Protection Fabric of the Standard Specifications. Class 8 RSP-fabric has lower weight per unit area and it also has lower toughness (tensile x elongation, both at break) than Class 10 RSP-fabric.

*** "Facing" RSP-Class has same gradation as Backing No. 1.

## Table 873.3 C

## Minimum Layer Thickness

| RSP-Class Layer | Method of Placement | Minimum Thickness |
|---|---|---|
| 8 T | A | 8.5 ft |
| 4 T | A | 6.8 ft |
| 2 T | A | 5.4 ft |
| 1 T | A | 4.3 ft |
| ½ T | A | 3.4 ft |
| 1 T | B | 5.4 ft |
| ½ T | B | 4.3 ft |
| ¼ T | B | 3.3 ft |
| Light | B | 2.5 ft |
| Facing | B | 1.8 ft |
| Backing No. 1 | B | 1.8 ft |
| Backing No. 2 | B | 1.25 ft |
| Backing No. 3 | B | 0.75 ft |

4 T Layer = 6.8 ft

1 T Layer = 4.3 ft

¼ T Layer = 3.3 ft

Backing No. 2 Layer = 1.25 ft

RSP Fabric = Effectively

\+ 0.0 ft

Total = 15.35 ft

5) 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.

6) 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)(b) “Design Height.” For depth of toe, the estimated scour was given as 5.5 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 500 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, see Figure 873.3D for example at ocean shore location.

(b) Rock Slope Shore Protection.

(1) General Features. Rock slope protection when used for shore protection, in addition to the general advantages listed previously for streambank rock slope protection, reduces wave runup as compared to smooth types of protection.

(a) Method A placement is normally specified for ocean shore protection since very large stone is typically needed. Rock mass for lake shores and protected bays are often based on the height of boat generated waves.

(b) Foundation treatment in shore protection may be controlled by tidal action as well as excavation difficulties and production may be limited to only two or three toe or foundation rocks per tide cycle. If toe rocks are not properly bedded, the subsequent vertical adjustment may be detrimental to the protection above. Even though rock is self-adjusting, the bearing of one rock to another may be lost. It is often necessary to construct the toe or foundation to an elevation approximating high tide in advance of embankment construction to prevent erosion of the embankment.

(2) Shore Protection Design.

(a) Stone Size -- For waves that are shoaling as they approach the protection the required stone size may be determined by Using Chart B, Figure 873.3G.

The nomograph is derived from the following formula:

$$W = \frac{0.003 d_B{}^3 sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

$d_B$ = maximum depth in feet of water at toe of the rock slope protection, see Figure 873.3C

$sg_r$ = specific gravity of stones

$sg_w$ = specific gravity of water (sea water = 1.0265)

$\alpha$ = angle of face slope from the horizontal

$\beta$ = 70 for broken rock, a constant

W = weight of minimum stable stone in lbs

In general, $d_B$ will be the difference between the elevation of the scour line at the toe and the maximum stillwater level. For ocean shore, $d_S$ may be taken as the distance from the scour line to mean sea level plus one-half the maximum tidal range.

If the deep-water waves, see Figure 873.3D, reach the protection, the stone size may be determined by using Chart A, Figure 873.3G. The nomograph is derived from the following formula:

$$W = \frac{0.00231 H_d{}^3 sg_r \csc^3(\beta - \alpha)}{\left(\frac{sg_r}{sg_w} - 1\right)^3}$$

Where:

$H_d$ = design wave in feet, see Index 873.2

If in doubt whether waves generated by fetch and wind velocity will be of sufficient size to be affected by shoaling, use both charts and adopt the smaller value.

## Figure 873.3D

## RSP Lined Ocean Shore

RSP placed at site subject to deep water wave attack. Terminal end of RSP tied into natural rock outcropping.

(b) Dimensions -- Rock should be founded in a toe trench dug to hard rock or keyed into soft rock. If bedrock is not within reach, the toe should be carried below the estimated depth of probable scour. If the scour depth is questionable, additional thickness of rock may be placed at the toe which will adjust and provide deeper support. In determining the elevation of the scoured beach line the designer should observe conditions during the winter season, consult records, or ask persons who have a knowledge of past conditions.

Wave run-up is reduced by the rough surface of rock slope protection. In order that the wash will not top the rock, it should be carried up to an elevation of twice the maximum depth of water ($2d_S$) or to an elevation equal to the maximum depth of water plus the deep-water wave height ($d_S + H_d$), whichever is the *lower*. See Figure 873.3C.

Consideration should also be given to protecting the bank above the rock slope protection from splash and spray.

Design thickness of the protection should be based on the same procedures as used for streambanks. For typical conditions the thickness required for the various sizes are shown on Table 873.3B. Except for toes on questionable foundation, as explained above, additional thickness will not compensate for undersized stones. When properly constructed, the largest stones will be on the outside, and if the wave forces displace these, additional thickness will only add slightly to the time of failure. Shore revetments, particularly ocean shore locations, are often candidates for using a mounded toe design. Where it is not practical to excavate to bedrock or to the anticipated scour depth to set the revetment toe, an alternative treatment is to place additional rock (i.e., mound) of the same mass as the outer layer at the toe. The volume to be placed should be slightly greater than the amount that would have been needed to extend the toe to the estimated scour depth. See figure 873.3C for a depiction of a mounded toe installation.

As scour occurs at the toe of the revetment, this mounded rock will drop into the scour hole. It is important in mounded toe designs to require that excess RSP fabric be placed so that as the scour hole develops and rock begins to drop, the excess RSP fabric will "unfold" and also drop into place to limit loss of embankment.

(c) 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. Wall type revetment is not fully self adjusting but has some flexibility. The mattress type is very flexible. For some locations, gabions may be more aesthetically acceptable than rock riprap. Where larger stone sizes are not readily available and the flow does not abrade the wire baskets, they may also be more cost effective. However, caution is advised regarding in-stream placement of gabions, and some form of abrasion protection in the form of wooden planks or other facing will typically be necessary, see Figure 873.3E.

## Figure 873.3E

## Gabion Lined Streambank

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

(d) 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 erosion 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.

*(3) 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 873.3F.

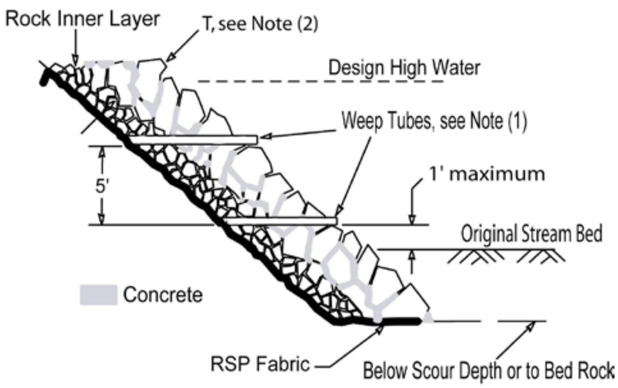
It has application in areas where rock of sufficient size for ordinary rock slope protection is not economically available.

(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.

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(2)(a)(2)(c) to select a stable rock size for a non-concreted design based on the site conditions. This non-concreted rock size is divided by a factor of roughly four or five to arrive at the appropriate size outer layer 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

## Figure 873.3F

## Concreted-Rock Slope Protection




Notes:

(1) If needed to relieve hydrostatic pressure.

(2) Refer to Table 873.3 C for section thickness.

Dimensions and details should be modified as required.

derived from this calculation to enter Table 873.3A (i.e., round up to the next larger rock mass, which will represent the 50-100 percentage larger than gradation range) and then select the appropriate RSP Class. The thickness and rock sizing of the inner layers can be based on the reduced sizing of the outer layer rock. Note that as shown in Figure 873.3F, the inner layers of rock are not concreted.

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 873.3H. 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.

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.

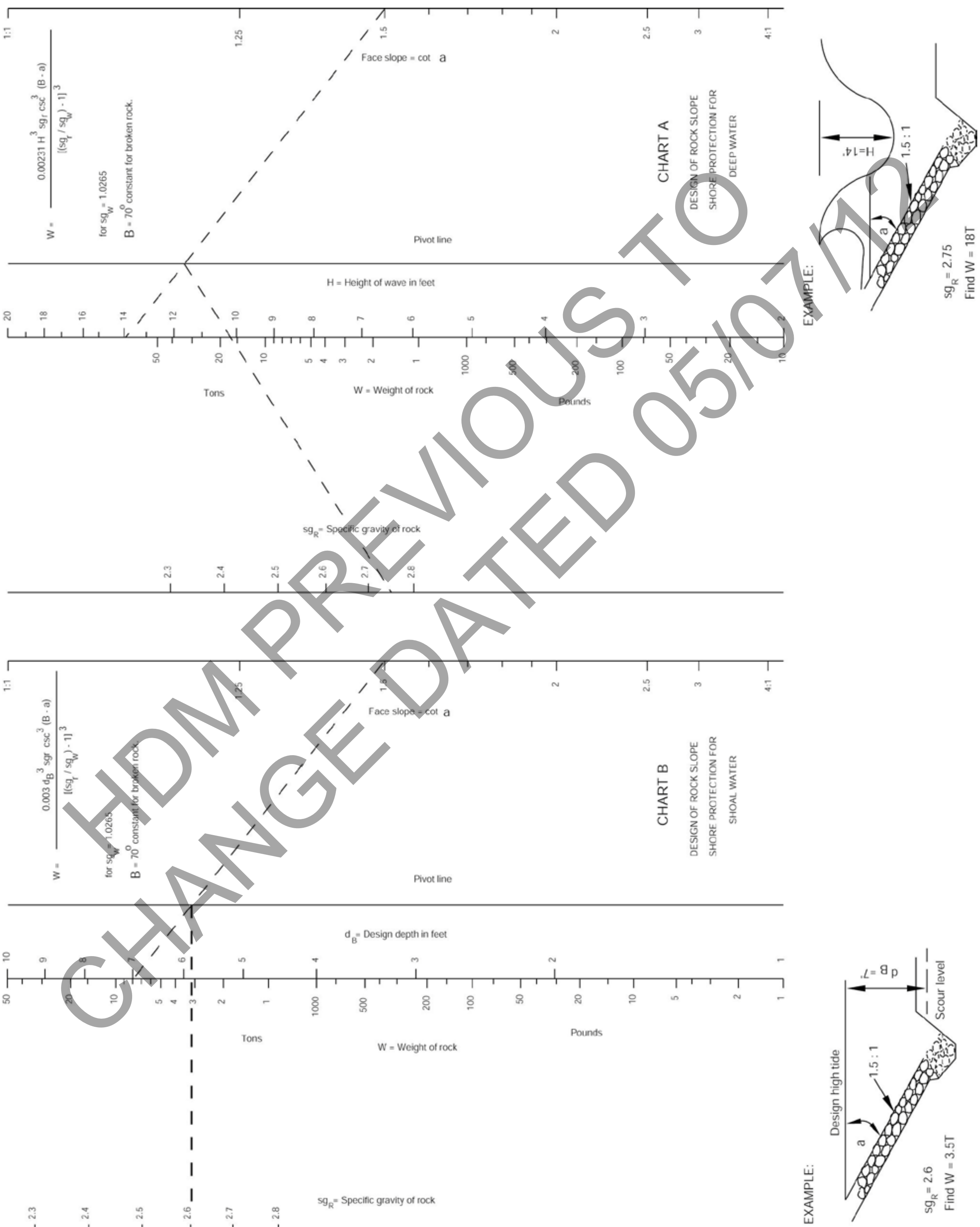
The volume of concrete required is based on filling roughly two-thirds of the void space of the outer rock layer, as shown in Figure 873.3F. The concrete is rodded or vibrated into place leaving the outer stones partially exposed. Void space for the various RSP gradations ranges from approximately 30 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.

## Figure 873.3H

## Toe Failure - Concreted RSP

Toe of concreted RSP that has been undermined.

# Figure 873.3G
# Nomographs For Design of Rock Slope Shore Protection



(2) 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) 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 88 should be placed behind all sacked concrete revetments. For revetments over 4 feet in height, weep tubes should also be placed, see Figure 873.3F.

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.

(c) Concrete Slope Paving.

(1) General Features. This method of protection consists of paving the embankment with portland cement concrete. Slope paving is used only where flow is controlled and will not over-top the protection.

It is particularly adaptable to locations where high-velocity flow is not detrimental but desirable and the hydraulic efficiency of smooth surfaces is important. It has been used very little in shore protection. On a cubic feet basis the cost is high but as the thickness is generally only 3 inches to 6 inches, the cost on a basis of area covered will usually be less than for sacked-concrete slope protection. This is especially so when sufficiently large quantities are involved and alignment is such as to warrant the use of mass production equipment such as slip-form pavers.

Due to the rigidity of PCC slope paving, its foundation must be good and the embankment stable. Although reinforcement will enable it to bridge small settlements of the embankment face, even moderate movements could lead to cracking of the paving or failure. The toe must be on bedrock or extend below possible scour. When this is not feasible without costly underwater construction, rock or PCC grouted RSP have been used as a foundation. A better but much more expensive solution is to place the toe on a PCC wall or piles.

Every precaution must be taken to exclude stream water from pervious zones behind the slope paving. The light slabs will be lifted by comparatively small hydrostatic pressures, opening joints or cracks at other points in a series of progressive failures leading to extensive or complete failure.

Considering the severity of failure from bank erosion or hydrostatic pressure after overtopping, 1 foot to 2 feet of freeboard above design high water is recommended for this type of revetment. Refer to HEC-11, Design of Riprap Revetment, Section 6.4, for further discussion on the use of concrete slope paving. Table 873.3D gives channel lining thickness.

**Table 873.3D**

**Channel Linings**

| Mean Velocity (ft/s) | Thickness of Lining (in) | | Minimum Reinforcement |
|---|---|---|---|
| | Sides | Bottom | |
| **Portland Cement Concrete or Air Blown Mortar** | | | |
| < 10 | 3 – 3.5 | 3.5 – 4 | 6 x 6-W2.9 x W2.9 welded wire fabric |
| 10 – 15 | 4 – 5 | 5 – 6 | #4 Bars at 12 in. and 18 in. centers |
| 15 or more | 6 – 8 | 7 – 8 | #3 Bars at 12 in. centers both ways |

(4) *Bulkheads.* A bulkhead is a steep or vertical structure supporting a natural slope or constructed embankment. As bank and shore protection structures, bulkheads serve to secure the bank against erosion as well as retaining it against sliding. As a slope protection structure, revetment design principles are used, the only essential difference being the steepness of the face slope. 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.

Along a shore, use of a bulkhead presumes a steep lake or sea bed profile, such that revetment on a 1.5:1 or flatter slope would project into prohibitively deep water or permit intolerable wave runup. Such shores are generally rocky, offering good foundation on residual reefs, but historic destruction of the overlying formation attests to the hydraulic power of the sea to be resisted by an artificial replacement. The face of such a bulkhead must be designed to absorb or dissipate as much as practical the shock of these forces. Designers should consult the U.S. Army Corps of Engineers EM-1110-2-1614, Design of Coastal Revetments, Seawalls, and Bulkheads, for more complete information and details.

(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 C7G. 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.

Design details for timber crib walls of dimensioned lumber are shown on Standard Plans C9A and C9B. Timber cribs of logs, notched to interlock at the contacts, may also be used. All dimensioned lumber should be treated to resist decay.

(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.

(5) *Vegetation*. Vegetation is the most natural method for stabilization of embankments and channel 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 862.2 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 873.3E.

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 are:

- Straw
- Excelsior
- Jute
- Woven paper

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:

- Gravel or cobble size riprap
- Fiberglass roving
- Geosynthetic mats
- Polyethelene cells or grids
- Gabion Mattresses

However, geosynthetics and plastic (polyethylene, polypropylene, polyamide, etc.) based mats 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.

## Table 873.3E

## Permissible Velocities for Flexible Channel Linings

| Type of Lining [1] | Permissible Velocity (ft/s) | |
|---|---|---|
| | Intermittent Flow | Sustained Flow |
| Vegetation: | | |
| Bermuda Grass, uncut | 4.0 | 2.5 |
| Bermuda Grass, mowed or Crab Grass, uncut | 4.0 | 2.5 |
| Riprap: | | |
| Gravel, 1 in | 3.0 | 2.0 |
| Gravel, 2 in | 3.5 | 2.5 |
| Cobble, 3 in | 5.0 | 4.0 |
| Cobble, 6 in | 7.5 | 6.5 |
| Temporary: | | |
| Woven Paper Net | 4.5 | 3.5 |
| Jute Net | 5.0 | 4.0 |
| Fiberglass Roving | 5.5 | 4.5 |
| Straw with Net | 6.5 | 4.5 |
| Curled Wood Mat | 6.5 | 4.5 |
| Synthetic Mat | 10.5 | 7.5 |

NOTE:
(1) Ref. HEC-15

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.

### 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. As shore protection, they control shoaling and scour by deflecting the strength of currents and waves.

The degree of permeability is among the most important properties of control structures. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce the strength of water velocity, currents or waves.

Training systems of the retard and permeable jetty types are similar in that they are usually extensive or multi-unit open structures like; piling, fencing, and unit frames. They are dissimilar in function and alignment, retards being parallel and groins oblique to the banks. The retard is a milder remedy than jetty construction.

(a) Retard Types. A retard is a bank protection structure designed to check riparian velocity and induce silting and accretion. They are usually placed parallel to the highway embankment or erodible banks of channels on stable gradients. Retards typically take the following forms of construction:

- Fencing - single or double lines
- Palisades - piles and netting
- Timber piling or pile bents
- Steel or timber jacks

Retards are applicable primarily on streams which meander to some extent within a mature valley. Typical uses include the following:

- Protection at the toe of highway embankments that encroach on a stream channel.
- Training and control to inhibit erosion upstream and downstream from stream crossings.
- Control of erosion redeposition of material where progressive embayments are creating a problem.

(1) Fence Type. Fence-type structures are used as retards, permeable or impermeable jetties, and as baffles. These structures can be constructed of various materials.

Fence type retards may be effective on smaller streams and areas subject to infrequent attack, such as overflow areas. Single and double rows of various types of fencing have been used. The principal difference between fence retards and ordinary wire fences is that the posts of retards must be driven sufficiently deep to avoid loss by scour.

Permeability can be varied in the design to fit the requirements of the location for single fences, the factor most readily varied is the pattern of the wire mesh. For multiple fences, the mesh pattern can be varied or the space between fences can be filled to any desired height. Making optimum use of local materials, this fill may be brush ballasted by rock, or rock alone.

(2) Piles and Palisades. Retards and jetties may be of single, double, or triple rows of piles with the outside or upstream row faced with wire mesh fencing material, boards or polymeric straps interwoven into a high-strength net. The facing adds to the retarding effect and may trap light brush or debris to supplement its purpose. This type

retard is particularly adapted to larger streams where the piles will remain in the water. The number of pile rows and amount of facing may be varied to control the deposition of material. In leveed rivers it is often desirable to discourage accretion so as to not constrict the channel but provide sufficient retarding effect to prevent loss of a light bank protection such as vegetation or light rock facing.

Typical design considerations include:

- If the stream carries heavy debris, the elevation of the top of the pile should be well below the high-water level in order that heavy objects such as logs will pass over the top during normal floods.
- Piles must have sufficient penetration to prevent loss from scour or impact by floating debris or both. This is especially important for the piles at the outer end of jetties. If scour is a problem, the pile may be protected by a layer of rock placed on the streambed. Piles should be long enough to penetrate below probable scour, with penetration of a least 15 feet in streams with sandy beds and velocities of 10 feet per second to 15 feet per second.
- Ends of the system should be joined to the bank in order to prevent parallel high-velocity flow between the retard and the bank. If the installation is long, additional bank connections may be placed at intervals.
- Facing material should be fastened to the upstream or channel side of the piling in order that the force of the water and impact of debris will not be entirely on the fasteners.

(3) Jacks and Tetrahedrons. Jacks and tetrahedrons are skeletal frames that can be used as retards or permeable jetties. Cables can be used to tie a number of similar units together in longitudinal alignment and for anchorage of key units to deadmen. Struts and wires are added to the basic frames to increase impedance to flow of water directly by their own resistance and indirectly by the debris they collect.

Both devices serve best in meandering streams which carry considerable bed load during flood stages. Impedance of the stream along the string of units will cause deposit of alluvium, especially at the crest and during the falling stage. Beds of such streams often scour on the rising stage, undercutting the units and causing their subsidence, often accompanied by rotation when one leg or side is undercut more than the other. Deposition of the falling stage usually restores the former bed, partially or completely burying the units. In that lowered and rotated position, they may still be completely effective in future floods.

Retards may be used alone or in combination with other types of slope protection. In combination with a lighter type of armor they may be more economical than a heavier type of protection. They can be used as toe protection for other types of slope protection where a good foundation is impractical because of high water or extreme depth of poor material.

Where new embankment is placed behind the retard consideration should be given to protecting the slope to inhibit erosion until the retard has had an opportunity to function. The slope protection used should promote the establishment of a natural cover, such as discussed under Index 873.3(5), Vegetation.

Retards on tangent reaches of narrow channels may, by slowing the velocity on one side, cause an increase in velocity, on the other. On wider reaches of a meandering stream they may, by slowing a rebounding high velocity thread, have a beneficial effect on the opposite bank. Where the prime purpose of the retard system is to reduce stream bank velocity to encourage deposition of material intended to alter the channel alignment the effect on adjacent property must be assessed. Where deposition of material is the primary function, the service life of the installation is dependent on the deposition rate and the ultimate establishment of a natural retard.

The length of a retard system should extend from a secure anchorage on the upstream end to anchorage on the downstream end beyond the area under direct attack. Since erosion often progresses downstream, this possibility should be considered in determining the planned length.

The top of a retard need not extend to the elevation of design high water. In major rivers and streams where drift is large and heavy it is essential that the retard be low enough to pass debris over the top during stages of high flow.

For further information on retards, refer to Section 6.4.4 of HDS No. 6.

(b) Jetty Types. A jetty is an elongated artificial obstruction projecting into a stream or the sea from bank or shore to control shoaling and scour by deflection or redirection of currents and waves. When used in stream environments, a common term used for these devices is spur dike.

This classification may be subdivided with respect to permeability. Impermeable jetties being used to deflect the stream and permeable jetties being used not only to deflect the stream but to permit some flow through the structure to minimize the formation of eddies immediately downstream. Most jetty installations are permeable structures.

Permeable jetties typically take the following forms of construction:

- Palisades -- piles and netting.
- Single and double rows of timber-braced piling.
- Steel or timber jacks.
- Precast concrete, interlocking shapes or hollow blocks.

Impermeable jetties typically take the following forms of construction:

- Guide and spur dikes, earth or rock.
- PCC grouted riprap dikes.
- Single and double lines of sheeting or sheet piling (steel, timber or concrete, framed and braced or on piling).
- Double fence, filled.
- Log or timber cribs, filled.

Impermeable jetties in the form of filled fences and cribs have been used with only limited success. Characteristic performance of these is the development of an eddy current immediately downstream which attacks the bank and often requires secondary protective measures.

Basic principles for permeable jetties are much the same as for retards, the important difference being that they deflect the flow in addition to encouraging deposition. The preceding comment on retards should be considered as related and applicable to jetties when qualified by this basic difference.

Permeable jetties are placed at an angle with the embankment and are more applicable in meandering streams for the purpose of directing or forcing the current away from the embankment, see Figure 873.4A. When the purpose is to deposit material and promote growth, the jetties 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.

They also encourage deposition of bed material and growth of vegetation. Retards build a narrow strip in front of the embankment, where as permeable jetties cover a wider area roughly limited by the envelope of the outer ends.

The relation between length and spacing of jetties should approximate unity as a general rule to assure complete entrapment and retention of material. The spacing can be increased if the resulting scalloped effect is not detrimental to the desired result. See HEC-23, Bridge Scour and Stream Instability Countermeasures, Design Guideline 9 for additional information.

(c) Guide Dikes/Banks. Guide banks are appendages to the highway embankment at bridge abutments, see Figure 873.4B. They are smooth extensions of the fill slope on the upstream side. Approach embankments are frequently planned to project into wide floodplains, to attain an economic length of bridge. At these locations high water flows can cause damaging eddy currents that scour away abutment foundations and erode approach embankments. The purpose of guide dikes is twofold. The first is to align flow from a wide floodplain toward the bridge opening. The second is to move the damaging eddy currents from the approach roadway embankment to the upstream end of the dike.

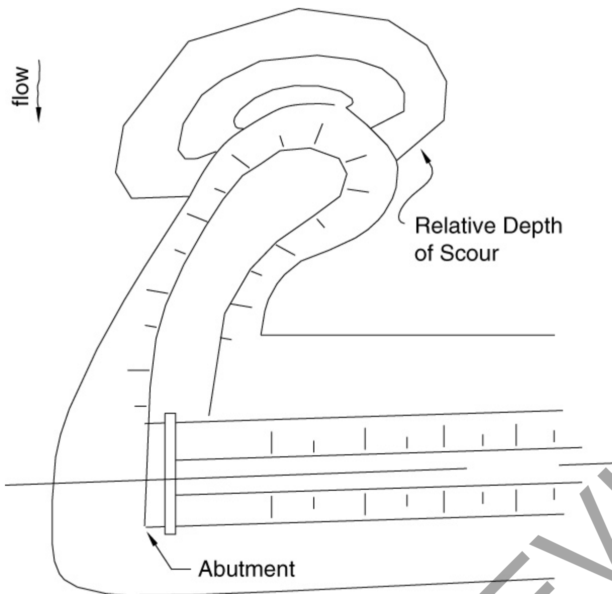
Guide banks are usually earthen embankment faced with rock slope protection. Optimum shape and length of guide dikes will be different for each site. Field experience has shown that an elliptical shape with a major to minor axis ratio of 2.5:1 is effective in reducing turbulence. The length is dependant on the ratio of flow diverted from the flood plain 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 flood plain 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.

For further information on spur dike and guide bank design procedures, refer to Section 6.4 of HDS No. 6. General design considerations and guidance for evaluating scour and stream stability at highway bridges is contained in HEC-18, HEC-20, and HEC-23.

## Figure 873.4B

## Bridge Abutment Guide Banks




(d) 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:

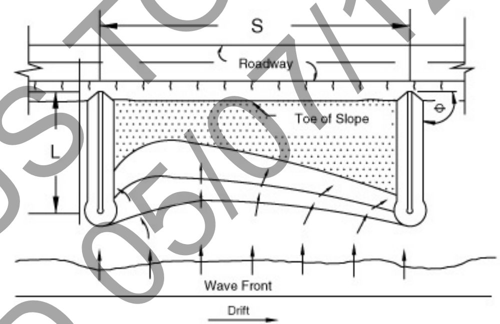
- Rock mound.
- Concreted-rock dike.
- Sand filled plastic coated nylon bags.
- Single or double lines of sheet piling.

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.

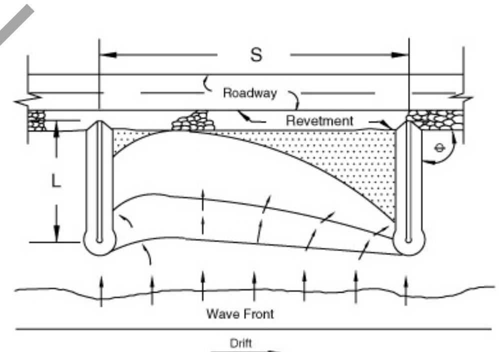
In its simplest or basic form, a groin is a spur structure extending outward from the shore over beach and shoal. A typical layout of a shore protection groin installation is shown in Figure 873.4C.

## Figure 873.4C

## Typical Groin Layout With Resultant Beach Configuration




LONG GROINS WITHOUT REVETMENT




SHORT GROINS WITH LIGHT STONE REVETMENT

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

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 grouns, 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.

For a groin to function satisfactorily, there must be littoral drift to supply and replenish the beach between groins. The groins detain rather than retain the drift and soon will be ineffective unless there is a steady source of replenishment. A new groin installation will starve the downcoast beach, temporarily at least, and permanently if the supply of drift is meager. Reference is made to the Army Corps of Engineers' Coastal Engineering Manual, Part III, for more detailed information on the littoral process.

Factors pertinent to design include:

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

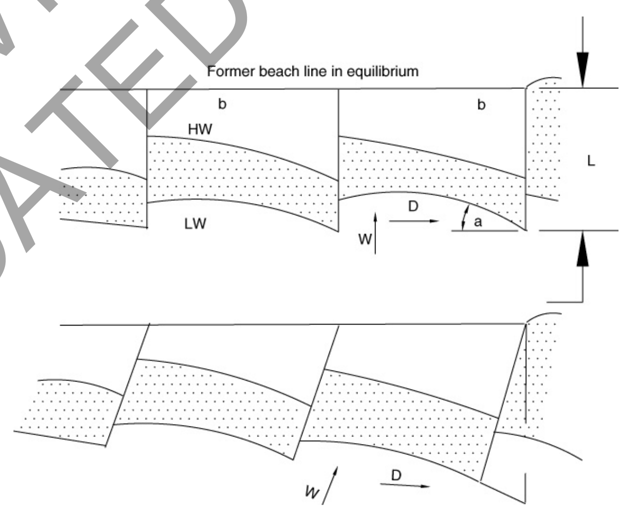
A field of groins acts as a series of headlands, with beaches between each pair aligned in echelon, that is, extending from outer end of the downdrift groin to an intermediate point on the updrift groin, see Figure 873.4D. The offset in beach line at each groin is a function of spacing of groins, volume 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 873.4D.

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.

## Figure 873.4D

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




(2) 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.

(3) 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 873.4D. 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,

$L = .35 \times 400 + 10 \times 11 = 250$ 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.

(4) Section. The typical section of a groin is shown in Figure 873.4E. 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 by Chart A or B, Figure 873.3G. 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 873.3F.

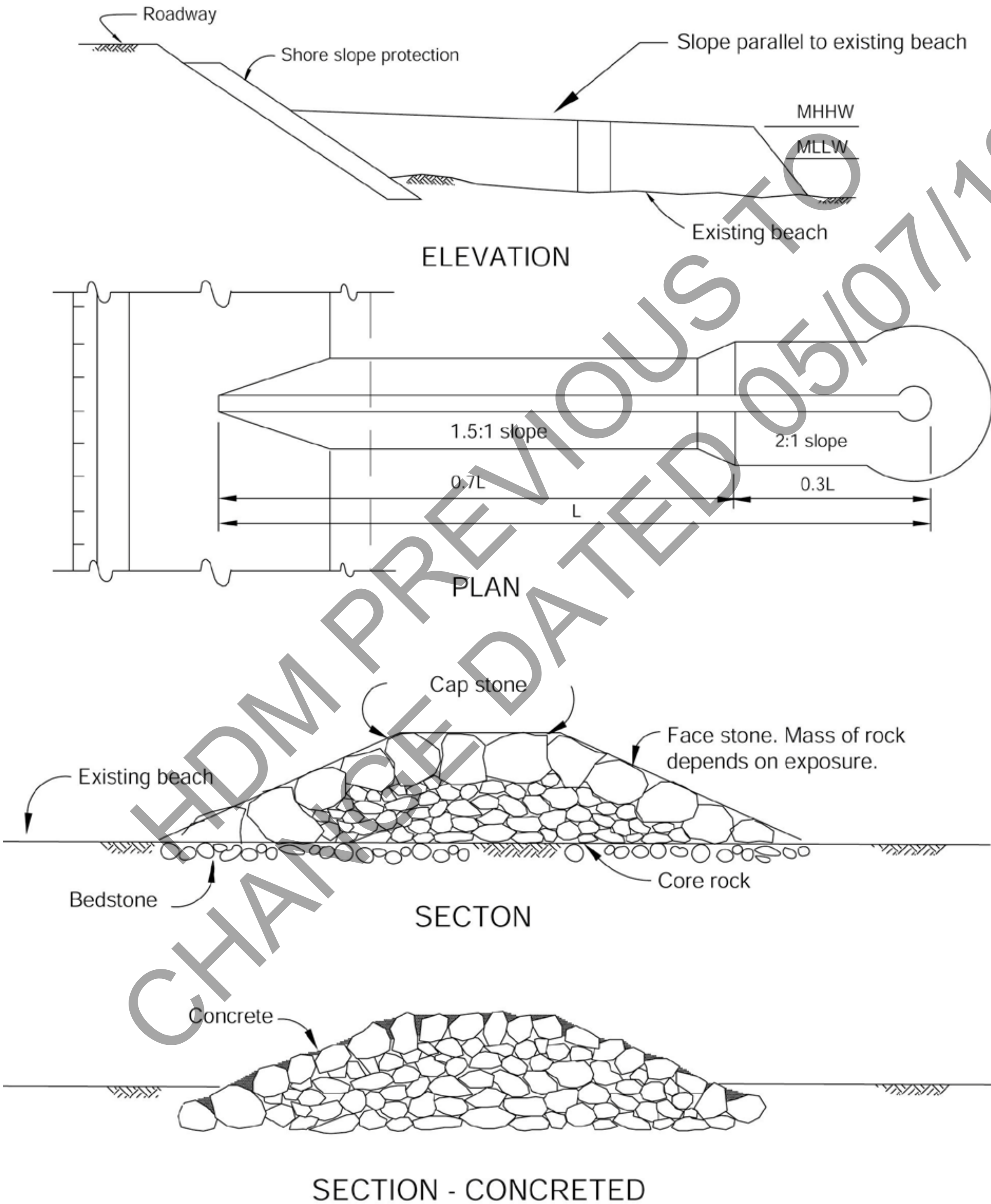
(e) Baffle. A baffle is a pier, vane, sill, fence, all or mound built on the bed of a stream to control, deflect, check or disturb the flow or to float on the surface to dampen wave action.

Baffles typically take the following forms of construction:

- Single or multiple lines of fence.
- Drop Structures (gabions, rock, concrete, etc.).
- Dikes of earth or rock.
- Floating boom.

These devices may vary in magnitude from a check dam on a small stream to a system of training dikes or permeable jetties for deflecting or directing flow. When using fences, palisades, or dikes as deflectors along the more mature valleys or meandering streams, the potential erosion

## Figure 873.4E
## Typical Stone Dike Groin Details






This is not a standard design.

Dimensions and details should be modified as required.

to previously unexposed areas, threat to adjacent property, eddy currents and possibility of scour should all be assessed. When used as a collecting system to control and direct the flow to new or existing drainage facilities or to bridge openings, the alignment of the installation should be developed as a series of curves and intervening tangents guiding the stream through transitions to maintain smooth and steady flow. The surface and curvature of the training device should be governed by the natural or modified velocity.

Drop structures or check dams are an effective means of gradient control. They may be constructed of rock, gabions, concrete, timber, sacked concrete, filled fences, 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. Refer to HDS No. 6, Section 6.4.11, for further discussion on the use of drop structures.

Floating booms are effective protection against the smaller wave actions common to lakes and tidal basins. Anchorage is the prime structural consideration.

### 873.5 Design Check List

The designer should anticipate the more significant problems that are likely to occur during the construction and maintenance of channel and shore 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, and sequence of construction should be carefully considered during the project design. The stream and shoreline morphology and their response to construction activities are an integral part of the planning process. Communication between the designer and those responsible for construction administration as well as maintenance are important.

Channel and shore 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 of the planned work with respect to:
    - The highway.
    - The stream or shore.
    - Right of way.
2. Datum control of the work, and relation of that datum to gage datum on streams, and both MSL and MLLW on the shore.
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. On fence-type construction the number of lines or rows of fence, spacing of lines, dimensions of posts, details of bracing and anchorage ties, details of ties at end.
19. The details of gabions and the filling material.
20. The size of articulated blocks, the placement of steel, and construction details relating to fabrication.
21. The corrosion considerations that may dictate specialty concretes, coated reinforcing, or other special requirements.