CHAPTER 880 – SHORE PROTECTION

Topic 881 – General

Index 881.1 – Introduction

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

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

Refer to Index 806.2 for definitions of drainage terms.

881.2 Design Philosophy

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

Information is also available from headquarters specialists in the Division of Design and Structures Design in the Division of Engineering Services (DES). The most effective designs result from involvement with Design, Environmental, Landscape Architecture, Structures, Construction, and Maintenance (for further discussion on functional responsibilities see Topic 802). For habitat characterization and assessment relative to design and obtaining project specific permits, the designer may also require input from biologists. The District Hydraulic Engineer will typically be able to

assist with selecting storm scenarios for design wave heights, the design of high water level (including sea or lake level change estimates) using coastal surge and wave models, flood analysis, water surface elevations/profiles, shear stress computations, scour analysis and hydraulic analysis for placement of coastal structures.

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

- Where avoidance is not feasible, the simplest way and generally the surest of success and permanence, is to locate the facility away from the erosive forces. This is not always feasible or economical, but should be the first consideration. Locating the facility to higher ground or solid support should never be overlooked, even when it requires excavation of solid rock, since excavated rock may serve as a valuable material for protection at other points of attack.
- The most commonly used method is to armor the shore with a more resistant material like rock slope protection. FHWA Hydraulic Engineering Circular 25 (HEC 25), Third

Edition, presents general issues and approaches in coastal highway design. Types of revetments for wave attack and coastal structures are covered in Chapters 7 and 8.

- *Rock Materials*. Optimum use should be made of local materials, considering the cost of special handling. Specific gravity and gradation of stone are major factors in shore protection and the specified minimum should not be lowered without increasing the mass of stones. See Index 873.3(3)(a)(2)(b) for equations to estimate rock size.
- *Nature*-based hybrid coastal protection solutions can be used as a protection strategy for highways.

881.3 Selected References

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

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

- HEC 14, Hydraulic Design of Energy Dissipators for Culverts and Channels (2006)
- HEC 18, Evaluating Scour at Bridges (2012)
- HEC 20, Stream Stability at Highway Structures (2012)
- HEC 23, Bridge Scour and Stream Instability Countermeasures (2009)
- HEC 25, Highways in the Coastal Environment (2020)

(b) "FHWA Nature-Based Solutions for Coastal Highway Resilience: An Implementation Guide" (2019) provides information on where and how nature based solutions can be used to protect roadway infrastructure. These techniques can also be used for shoreline protection in certain scenarios. These techniques are more appropriate in bays, estuaries, or other areas without high energy waves, therefore, nature based solutions are only appropriate in very limited areas. Hybrid techniques can combine nature based and structural techniques, to meet both goals.

(c) AASHTO Highway Drainage Guidelines – General guidelines for good erosion control practices are covered in Chapter 3 – Erosion and Sediment Control in Highway Construction; and Chapter 11 - Guidelines for Highways along Coastal Zones and Lakeshores.

(d) AASHTO Drainage Manual (2014) – Refer to Chapters; 12 – Energy Dissipators; 18 – Channel and Stream Bank Stabilization ; and 19 – Coastal Zone; and 20 – Erosion and Sediment Control. This document supersedes the 2005 AASHTO Model Drainage Manual and provides guidance on engineering practice in conformance with FHWA's HEC and HDS publications and other nationally recognized engineering policy and procedural documents.

(e) USACE 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 to provide wave equations and potential screening criteria.

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

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

(h) Caltrans Design Manual for Hybrid Coastal Protection Strategies (2022) – Provides design guidance on analysis for coastal protection structures, hybrid strategies, nature-based strategies, design recommendations and examples.

Topic 882 – Planning and Location Studies

882.1 Planning

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

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

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

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

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

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

Considerations at this stage are:

- Design life and whether the protection need be permanent or temporary.
- The severity of wave attack.
- The coastal water level and future sea level.
- Littoral drift of the beach sands.
- Seasonal shifts of the shore.
- The ratio of cost of highway replacement versus cost of protection.
- Analysis of foundation and materials explorations.
- Access for construction
- Slope (H:V)
- Vegetation type and location
- Physical habitat

- Failure mode (see Table 872.2)
- Total length of protection needed
- Distance to an alternative route

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

882.2 Site Consideration

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

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

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

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

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

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

Beach protection considerations include:

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

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

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

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 shoreline 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 revetments to aggrade the beach at embankment toe.

Topic 883 – Design

883.1 Introduction

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

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

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

883.2 Design High Water, Design Wave Height and Sea Level Rise

Information needed to design shore protection is:

- Design High Water Level
- Design Wave Height

(1) Design High Water Level

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

Per Index 873.2, a suggested starting point to provide an initial screening step is to determine the design high water level to ensure the protection withstands high water levels caused by meteorological conditions having a recurrence interval of one-half the service life of the protected facility. Depending on the type of facility, it may be appropriate to base the preliminary evaluation on a high water elevation resulting from a storm or flood with a 2 percent probability of exceedance (50 year frequency of recurrence). The first evaluation may have to be adjusted to conform with a subsequent analysis which considers the level of related risks, local historic high water marks, sea level rise and climate change. Scour countermeasures protecting structures designed by the Division of Engineering Services (DES) should include consideration of floods greater than a 1 percent probability of exceedance (100-year frequency of recurrence). See Index 873.6.

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

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

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

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

(b) Coastal Locations.

Except for inland tidal basins effected by wind tides, floods and seiches, the static or still-water level used for design of shore protection is the highest tide. In tide tables, this is the stage of the highest tide above "tide-table datum" at Mean Lower Low Water (MLLW). NOAA provides information on datum conversions at select tide gage locations. For locations without a gage, you can use VDATUM to make datum conversions. The online software is available at: https://vdatum.noaa.gov/vdatumweb/

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

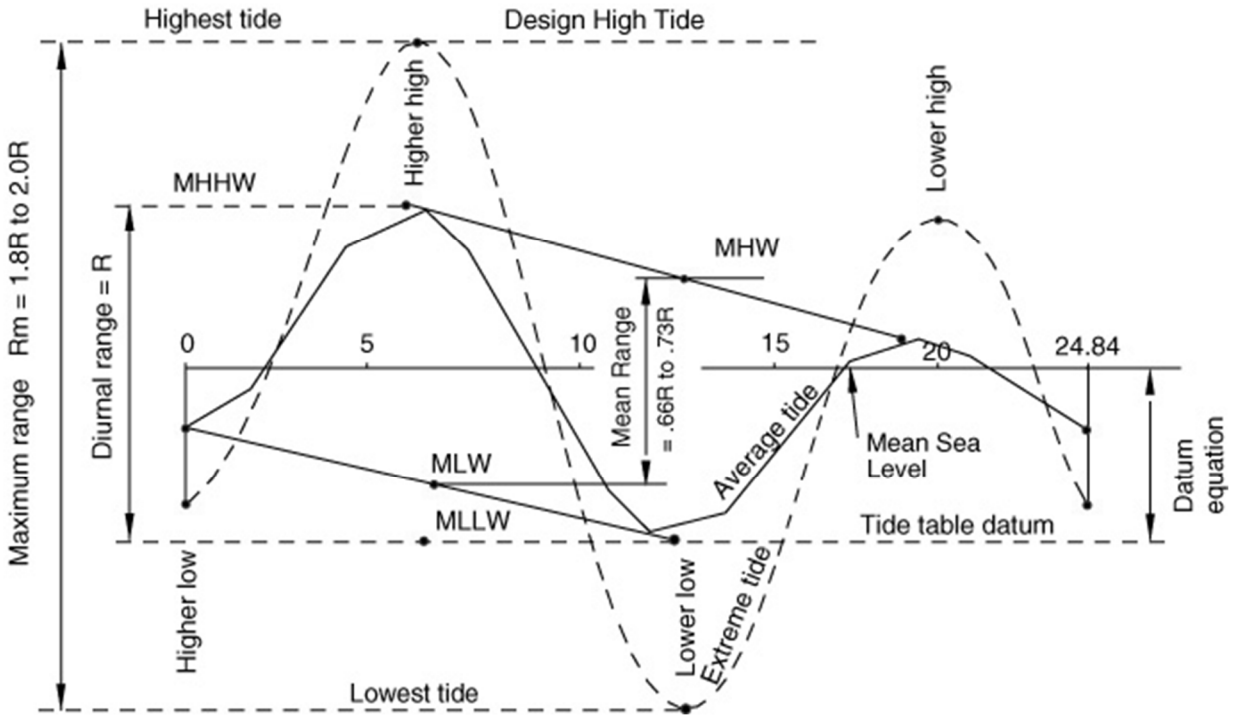
The average of all highs (indicated graphically as the mean of higher high and lower high) is the MHW, and of all the lows, MLW. Vertical difference between these two stages is the *mean range*.

See the Surveys Manual and the interim survey guidelines for Estimating Sea Level. For more information, see the Office of Land Surveys website.

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

Figure 883.2A

Nomenclature of Tidal Ranges



NOTES:

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

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

(2) Design Wave Heights.

(a) General. Even for the simplest of cases, the estimation of water levels caused by meteorological conditions is complex. For preliminary design or low risk coastal design, simplified techniques as described below may be used. Project designs with medium to high risk should use computer models as described in HEC 25, Index 5.4 and the Design Manual for Hybrid Coastal Protection Strategies. Simplified techniques may be used to predict acceptable wind wave heights for the design of highway protection facilities along the shores of embayments, inland lakes, and reservoirs. The Coastal Engineering Manual provides a simplified wave prediction method which is suitable for most riprap sizing applications.

The method is described in HEC 23, Volume 2, Index 17.2.2 of Design Guideline 17. It is recommended that for ocean shore protection designs the assistance of FHWA or a certified coastal engineer be requested.

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

(b) Wave Distribution Predictions. Wave prediction is called hindcasting when based on past meteorological conditions and forecasting when based on predicted conditions. The same procedures are used for hindcasting and forecasting. The only difference is the source of the meteorological data. Reference is made to the Army Corps of Engineers, Coastal Engineering Manual – Part II, and the Design Manual for Hybrid Coastal Protection Strategies 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. Some Boussinesq wave models also contain capabilities to estimate wave heights generate from boats.

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

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

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

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

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

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

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

(1) Hindcasting – The U.S. Army Corp of Engineers has historical records of onshore and offshore weather and wave observations for most of the California coastline. Design wave height predictions for coastal shore protection facilities should be made using this information and hindcasting methods.

Deep-water ocean wave characteristics derived from offshore data analysis may need to be transformed to the project site by refraction and diffraction techniques. As mentioned previously, it is strongly advised that the Corps technical expertise be obtained so that the data are properly interpreted and used.

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

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

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

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

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

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

An initial estimate of wind generated significant wave heights can be made by using Figure 883.2B. If the estimated wave height from the nomogram is greater than 2 feet, the procedure may need to be refined. Refer to the Design Manual for Hybrid Coastal Protection Strategies or HEC-25 for recommended calculation methods for larger wave heights. It is recommended that advice from the FHWA or a coastal engineer be obtained to refine significant wave heights, Hs, greater than 2 feet.

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

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

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

(f) Wave Run-up. Run-up is the extent, measured vertically, that an incoming wave will rise on a structure. An estimate of wave run-up, in addition to design wave height, will typically be needed and is required by policy for projects subject to California Coastal Commission (CCC) jurisdiction Techniques for calculating wave runup on permeable and impermeable structures can be found in Hydraulic Response and Armor Layer Stability on Coastal Structures (2015) See the University of Delaware's Center for Applied Coastal Research CACR Reports website at:

https://coastal.udel.edu/research-2/cacr-reports/.

Additional guidance on procedures for estimating wave run-up for rough surfaces (e.g., RSP) are contained in the U.S. Army Corps of Engineers manual, Design of Coastal Revetments, Seawalls, and Bulkheads, (EM 1110-2-1614) published in 1995.

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

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

(g) Littoral Processes. See Index 882.3(2). Littoral processes result from the interaction of winds, waves, currents, tides, and the availability of sediment. The rates at which sediment is supplied to and removed from the shore may cause excessive accretion

Determining Design Wave

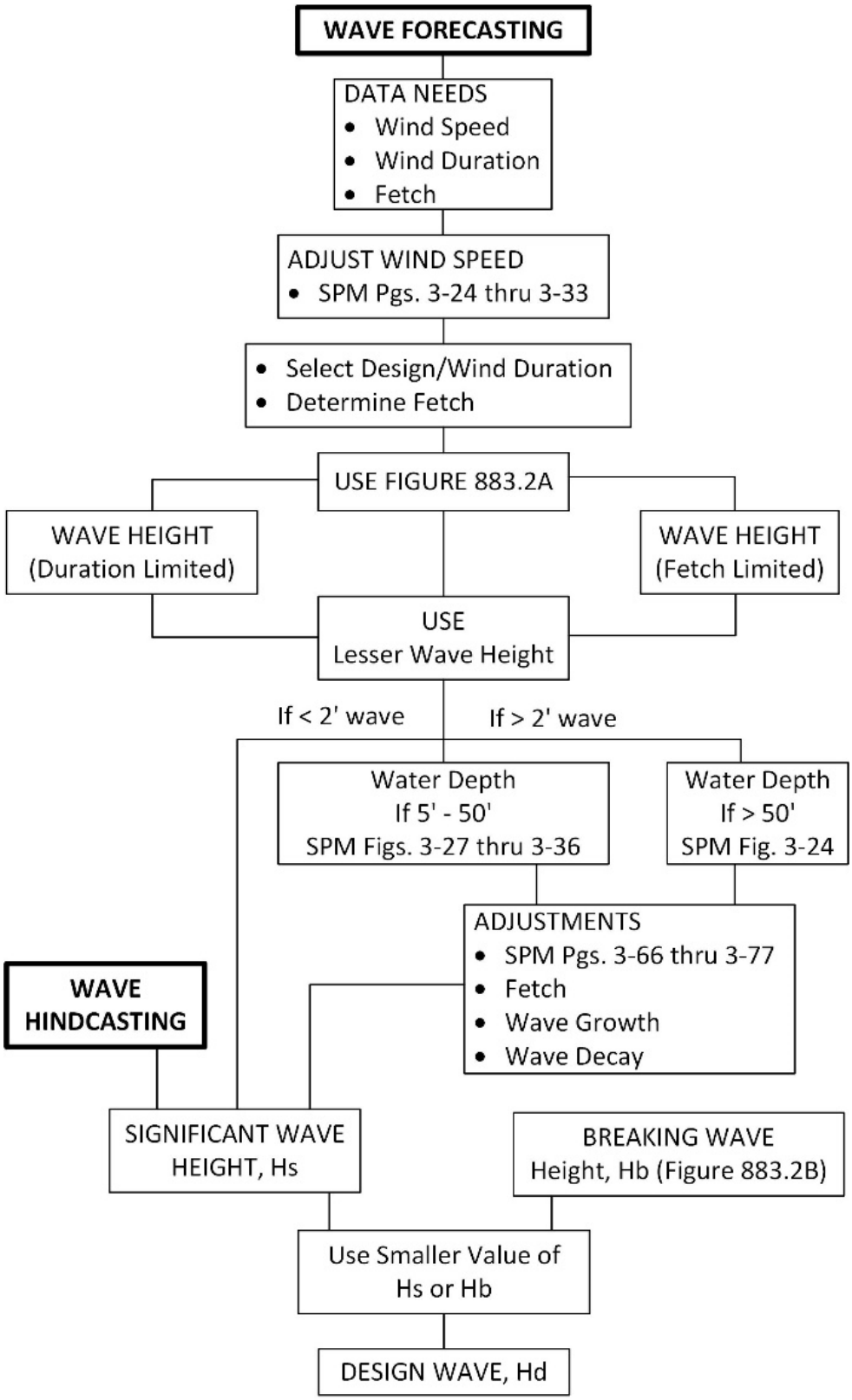


Figure 883.2B

Significant Wave Height Prediction Nomograph

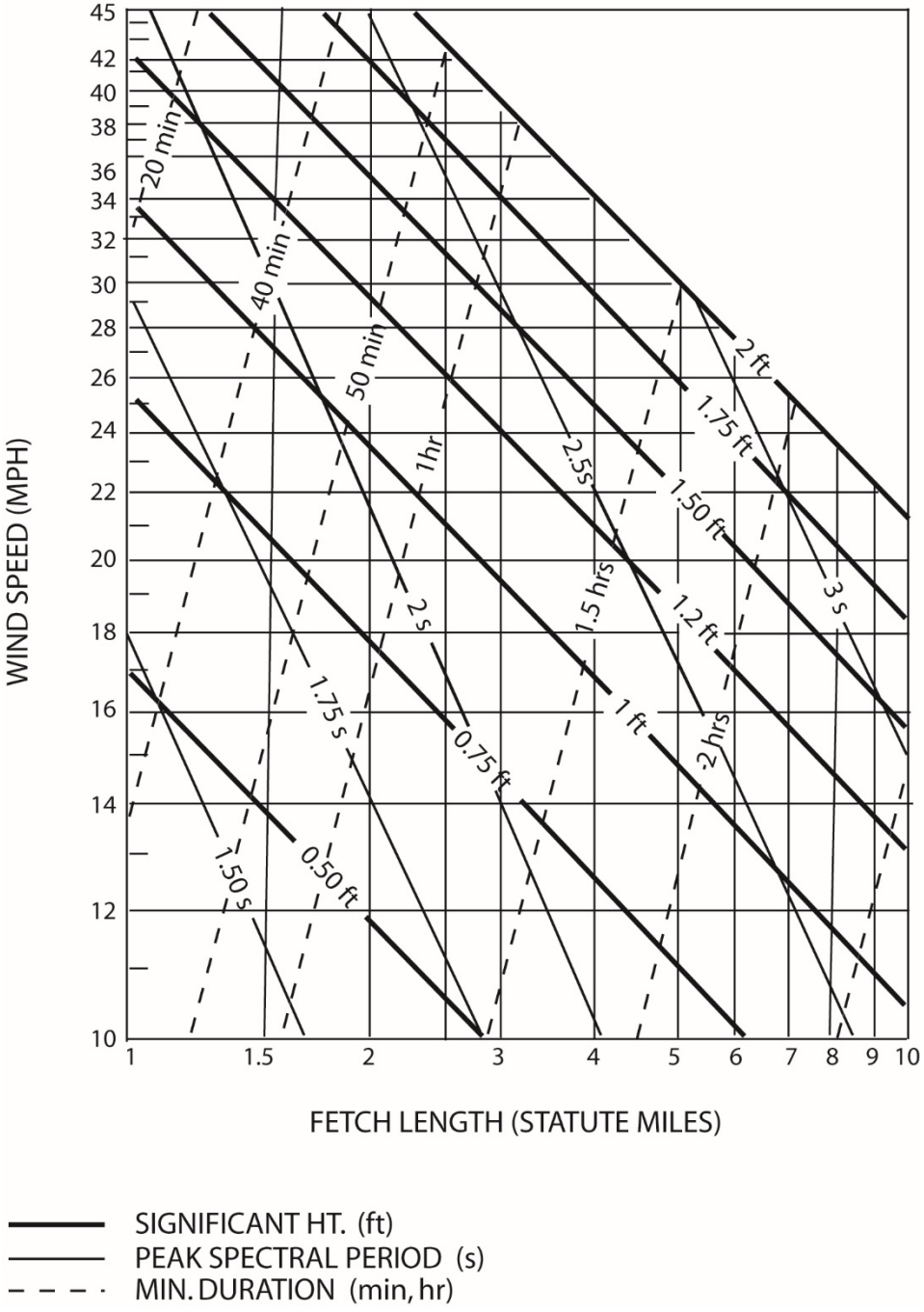
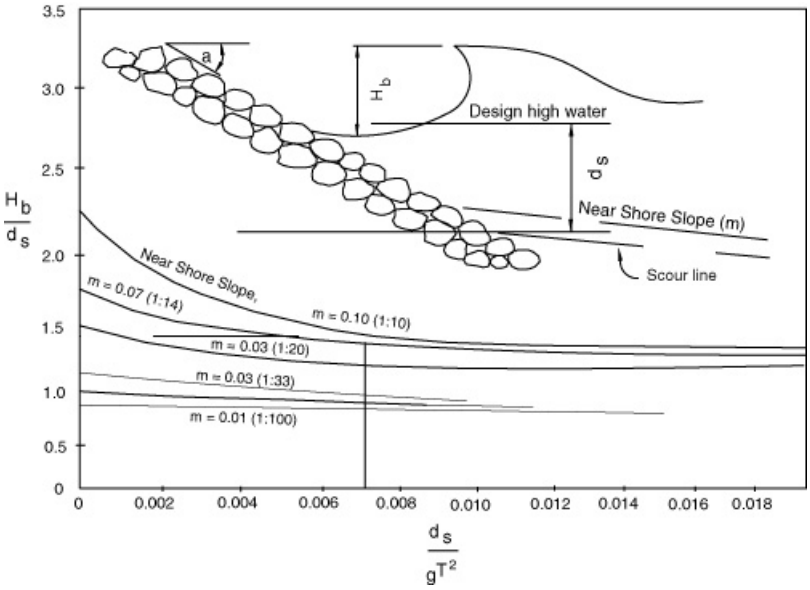


Figure 883.2C

Design Breaker Wave



Example:

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

Solution:

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

From Graph - $H_b/d_s = 1.5$

$H_b = 2 \times 1.5 = 3.0$ 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 3.0 feet.

T = Wave Period (SPM)

or erosion that can affect the structural integrity of shore protection structures or functional usefulness of a beach. The aim of good shore protection design is to maintain a stable shoreline where the volume of sediment supplied to the shore balances that which is removed. Designers interested in a more complete discussion on littoral processes should consult the U.S. Army Corps of Engineers' Coastal Engineering Manual (CEM) – Part III.

(3) *Sea Level Rise.* The California Ocean Protection Council (OPC) has developed sea-level rise guidance for use by state and local governments to assess the associated risks with sea-level rise and incorporate sea-level rise into planning, permitting and investment decisions. The "State of California Sea-Level Rise Guidance 2024 Science and Policy Update" provides estimates of sea-level rise based upon the best available science. The sea level rise scenarios are derived from probabilistic projections developed in the Intergovernmental Panel on Climate Change Sixth Assessment report (IPCC AR6). A step by step approach to selecting a value for sea-

level rise based on OPC 2024 guidance is provided in the steps below. More detail on each step is available in the 2024 OPC guidance. This method of evaluating sea-level rise could be revised and updated in the future based on the most current guidance provided by OPC or other responsible agencies. Routine maintenance projects are exempt from Sea Level Rise analysis requirements (i.e., pavement replacement, culvert replacement, etc.).

Step 1. Identify the nearest tide gauge. The rates of sea-level rise along the California coast is dependent on land elevations resulting from tectonic activity as well as land subsidence. There are 14 active tide gauges along the California coast and sea-level rise projections vary across the tide gauges based on trends in tectonic activity and land subsidence. The tide gauges incorporate the localized effects of vertical land motion.

Identify the tide gauge nearest to the project site. If the project is located equidistant between two tide gauges it would be appropriate to interpolate between the two gauges or average the two gauges. The 14 tide gauges along the California coast are identified in Figure 883.2 D.

Step 2. Evaluate planning and/or project time horizon(s). Determine the project lifespan (design life) for selection of appropriate year for associated sea-level rise. The California Transportation Commission has adopted asset classes associated with the State Highway System and the Primary Asset Classes are defined as: (a) Pavement, (b) Bridges, (c) Drainage, (d) Transportation Management Systems, and (e) Supplementary Assets. In the absence of a designated project lifespan, the design life associated with an asset class should be used to determine the year associated with the projected sea-level rise. Design lives of pavement projects are referenced in Section 612, and maintenance free service life of culverts (typically 50-years) referenced in chapter 850 of this manual. Bridge Design Life (per AASHTO LRFD Bridge Design Specifications 9th Edition Sec. 1.2) is 75 years.

Lifespan (design life) will help focus the decision of the appropriate sea level rise scenario. The OPC 2024 update shows a greater certainty in the sea level rise expected in the next 30 years compared with the sea level rise projections in the 2018 OPC guidance. In the mid-term (2050-2100) there is a large range of sea level rise as there is greater uncertainty in the projected future warming from different emission pathways and certain physical processes such as rapid ice sheet melt. The range of sea level rise over the long term (2100 and beyond) becomes increasingly large due to uncertainties associated with physical processes. Once the design life of the project location is complete, reevaluate at that time using the best available science.

Step 3. Choose multiple Sea Level Rise Scenarios for vulnerability assessment. Vulnerability of people, communities, natural resources, infrastructure and properties should be considered for developing a range of sea-level rise projections. There are five Sea Level Rise scenarios: Low, Intermediate-Low, Intermediate, Intermediate-High, and High. The H++ Scenario (from previous guidance) has been determined as not physically plausible and is not included in the 2024 OPC report. The Low Scenario is the lower bound for the plausible sea level rise in 2100 with aggressive emission reduction scenarios. The Intermediate-Low Scenario is the reasonable lower bound for the most likely range of sea level rise by 2100. With this scenario, low confidence processes such as ice sheet melt are not included. The Intermediate Scenario is driven dominantly by high emission scenarios and is a reasonable estimate of the upper bound of most likely sea level rise in 2100. Low confidence processes contribute about 25% of the pathways for reaching this scenario. The Intermediate-High Scenario reflects intermediate to high future emissions and high warming where rapid ice sheet loss processes (low confidence processes) are contributing to sea level rise. The High Scenario reflects both high emissions and low confidence processes. This scenario relies on large contributions from rapid ice-sheet loss and processes where there is low confidence in their understanding.

For exceedance probabilities of each scenario see Table 2.2 in the 2024 OPC guidance document.

FHWA and others recommend looking at multiple climate change scenarios to determine the appropriate scenario for the project or asset. Consideration should be given to risk, the criticality of the asset, stranded assets, adaptive capacity, cost and other economic, engineering, social and environmental concerns. As there is uncertainty in sea level rise projections and climate change scenarios, it can be practical to choose a lower sea level rise scenario for design and develop an adaptation plan for when and if the higher scenario occurs. Adaptation plans allow for flexibility in design and address uncertainties in sea level rise projections. Jurisdictional agencies (such as the California Coastal Commission) may require consideration of sea-level rise projections under a High Scenario; however project design may not necessarily include incorporation of the highest value of sea-level rise selected. Factors such as project costs and feasibility, may require a negotiated agreement with the agencies to develop a modular approach to design using a value associate with a shorter time frame than the selected design life of the project with the understanding that successive projects over time would build upon the proposed design to ultimately provide a resilient infrastructure.

Figure 883.2D

California 14 Tide Gauges

For low risk facilities, using lower sea level rise scenarios such as Low or Intermediate-Low is warranted. Examples may include a parking lot within the coastal area, or a constructed trail leading down towards a beach. Should such assets be damaged or destroyed, they may be relatively easy to repair or replace.

Intermediate-Low or Intermediate sea level rise scenarios may be considered for medium risk projects. Such risk may be exercised for a segment of roadway that if inaccessible would not jeopardize public safety or public health. Additionally, such a risk may be adopted if an asset would be cost effective to repair/replace as opposed to major resiliency redesign, and whose inaccessibility would not negatively impact natural resources or properties. Another example may be culvert outfalls that may tend to be inundated by sea-level rise on a coastal highway. The Intermediate-Low or Intermediate Scenario may be assumed if a contingency plan exists to retrofit the culvert outfalls with tidal flap gates to prevent backflow.

Intermediate or Intermediate-High Scenarios may be considered when an asset is expected to be needed for public health and safety. The likelihood that sea-level rise may meet or exceed this value is low. A highway expected to be used as an emergency evacuation route for people/communities, as access to and from hospitals, as a major route for support of local/regional economies, may be evaluated for sea-level rise under the Intermediate-High Scenario.

High risk projects can consider the Intermediate-High or High Scenarios and may be used for projects that have little to no adaptive capacity, that are essential for public safety and health, that is cost prohibitive to replace or repair, and with a design life well beyond 2050. An example would be a major bridge connecting communities with access to hospitals and economic interests and spanning a water body directly impacted by sea-level rise, and where freeboard requirements are necessary for passage of ships, boats or other crafts. Such situations with project design lives extending into the 22nd century where a minimum freeboard is required for passage of watercraft may require consideration of the High Scenario.

Sea-level rise projections for each tide gauge are provided in the "State of California Sea Level Rise Guidance 2024 Update" Appendix 1 and may be accessed at:

https://opc.ca.gov/wp-content/uploads/2024/05/Item-4-Exhibit-A-Final-Draft-Sea-Level-Rise-Guidance-Update-2024-508.pdf

Step 4. Conduct vulnerability assessment. Evaluate impacts from projected sea level rise scenarios, including exposure, sensitivity, and adaptive capacity. This step includes looking at exposure maps of sea level rise inundation and flooding, erosion, and groundwater considerations. It is important to consider impacts to the asset. Adaptive capacity is the ability of an asset to evolve in response to or cope with the impacts of sea level rise. Evaluate potential impacts and adaptive capacity across a range of sea-level rise projections. Evaluate the potential impacts of sea-level rise on the project in terms of vulnerable communities, critical infrastructure, and economic burden.

Step 5. Explore adaption options and feasibility: Explore project-specific adaptation options and possibly a cost-benefit analysis considering physical, economic, environmental, social and legal constraints. Evaluate impacts of sea-level rise by using sea-level rise mapping tools (sea-level rise) viewer available at:

https://coast.noaa.gov/slr/#/layer/slr/0/13566681.667176013/4585243.78640795/9/satellite/none/0.8/2050/interHigh/midAccretion

NOAA's sea-level rise viewer evaluates the impacts of sea-level rise at water surface elevations derived from adding the selected value of sea-level rise to MHHW elevation of the sea in the vicinity of the project. MHHW values for various stations may be obtained from:

https://tidesandcurrents.noaa.gov/stations.html?type=Datums

Select appropriate station and datum from website. Add selected value of sea-level rise to MHHW to obtain water surface elevation for design.

Step 6: Select phased adaptation approach and/or implement project. Select sea-level rise projections based on risk tolerance and incorporate appropriate resiliency into design. Adaptation plans may be included in case sea-level rise exceeds design projections. Consider risk, budget, regulatory constraints, environmental and community impacts and stakeholder input. An example for selection of sea-level rise for two hypothetical projects near Crescent City, Del Norte County is provided below.

Example Project # 1 **:** An HM pavement project is to be constructed south of Crescent City. The project will include cold planing and repaving the existing highway. Sea level rise considerations do not need to be evaluated on this project as it is a routine maintenance project.

Example Project # 2 : A segment of SR 101 is to be reconstructed south of Crescent City. A parking lot for access to the beach is also included in the scope of the project as shown in Figure 883.2E.

Assumed project scope includes reconstruction of segment of SR-101 south of Crescent City. A parking lot is to be constructed for beach access for recreational purposes. Consideration of sea-level rise for proposed project is as follows:

The nearest tide gauge is Crescent City. The data for sea-level rise at Crescent City is provided in Table 883.1B. Per HDM Index 612.2, for roadside facilities such as a parking lot, 20 year design life may be used, while the pavement design life for new construction or reconstruction projects should be 40 years. Applicable sea-level rise for the parking lot will be for year 2040 (20 years from the project design date). Applicable sea-level rise for roadway reconstruction will be for year 2060 (40 years from the project design date). Districts have flooding records which can be used to determine if sea level rise or flooding is an issue that needs to be addressed.

Consider range of sea-level rise for varying risk and scenarios. For the parking lot, sea-level rise for projects prior to 2050 are described as near term and show much greater certainty in the amount of sea level rise expected. Refer to Table 883.1B for information:

- Sea-level rise associated with a Low Scenario for year 2040 is 0.1 feet.
- Sea-level rise associated with an Intermediate-Low Scenario for the year 2040 is 0.2 feet.
- Sea-level rise associated with the Intermediate Scenario for the year 2040 is also 0.2 feet as there is not much difference in the near term for different sea level rise scenarios.

Figure 883.2E

Crescent City Example

Now evaluate the impact of the potential loss of the parking lot.

- The loss of the parking lot is not expected to have a significant impact on public health and safety. The loss would be expected to have an insignificant economical impact on any community.
- When there is no significant economical loss, no threat to public safety, public health or transportation resulting from the loss of the parking lot, an evaluation of the costs of construction, repair and replacement should determine the risk factor to be adopted for selection of an appropriate value for sea-level rise.
- Although sea-level rise associated with a Low scenario may be justified, costs of construction, future right of way issues, future repair or replacement should be examined. For the parking lot the differences between sea-level rise values associated with the Low scenario, Intermediate-Low Scenario and the Intermediate Scenario is very small (ranges from just over 1 inch to almost 2.5 inches) and based on costs an appropriate scenario may be selected.

For highway reconstruction, sea-level rise projections for projects with design life extending beyond 2050 fall under the mid-term range where the differences become increasingly large and are more closely associated with potential future greenhouse gas emissions. With a 40-year design life for pavement reconstruction projects sea-level rise for year 2060 may be considered. Refer to Table 883.1B for information. Review sea-level rise projections for multiple scenarios. The comparisons for year 2060 are provided Table 883.1A.

Table 883.1A

Crescent City Example Comparison for 2060

Low Scenario	Intermediate-Low Scenario	Intermediate Scenario
0.1 foot	0.4 foot	0.6 foot

Table 883.1B

Sea-Level Scenarios Crescent City

YEAR	LOW	INT-LOW	INTERMEDIATE	INT-HIGH	HIGH
2020	0.1	0.1	0.1	0.1	0.1
2030	0.1	0.1	0.2	0.2	0.2
2040	0.1	0.2	0.2	0.3	0.4
2050	0.1	0.3	0.4	0.6	0.8
2060	0.1	0.4	0.6	1.0	1.5
2070	0.2	0.4	0.8	1.6	2.3
2080	0.2	0.6	1.2	2.3	3.4
2090	0.2	0.7	1.7	3.0	4.5
2100	0.2	0.8	2.3	3.9	5.6
2110	0.2	0.9	2.9	4.7	6.9
2120	0.2	1.0	3.4	5.3	7.9
2130	0.2	1.2	3.8	5.8	8.7
2140	0.2	1.3	4.2	6.3	9.6
2150	0.2	1.4	4.7	6.8	10.3

Determine impact of potential loss of this segment of highway on communities:

- Will the loss of this segment impact transport of patients to and from a hospital?
- Will the loss of this segment impact response times for emergency vehicle?
- Will the loss of this segment impact freight and deliveries resulting in economic losses?
- Can traffic be detoured easily around this segment?

The value of sea-level rise for designing the roadway may be selected after evaluating relevant issues such as mentioned above.

If the highway segment is important for public safety, health and local economy, the Intermediate or Intermediate-High Scenario's value of 0.6 or 1.0 feet respectively, may be selected. The design would not only incorporate a higher elevation of the roadway but would also include measures for protecting the roadway (Armoring sea approach to roadway embankment).

Although the project does not have to be designed for the High Scenario, it may be considered. Sea-level rise associated with the High Scenario is 1.5 feet. A plan for future modular adaptation may be included should it become apparent at some time in future that sea levels are heading towards the High Scenario projections. The plan may include raising the profile of the highway and associated protection measures against the 1.5 feet projected sea-level rise.

Determine MHHW elevation from NOAA's Tides and Currents website for the Crescent City, CA Datum at:

https://tidesandcurrents.noaa.gov/datums.html?id=9419750.

Figure 883.2F shows the results.

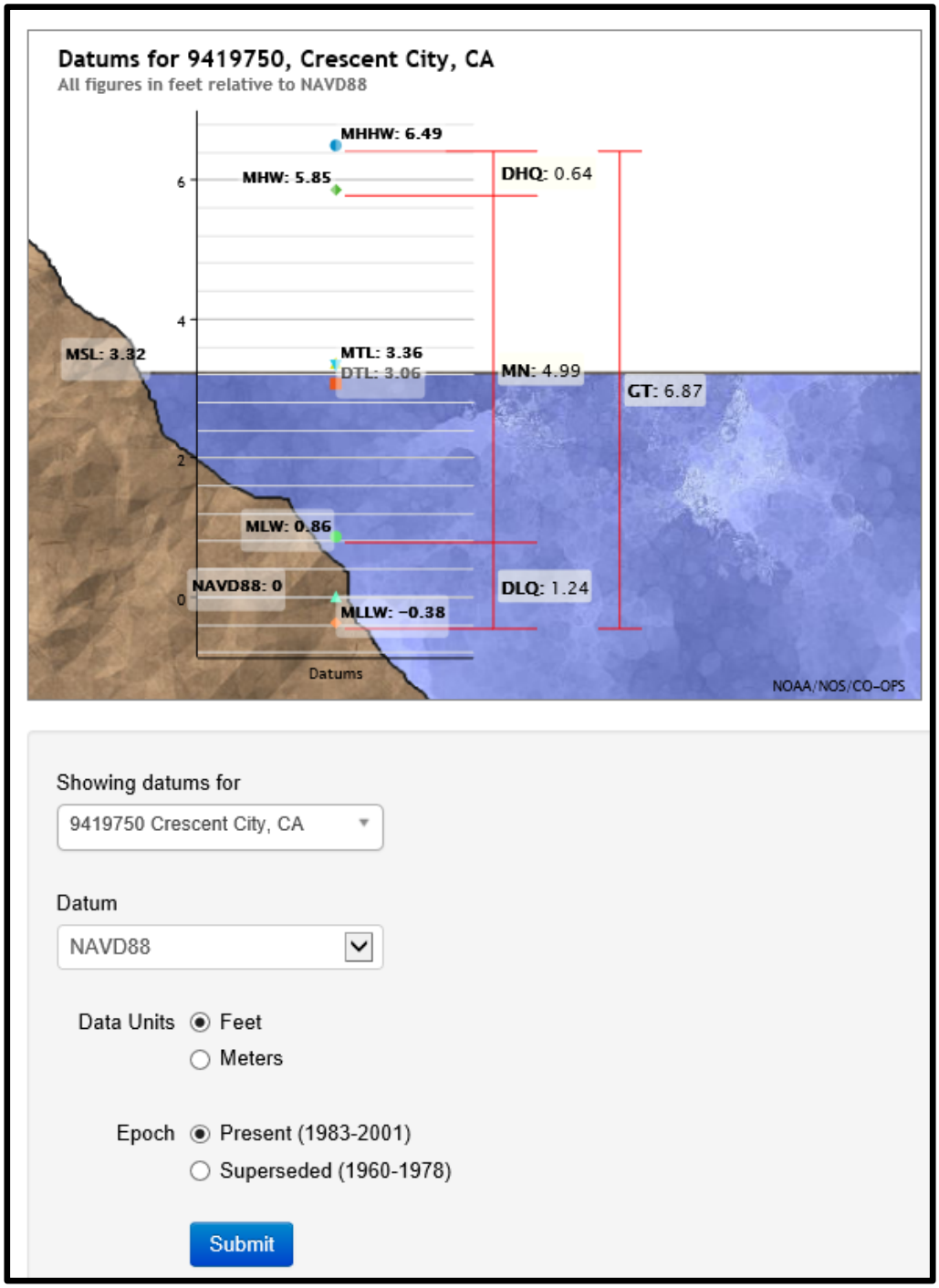
Add MHHW to projected sea-level rise:

For the roadway project add 1.0 feet to 6.87 feet. Elevation of water surface including sea level rise associated with the Intermediate-High Scenario is 7.87 feet. Evaluate impact of sea-level rise on project by using NOAA sea-level rise viewer at: https://coast.noaa.gov/slr/.

(4) Assessing Extreme Events and Climate Change. Chapter 14 of HEC 25, presents guidance on specific methodologies for assessing exposure of coastal transportation infrastructure to extreme events and climate change. For all projects, as a minimum, the use of existing data and resources should be utilized through the use of existing inundation (FEMA) or tsunami hazard maps to determine the exposure of infrastructure under selected sea (lake) level change scenarios, and sensitivity to depth-limited wave or wave runup processes. See HEC 25, Indexes 14.2 and 14.6.1 Level of Effort 1: Pacific Coast – Storms.

Figure 883.2F

Crescent City MHHW



883.3 Coastal Protection

(1) General. For Caltrans, coastal protection is divided into two categories; hard shoreline protection and nature-based hybrid coastal protection strategies. Nature-based hybrid strategies merge structural shore protection devices with nature-based coastal protection solutions. Executive Orders B-30-15 and N-82-20 obligate State of California agencies to prioritize and accelerate the use of natural infrastructure solutions. Armor is the artificial surfacing of shore or embankment to resist erosion or scour. Armor devices can be flexible (self-adjusting) or rigid. The distinction between revetments (layers of rock or concrete), seawalls, and bulkheads is one of functional purpose. Revetments usually consist of rock slope protection on the top of a sloped surface to protect the underlying soil. Seawalls are walls designed to protect against large wave forces. Bulkheads are designed primarily to retain the soil behind a vertical wall in locations with less wave action. Design issues such as tie-backs, depth of sheets are primarily controlled by geotechnical issues. The use of each one of the three types of coastal protection depends on the relationship between wave height and fetch (distance across the water body). Bulkheads are most common where fetches and wave heights are small. Seawalls are most common where fetches and wave heights are large. Revetments are often common in intermediate situations such as on bay or lake shorelines.

(2) Revetments.

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

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

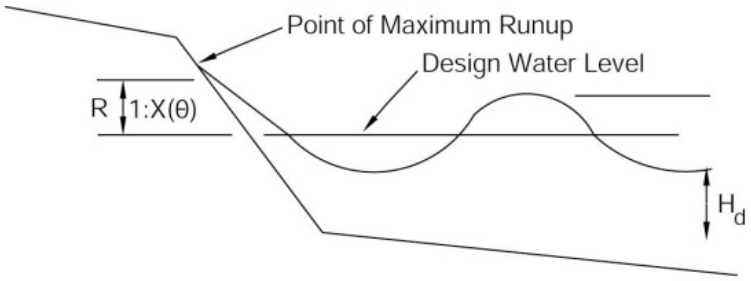
For RSP designs along coastal and lake shores, for wave heights five feet or less, the methodology presented in HEC 23, Volume 2, Design Guideline 17- Riprap Design for Wave Attack has been formally adopted by the Caltrans Bank and Shore Protection Committee. See also HDM Chapter 870, HEC-23 and HEC-25 for additional guidance on RSP. Section 72 of the Standard Specifications provides all construction and material specifications.

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

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

Figure 883.2G

Wave Run-up on Smooth Impermeable Slope



R = Wave Runup Height (ft)
H_d = Wave Height (ft)
θ = Bank Angle with the Horizontal

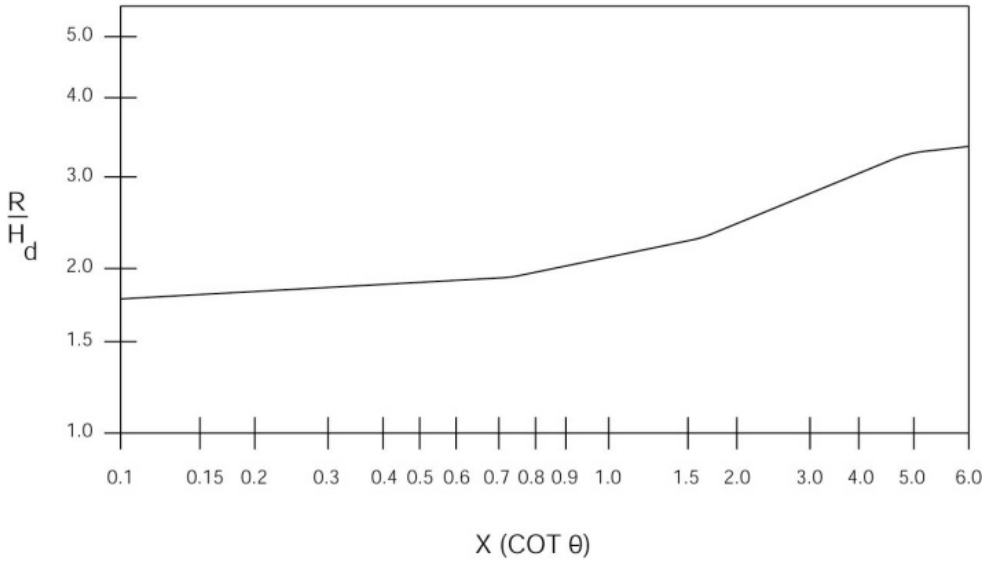


Figure 883.2H

RSP Lined Ocean Shore

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

- Depth at which the stones are founded (bottom of toe trench. See Figure 883.2I and Figure 17.2 in HEC 23, Volume 2, Design Guideline 17).
- Elevation at the top of protection.
- Rock size, specific gravity and section thickness.
- Need for geotextile or rock filter material.
- Face slope.

Well designed coastal rock slope protection should:

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

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

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

(a) The Hudson Method. Applications of Hudson's equation in situations with a design significant wave height of H=5 feet or less have performed well. This range of design wave heights encompasses many coastal revetments along highway embankments. When design wave heights get large and the design water depths get large, problems with the performance of rubble-mound structures can occur. A more conservative design approach should use a more conservative H statistic. The proper input wave height statistic is required and discussed in Section 7.3 of HEC 25. RSP with design wave heights much greater than H=5 feet require more judgment and more experience and input from a trained, experienced coastal engineer. Therefore, when design wave heights are much greater than H=5 feet, contact the District Hydraulic Engineer. The Hudson method considers wave height, riprap density, and slope of the bank or shoreline to compute a required weight of a median-size riprap particle.

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

Where:

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

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

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

K_d = empirical coefficient equal to 2.2 for riprap

S_r = specific gravity for riprap

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

θ = angle of slope inclination

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

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

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

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

Where:

ξ = dimensionless breaker parameter

θ = angle of slope inclination

L_o = wave height, (ft)

H_s = significant wave height, (ft)

T = wave period, (sec)

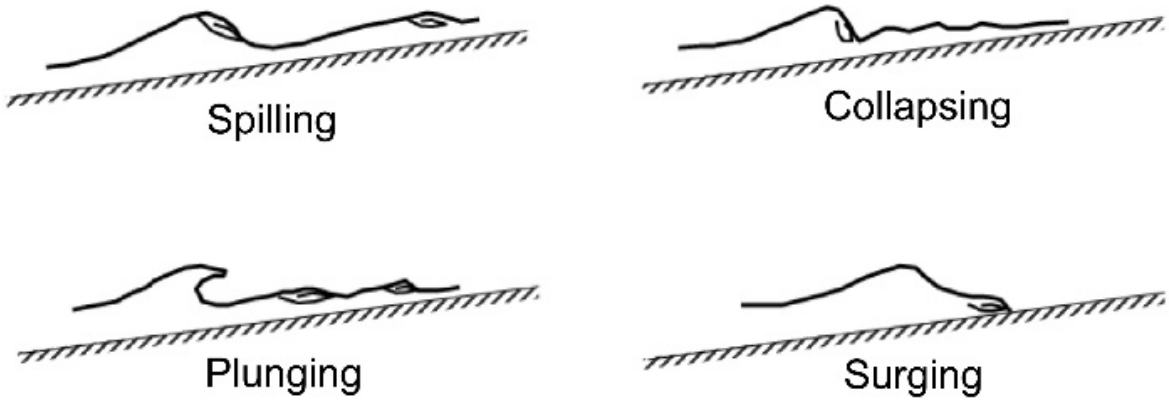
K_u = coefficient equal to 2.25 for wave height, (ft)

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

Table 883.2

Dimensionless Breaker Parameter and Wave Types

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



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

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

Where:

H_s = significant wave height, (ft)

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

D = armor size thickness, (ft)

ψ_u = stability upgrade factor (1.0 for good riprap)

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

θ = angle of slope inclination

ξ = dimensionless breaker parameter

b = exponent (0.5 for riprap)

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

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

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

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

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

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

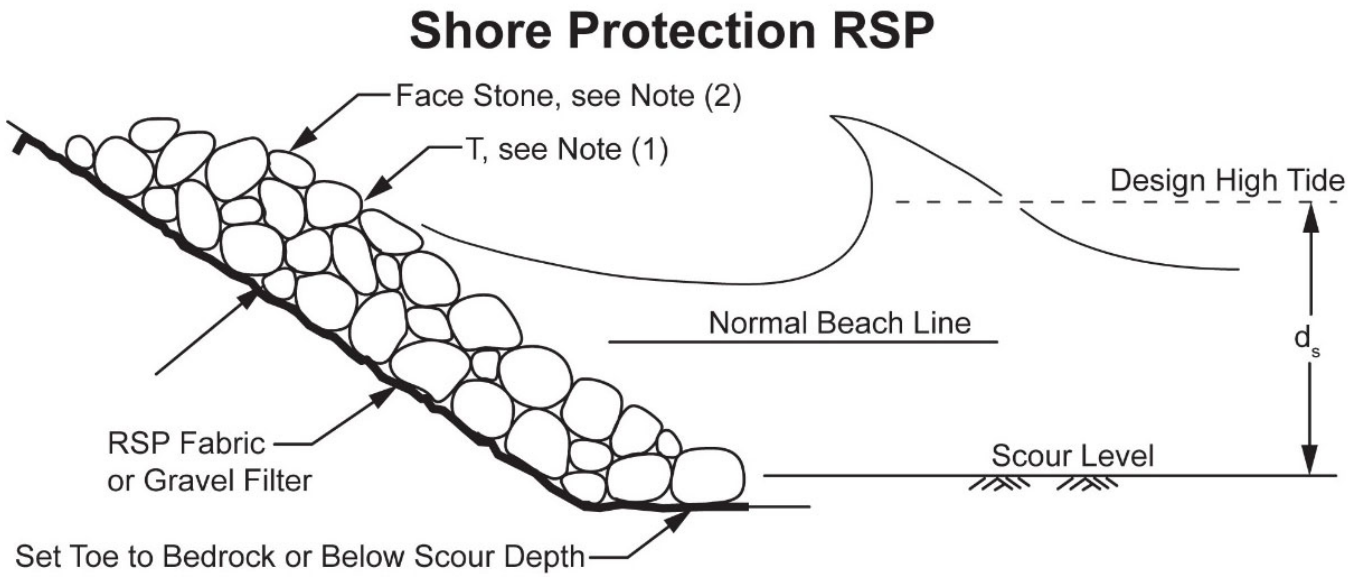
(3) Bulkheads. The bulkhead types are steep or vertical structures, like retaining walls, that support natural slopes or constructed embankments which include the following:

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

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

Figure 883.2l

Rock Slope Protection



NOTES:

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

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

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

(b) 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 counter forted 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.

(4) *Sea Walls.* Sea walls are structures, often concrete or stone, built along a portion of a coast to prevent erosion and other damage by wave action. Seawalls can be rigid structures or rubble-mound structures specifically designed to withstand large waves. Often they retain earth against the shoreward face. A seawall is typically more massive and capable of resisting greater wave forces than a bulkhead. Index 7.1 of HEC 25, provides several examples of seawall designs.

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

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

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

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

As stated in HEC-25, groins were probably the most common shoreline stabilization technique in the first half of the 20th century. However, they are much less acceptable today due to their potential downdrift negative impacts. Such negative impacts can be mitigated through use of modern coastal engineering principles to appropriately address downdrift impacts when used in combination as a hybrid solution. Refer to Design Manual for Hybrid Coastal Protection Strategies for detailed groin design guidance.

(6) *Nature-Based Hybrid Coastal Protection.* This type of coastal protection incorporates soft nature-based techniques with traditional hard protection methods. The soft engineering measures provide an environmentally friendly component that can enhance ecological connectivity, while the hard measures provide resistance and protection against the effects of extreme tidal and storm events as well as sea level rise. The use of nature-based hybrid protection strategies will satisfy directions presented in California Executive Orders B-30-15 and N-82-20 to consider climate change adaptation strategies and promote coastal resiliency through the use of natural infrastructure solutions.

Given the high energy, varying tidal events, and dynamic coastline of California, nature-base hybrid protection strategies are preferred over stand-alone nature-based strategies. The hard protection components is needed to protect the coastline and infrastructure from vulnerabilities during extreme events caused by sea level rise, wave runup and overtopping, king tide events, and tsunamis. While the soft components can provide attenuation of wave energy during smaller magnitude tidal and storm events, their bigger benefit is to improve ecology and habitat along the coastline.

Suitable soft components of nature-based hybrid protection for the California coastline are beach nourishment and dune construction. By placing site-specific beach sediment to replenish the sediment supply, the retreating shorelines in California can be reestablished and widened that will create a buffer zone and protect adjacent infrastructure from wave attack. Dune construction on the beach will also protect infrastructure from wave attack by physically attenuating the wave energy. If it is site-appropriate, dunes and bluffs can be planted to further improve the ecology and habitat.

Regarding hard component examples of a nature-based hybrid protection strategy, rock can be placed at the toe of a slope or a bluff along a beach at proper depth to resist scour of the toe and possible failure of a slope or bluff. The rock along the toe of slope can be hidden beneath beach sediment. Similarly, rock can be hidden in the inner core of a dune. The exterior shell of the dune would be comprised of beach sediment. If wave forces exceed the resistance capacity of the sediment outer shell, the interior rock can provide resistance and continue the wave energy attenuation to

protect the shoreline and adjacent infrastructure. This would prevent complete failure of the dune and allow for its continued function.

The nature-based hybrid protection adaptation strategy combines the positive benefit of ecological preservation with the positive benefit of extreme event protection to provide needed resiliency for the California coastline. Refer to the Caltrans *Design Manual for Hybrid Coastal Protection Strategies* for design and analysis guidance as well as maintenance considerations for nature-based hybrid coastal protection.