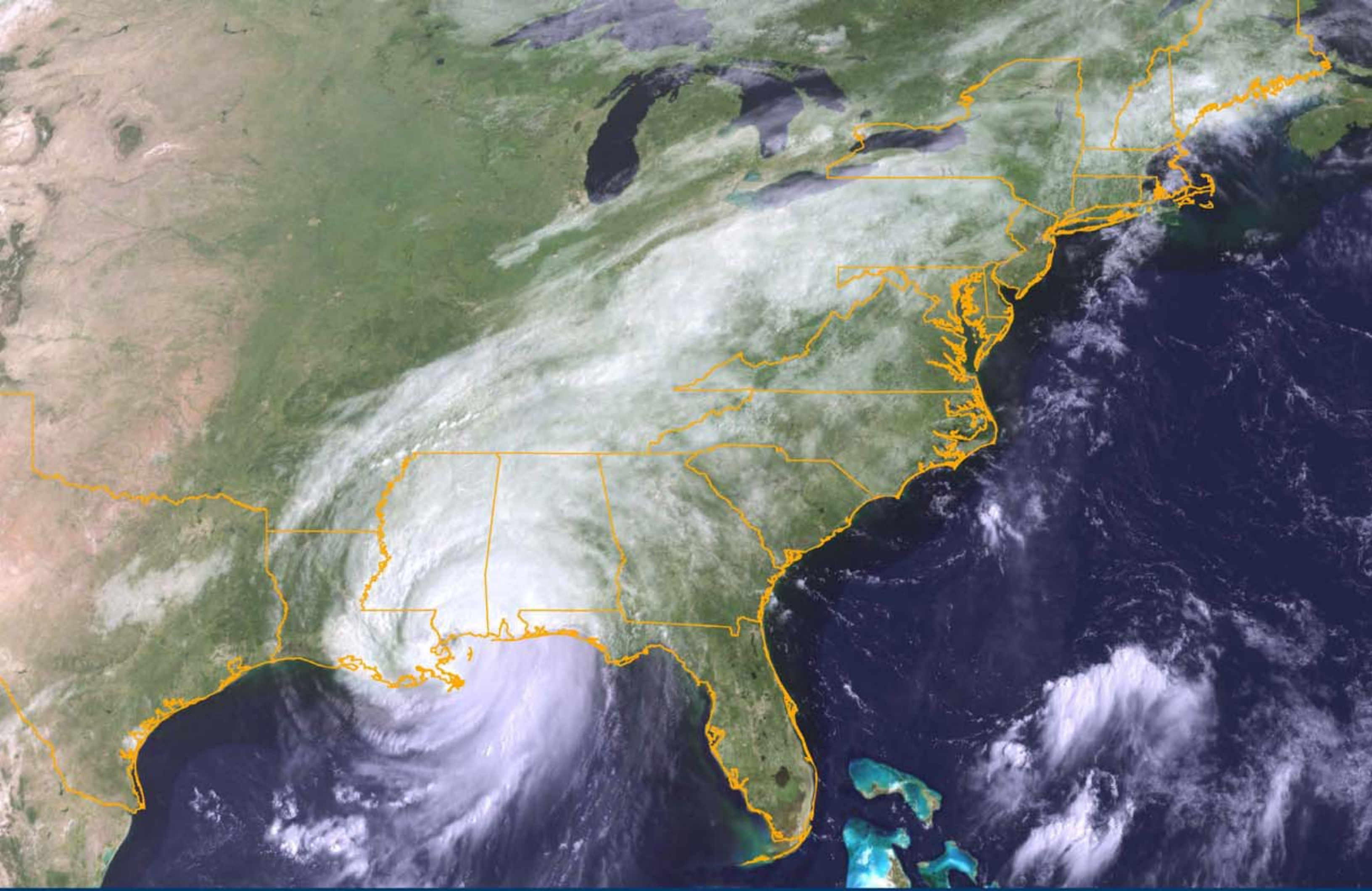

Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update

Final Draft

February 2007

FEMA

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PROJECT Summary

PS.1 Purpose of Study

The Federal Emergency Management Agency (FEMA) is responsible for preparing Federal Insurance Rate Maps (FIRMs) that delineate flood hazard zones and Base Flood Elevations (BFEs) in coastal areas of the United States. These areas are among the most densely populated and economically important areas in the nation.

FEMA guidance for coastal flood hazard mapping resides in Appendix D of the "Guidelines and Specifications for Flood Hazard Mapping Partners." A project to update the guidance for analyzing and mapping coastal flood hazards was initiated in 2003. The study consisted of three phases:

- Phase 1: to evaluate and report on the existing FEMA procedures for delineating coastal flood hazard areas in three major coastal regions of the United States (Atlantic, Gulf of Mexico, and Pacific)

- Phase 2: to develop recommended guidelines and procedures for mapping flood hazards on the Pacific coast, where only limited guidance was available previously

- Phase 3: to update coastal flood hazard mapping guidance for the Atlantic and Gulf of Mexico coasts

The purpose of this report is to complete the third phase of this project and implement as many of the short-term enhancements and needed revisions to the existing Appendix D guidance for the Atlantic and Gulf of Mexico Coasts, including the coastal methodology recommendations contained in FEMA Policy Memorandum No. 37 (PM 37), dated August 1, 2005.

This project was authorized cooperatively by FEMA Headquarters and FEMA Region VI, as the follow-up to previous phase 1 and 2 projects supported by FEMA Regions IX and X. The Phase 3 Project Coordinator was Gary Zimmerer, Project Engineer for FEMA Region VI. Michael Baker Jr., Inc, the National Service Provider to FEMA for Map Modernization, assumed the lead consultant and manager role for the project.

PS.2 Description of Needs for Atlantic And Gulf Coast Regions

Guidelines for the Atlantic and Gulf Coasts were assembled from elements developed over the course of many years, with the initial guidance established in 1989; however, no comprehensive assessment had been done to evaluate their effectiveness in hazard mapping since the last published effort in 1995. A comprehensive review of the existing guidelines was needed, in light of recent experience and new technology, and was recommended in the Phase 1 Summary Report. Procedures need to be modified or developed to incorporate experience from previous studies and appeals, information on actual damages, and post-storm verification data. In addition, the existing procedures needed review because recent research and new data has produced an improved understanding of ocean and coastal processes. The existing procedures included little guidance on the analysis of storm meteorology, storm surge, or wave setup. In addition, the guidelines may need to be expanded to address flood hazards in coastal areas not directly exposed to ocean swell and waves generated by distant weather conditions, such as bays and estuaries.

PS.3 Project Approach and Schedule

The Phase 3 project approach included a team of technical experts (Technical Working Group or TWG), which was assembled from the core group of Phase 1 and 2 experts. The Phase 3 TWG is composed of coastal experts from private industry, academic and research institutions, Federal agencies, Flood Insurance Study (FIS) contractors, map coordination contractors, and FEMA Headquarters and regional engineers. A list of the members is provided at the end of this summary. This group was organized to implement a collaborative approach to follow up on the recommendations of the Phase 1 Summary Report, to identify new needs and priorities for improved coastal flood hazard mapping procedures, to consider potential alternatives, and to develop recommendations based on a consensus among coastal experts.

The project schedule was established based on FEMA's targets for the Map Modernization Plan. The project approach recognized that improvements to the Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update would need to be prioritized to maintain the adopted schedule. Only limited improvements could be incorporated into Appendix D during Phase 3, since development and testing of more extensive improvements would require several years of technical study and research, and/or regulatory changes within the NFIP. The Phase 3 studies and report formulation were initiated at the first workshop in December 2005 after delays caused by Hurricane Katrina. A second workshop was held in May 2006, and a final peer review workshop in was held in August 2006. The Final Draft Guidelines for the Atlantic Ocean and Gulf of Mexico Coastal Update are appended to this Project Summary.

PS.4 Summary

These Guidelines offer insight and recommend methods to analyze hurricane and northeaster flood events in the Atlantic and Gulf Coastal regions in a reasonable and consistent way. However, they require technical judgment and experience in their application and do not generally offer a prescriptive technique that can be applied uniformly in all study areas. The Guidelines are intended to apply to a range of settings, but they cannot address all settings and conditions, due to the variability of the Atlantic and Gulf Coasts. They include some new methods that were developed over a one-year period by the TWG, but most guidelines are updates to existing methodologies. Methods were selected and developed through collaboration and consensus, but some of these methods had not yet been fully tested in FISs at the release date of this document. Therefore, the TWG recommends that the new methods and guidelines be thoroughly tested in a variety of settings.

Experience and judgment in coastal engineering is required in order to apply the procedures provided in the Atlantic Ocean and Gulf of Mexico Coastal Update. The Mapping Partner may determine that minor modifications or deviations from the Guidelines are necessary to adequately define the coastal flooding conditions and map flood hazard zones in specific areas.

In these cases, documentation of these differences is required as part of the intermediate and final study submittals.

The project approach relied heavily on the collaboration of TWG members within a compressed schedule. It is envisioned that the next phase of guidelines development for coastal flood hazards will be guided by advancements that are occurring in the coastal field due to the catastrophic events of the 2005 Hurricane Season. Advancements in current technology are being made at a rapid pace and these Guidelines need to be revisited in the future to incorporate these changes. In addition, the next phase of the guidelines development should include the long-term TWG recommendations for testing, extending, and refining the procedures referenced in the Phase 1 Summary Report. FEMA recognizes that the Guidelines are an evolving documentation of the study procedures for the NFIP and may not necessarily be consistent with study methodologies for other Federal or State agency purposes.

PS.5 Acknowledgements

FEMA gratefully acknowledges the significant effort, collaboration, and interaction of the members of the TWG to produce this highly technical work product. Members of the TWG are listed below in alphabetical order.

FEMA Project Coordinator

Gary Zimmerer
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Denton, TX

Technical Working Group

The following individuals (listed alphabetically) participated in the TWG to prepare focused studies, attend workshops, and prepare reporting for the Phase 1 Summary Report; some also participated in the preparation of the Final Draft Guidelines.

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Table of Contents

Table of Contents (cont.)

Table of Contents (cont.)

D.2 Coastal Flooding Analyses and Mapping: Atlantic and Gulf Coasts

This subsection of Appendix D provides guidance for coastal flood hazard analyses and mapping that is specific to the Atlantic and Gulf of Mexico (herein referred to as Gulf) Coasts of the United States, generally referred to as "guidelines." The procedures described in this subsection were developed by a Technical Working Group (TWG) assembled by the Department of Homeland Security's Federal Emergency Management Agency (FEMA) in November 2005. They are intended to provide guidance that is generally independent of other Appendix D subsections, and that is based on the specific physical processes that influence coastal flooding on the Atlantic and Gulf coasts.

This section focuses on the Atlantic coast from the Maine-Canada border to the southernmost reaches of the Florida, the Gulf coast from Florida to the Texas-Mexico border, and the Puerto Rico and US Virgin Island Coasts, as shown in Figure D.2-1. The Great Lakes and Pacific coastlines are specifically addressed in Sections D.3 and D.4, respectively. However, much of the guidance in Section D.2 may be considered applicable in those geographic areas, if it is supplemented with engineering judgment and methods to address geographically unique processes or settings.

The mapping of V zones under the National Flood Insurance Program (NFIP) began in the early 1970s. The objective was to identify hazardous coastal areas in a manner consistent with the original regulatory definition of coastal high hazard areas as an "area subject to high velocity waters, including but not limited to hurricane wave wash." The initial technical guidance for identifying V zones was provided in a June 1973 report by the U.S. Army Corps of Engineers (USACE), Galveston District, titled "General Guidelines for Identifying Coastal High Hazard Zones, Flood Insurance Study - Texas Gulf Coast Case Study" (USACE, 1973). The USACE report identified a breaking wave height of 3 feet as critical in terms of causing significant structural damage and illustrated procedures for mapping the limit of this 3-foot wave (V zone) in two distinct situations along the Texas coast: undeveloped areas and highly developed areas.

In June 1975, the USACE, Galveston District, issued a followup report entitled "Guidelines for Identifying Coastal High Hazard Zones," which maintained the basic recommendations contained in the 1973 report for identifying V zones in undeveloped and developed areas; however, the 1975 report also included guidance for determining effective fetch lengths, a technical discussion justifying the 3-foot wave height criterion for V zones, an abbreviated procedure for V-zone mapping in undeveloped areas, an expanded discussion of V-zone mapping in developed areas, and historical accounts of several severe storms that affected developed areas along the Atlantic and Gulf coasts.

Between 1975 and 1980, the Federal Government (U.S. Department of Housing and Urban Development until 1978 and FEMA thereafter) published Flood Insurance Rate Maps (FIRMs) with V zones for approximately 270 communities along the Atlantic and Gulf coasts using the USACE guidance for V-zone mapping. During this period, the procedures for determining and delineating V zones in developed areas differed among studies. At that time, the regulatory Base

(1-percent-annual-chance) Flood Elevations (BFEs), for both insurance and construction purposes, were the 1-percent-annual-chance stillwater elevations (SWELs), which consisted of the astronomical tide and storm surge caused by low atmospheric pressure and high winds. Although V zones were identified, the increase in water-surface elevation due to wave action was not included. The Federal Government recognized that this practice did not accurately represent the flooding hazard along the open coast, but an adequate method for estimating the effects of wave action, applicable to most coastal communities, was not readily available at the time.

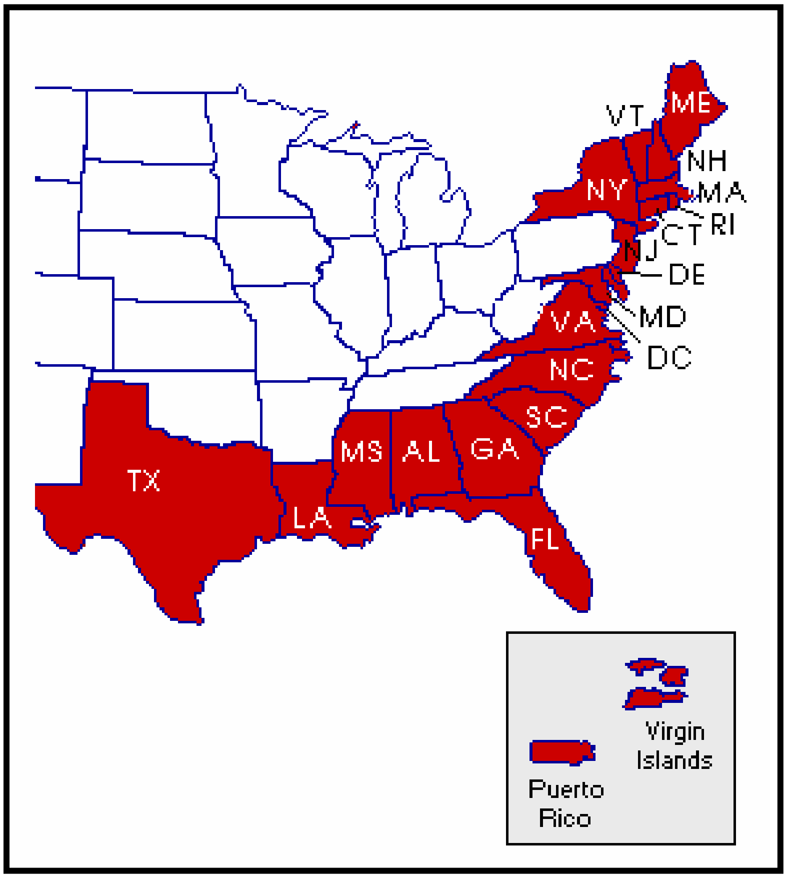

Figure D.2-1. Appendix D.2 Applicable Area – Atlantic and Gulf Coast Guidelines

In 1976, the National Academy of Sciences (NAS) was asked to provide recommendations about how calculations of wave height and runup should be incorporated in Flood Map Projects for Atlantic and Gulf coast communities to provide an estimate of the extent and height of stormwater inundation having specified recurrence intervals. The NAS concluded that the prediction of wave heights should be included in Flood Map Projects for coastal communities and provided a methodology for the open coast and shores of embayments and estuaries on the Atlantic and Gulf coasts. The report documenting the NAS findings, “Methodology for Calculating Wave Action Effects Associated with Storm Surges” (NAS, 1977), included means for taking into account varying fetch lengths, barriers to wave transmission, and regeneration of waves likely to occur over flooded land areas. NAS did not address the extent and elevation of wave runup, amount of barrier overtopping, and coastal erosion.

In 1979, FEMA adopted the NAS methodology. In 1980, FEMA issued "Users Manual for Wave Height Analysis," which was subsequently revised in February 1981 (FEMA, 1981). FEMA also introduced a computer program, Wave Height Analysis for Flood Insurance Studies (WHAFIS), in 1980. With WHAFIS, FEMA initiated a large effort to incorporate the effects of wave action on the FIRMs for communities along the Atlantic and Gulf coasts.

Along the coast of New England, with its very steep shore, the NAS methodology proved to be insufficient. Structures that were shown as being outside of the Special Flood Hazard Area on effective FIRMs experienced considerable wave damage from storms, most notably the northeaster of February 1978, a near 1-percent-annual-chance flood event. The need to account for the effects of wave runup was recognized. In 1981, FEMA approved a methodology that determined the height of wave runup landward of the stillwater line (Stone & Webster Engineering Corporation, 1981).

Two additions were made to the NAS methodology in 1984 to account for coastal situations involving either marsh grass or muddy bottoms. The NAS methodology did not account for flexible vegetation; in particular, marsh plants. Experts surmised that the motion of submerged marsh plants absorbed wave energy, reducing wave heights. In 1984, a FEMA task force examined this phenomenon in detail and developed a methodology that adjusted the wave height to reflect energy changes resulting from the flexure of various types of marsh plants and the wind, water, and plant interaction (FEMA, 1984). FEMA incorporated the new methodology into WHAFIS.

In 1987, FEMA modified its computer model for runup elevations slightly to increase the convenience of preparing input conditions. In 1990, FEMA modified the model again to improve computational procedures and application instructions to conform to the best available guidance on wave runup (Dewberry & Davis, 1990).

The muddy bottom situation occurs only at the Mississippi River Delta in the United States. The Mississippi River has deposited millions of tons of fine sediments into the Gulf of Mexico to form a soft mud bottom in contrast to the typical sand bottom of most coastal areas. This plastic, viscous bottom deforms under the action of surface waves. This wave-like reaction of the bottom absorbs energy from the surface waves, thus reducing the surface wave heights. A methodology was developed for FEMA to calculate the wave energy losses due to muddy bottoms (Suhayda, 1984). Waves in the nearshore areas are tracked over the mud bottom, resulting in lower incident wave heights at the shoreline. This is a phenomenon unique to the Mississippi River Delta, and FEMA has not incorporated the methodology into WHAFIS.

In 1988, FEMA upgraded WHAFIS to incorporate revised wave forecasting methodologies described in the 1984 edition of the "USACE Shore Protection Manual" (USACE, 1984) and to compute an appropriately gradual increase or decrease of SWELs between two given values (FEMA, September 1988).

In the performance of wave height analyses and the preparation of Flood Map Projects, erosion considerations were left to the judgment of FEMA contractors. Coastal erosion was to be considered a hazard when there was historical evidence of erosion from previous storms, but before 1986 objective procedures for treating erosion were not available. Consequently, some shorefront dunes were designated as stable barriers to flooding and some were not. In 1986, FEMA initiated studies aimed at providing improved erosion assessments in Flood Map Projects for coastal communities.

In response to criticisms that indicated a significant underestimation of the extent of Coastal High Hazard Areas, FEMA undertook an investigation to reevaluate V zone identification and mapping procedures. The resulting report, titled "Assessment of Current Procedures Used for the Identification of Coastal High Hazard Areas (V Zones)" (FEMA, 1986), presented a number of recommendations that allowed a more realistic delineation of V zones and better fulfilled the NFIP objectives, namely, actuarial soundness and prudent floodplain development. One recommendation called for full consideration of storm-induced erosion and wave runup in determining BFEs and mapping V zones.

As part of its investigation, FEMA performed a study of historical cases of notable dune erosion. In this quantitative analysis, field data for 30 events (later increased to 38 events) yielded a relationship of erosion volume to storm intensity as measured by flood recurrence interval. For the 1-percent-annual-chance storm, FEMA determined that, to prevent dune breaching or removal, an average cross-sectional area of 540 square feet is required above the SWEL and seaward of the dune crest. That standard for dune cross section has a central role in erosion assessment procedures on the Atlantic and Gulf coasts.

The USACE Coastal Engineering Research Center (CERC) performed a study of the available quantitative erosion models for FEMA (CERC, 1987). CERC determined that only empirically based models produce reasonable results with a minimum of effort and input data, that each available model for simple dune retreat has certain limitations, and that dune overwash processes are poorly documented and unquantified. After further investigations, FEMA decided to employ a set of simplified procedures for objective erosion assessment (FEMA, November 1988). These procedures have a direct basis in documented effects due to extreme storms and are judged appropriate for treating dune erosion in Flood Map Projects for coastal communities.

As the official basis for treating flood hazards near coastal sand dunes, FEMA published new rules and definitions in the Federal Register that became effective on October 1, 1988. FEMA included the following revised definition in Section 59.1 of the NFIP regulations:

> *Coastal high hazard area* means an area of special flood hazard extending from offshore to the inland limit of a primary frontal dune along an open coast and any other area subject to high velocity wave action from storms or seismic sources.

FEMA also added a clarification of this matter, a definition of primary frontal sand dune, in Section 59.1:

> *Primary frontal dune* means a continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent

> to the beach and subject to erosion and overtopping from high tides and waves during major coastal storms. The inland limit of the primary frontal dune occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope.

FEMA also included a new section in Part 65 of the NFIP regulations, identifying a cross-sectional area of 540 square feet as the basic criterion to be used in evaluating whether a Primary Frontal Dune (PFD) will act as an effective barrier during the 1-percent-annual-chance flood. Another consideration is the documented historical performance of coastal sand dunes in extreme local storms.

In 1989, CERC completed a review for the NFIP regarding coastal structures as protection against the 1-percent-annual-chance flood and published Technical Report CERC 89-15, "Criteria for Evaluating Coastal Flood-Protection Structures"(CERC, 1989). Predictions of wave forces, wave overtopping, and wave transmission for commonly constructed coastal protection structures were among technical topics addressed in the CERC report. FEMA summarized the CERC 89-15 report for use in the NFIP in a 1990 memorandum, Criteria for Evaluating Coastal Flood Protection Structures for NFIP Purposes. The guidelines in this Appendix incorporate procedural criteria recommended by CERC for evaluating structural stability as presented in the 1990 memorandum.

In 2003, recognizing that coastal areas are among the most densely populated and economically important areas in the nation, FEMA created a TWG of Coastal Engineers and Scientists and authorized an evaluation of the existing FEMA procedures for delineating coastal flood hazard areas in three major coastal regions of the United States: Atlantic, Gulf, and Pacific. The final products of the TWG were included in "Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States," and the subsequent FEMA Procedure Memorandum No. 37, "Protocol for Atlantic and Gulf Coast Coastal Flood Insurance Studies in FY05," issued on August 1, 2005. Procedure Memorandum No. 37 presents revisions and modifications to existing protocols in Appendix D, "Guidelines for Coastal Flooding Analyses and Mapping," of FEMA's *Guidelines and Specifications for Flood Hazard Mapping Partners* (FEMA, 2003) for performing detailed coastal hazard assessments for communities along the Atlantic and Gulf coasts.

The developments presented in Procedure Memorandum No. 37 were determined by TWG during the Pacific Coast Study Guidance project and deemed applicable to the Atlantic and Gulf coasts. Many of the recommendations of the TWG for the Atlantic and Gulf coasts still require additional development and testing. Updates and new recommendations for tide gage analyses, coastal structures, storm meteorology, wave runup, wave setup, and other aspects of coastal flood hazard identification have been incorporated into Section D.2.

D.2.1 Atlantic and Gulf Coast Guidelines Overview

Section D.2 is organized to:

- Present background information (Section D.2.1);
- Provide guidance on selecting study methodologies (Section D.2.2);
- Provide a set of technical methods as potential tools to be used in various study settings (Sections D.2.3 to D.2.10);
- Provide guidance on flood hazard mapping (Section D.2.11);
- Provide guidance on study documentation (Section D.2.12); and
- Provide reference information (Sections D.2.13 to D.2.16).

Figure D.2.1-1 shows the general layout of the document. Because it is anticipated that few readers will use the guidance by reading sequentially from beginning to end, Subsection D.2.2 provides a framework for overall study methodologies that Mapping Partners can use to refer to more detailed analysis methods in subsequent subsections. In many cases, multiple methods are presented for analysis of a single coastal process. Often, coastal processes necessitate that the analysis begin offshore and proceed onshore to produce hazard zone designations for a coastal Flood Map Project. Subsection D.2.2 provides guidance on selecting analysis methods that are applicable to particular coastal settings and on linking the analysis of individual coastal processes together in a study methodology. In this sense, the document is organized with a set of general instructions in Subsection D.2.2, and a toolbox for selection of specific methods in Subsections D.2.3 to D.2.10. The appropriate tools must be selected based on study objectives, coastal exposure, geomorphic setting, and available data.

Coastal flooding on the Atlantic and Gulf coasts is a product of combined offshore, nearshore, and shoreline processes. The interrelationships of these processes are complex, and their relative effects vary significantly with coastal setting. These complexities present challenges in the determination of the base (1-percent-annual–chance) flood for FEMA hazard mapping purposes. The fundamental philosophy of this subsection is to provide a set of technical tools that can be selected and applied, as needed, to match specific site conditions and physical processes relevant to coastal flood hazards.

These guidelines offer insight and recommended methods to analyze complex Atlantic and Gulf coast flood processes in a reasonable way. However, they require technical judgment and experience in their application, and are not a prescriptive technique that can be applied uniformly in all study areas. The guidelines are intended to apply to a range of settings, but they cannot address all settings and conditions due to the broad variability of the Atlantic and Gulf coasts.

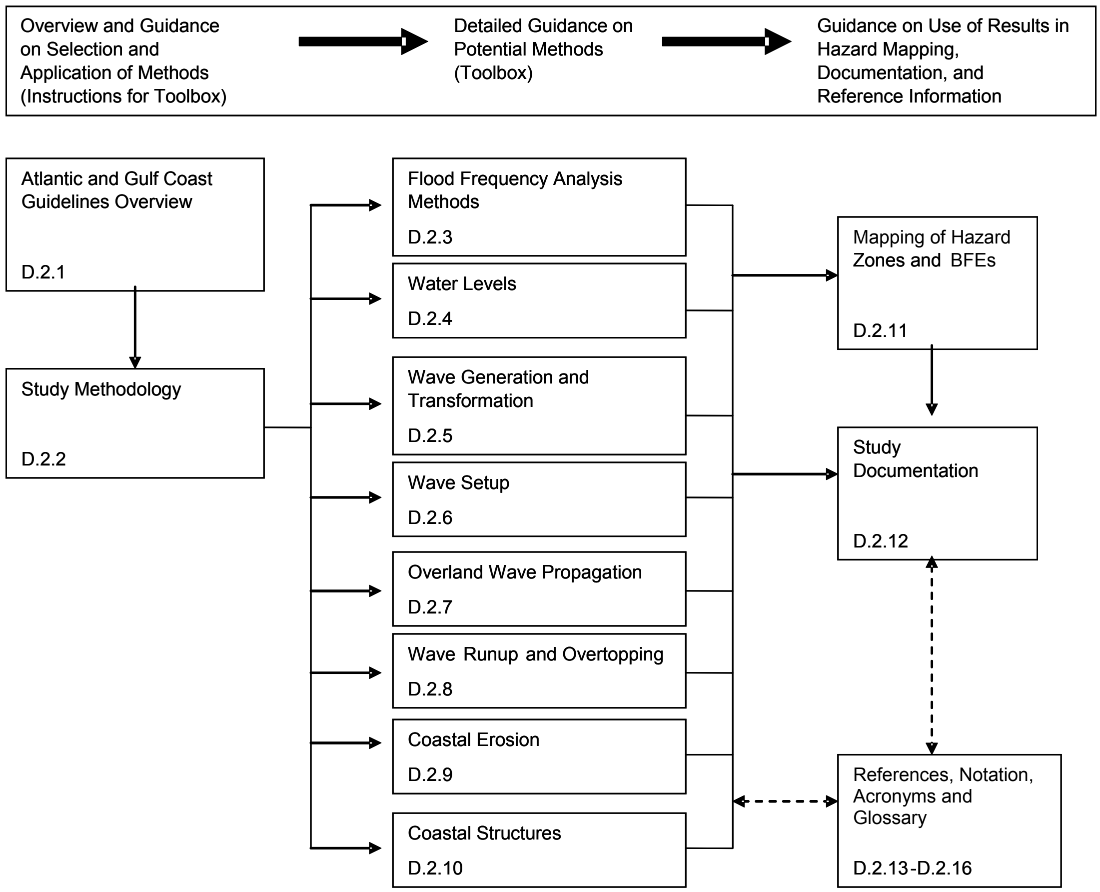

Figure D.2.1-1. Atlantic and Gulf Coast Guidelines Overview

These guidelines include new methods that were developed over a 1-year period by the Technical Working Group (TWG) assembled by FEMA. As always, application of experience and judgment in coastal engineering is necessary to apply the procedures described. The Mapping Partner may determine that minor modifications or deviations from these guidelines are necessary to adequately define the coastal flooding conditions and map flood insurance risk zones in specific areas. In these cases, documentation of these differences is required as part of intermediate and final study submittals.

Other appendices provide specific information on subjects such as project scoping (Appendix I), aerial mapping and surveying (Appendix A), treatment of levee systems (Appendix H), formats for FIS reports and rate maps (Appendices J and K), formats for draft digital data and Digital Flood Insurance Rate Map (DFIRM) databases (Appendix L), guidance for technical and administrative support data (Appendix M), and draft data capture standards and guidelines (draft Appendix N). The guidance provided here is intended only to supplement these subsections with information specific to coastal flooding on the Atlantic and Gulf coasts. The Mapping Partner shall refer to other appendices where specific guidance is required on technical elements common to most FEMA Flood Map Projects.

Subsection D.2.1.1 provides an overview of the Atlantic and Gulf Coast settings relevant to flood hazards, and Subsection D.2.1.2 provides an introduction to FEMA Flood Map Projects for the Atlantic and Gulf coasts.

D.2.1.1 Atlantic and Gulf Coast Settings and Characteristics

The Atlantic and Gulf coasts of the contiguous United States are approximately 1,800 and 1,500 miles in overflight length, respectively, but significantly longer when inlets, bays, headlands, and islands are considered. They encompass a broad spectrum of geological and biological provinces.

Trailing-edge coasts occur on the trailing edge of a landmass that moves with the plate. They are thus situated on passive continental margins that form the stable portion of the plate, well away from the plate margins. The Atlantic coast is an example of a mature, trailing-edge coast. These coasts typically have broad continental shelves that slope into deeper water without a bordering trench. The coastal plain is also typically wide and low-lying and usually contains lagoons and barrier islands.

Marginal sea coasts are those that develop along the shores of seas enclosed by continents and island arcs. Except for the Mediterranean Sea, these coasts do not usually occur along plate margins because the spreading center margins are commonly in ocean basins, while the collision edges of plates face oceans. These coasts are typically bordered by wide shelves and shallow seas with irregular shorelines. The coastal plains of marginal sea coasts vary in width and may be bordered by hills and low mountains. Rivers entering the sea along marginal sea coasts often develop extensive deltas because of the reduced intensity of wave action associated with small bodies of water. The Gulf of Mexico is an example of a typical marginal sea coast (Inman, 1994).

Just as the geology differs spatially along the coasts, so too do the risks associated with flood hazard events. The most severe Atlantic and Gulf coast storms can generally be classified as one of two types: hurricanes and northeasters.

Hurricanes are characterized by large windfields driven by pressure gradients from a central low pressure and temperature gradients in the atmosphere. They can sustain winds of more than 150 miles per hour and are accompanied by large storm surges and waves. The States along the Gulf and Atlantic coasts, from Texas to New York, are most at risk, though hurricanes have been known to reach as far north as Maine.

Unlike hurricanes, northeasters are frontal storms that track the shoreline as they progress northwards following the Gulf Stream. They move slowly and although the winds are typically weaker than hurricanes, they still pose a significant risk because they are accompanied by considerable precipitation and can affect a given area for multiple continuous days. Northeasters are primarily hazards for Atlantic coast states from Maine to North Carolina. (See Figure D.2.1-2.) It should be noted, however, that these regional distinctions are presented for guidance to the Mapping Partner when considering local risks in the study area and do not indicate a prescriptive technique for identifying hazards.

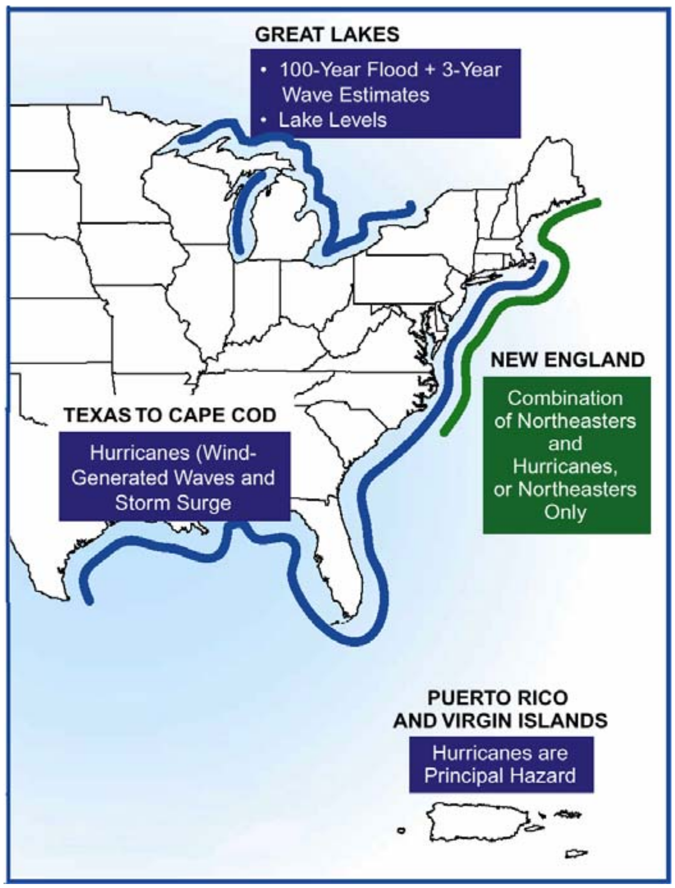

Figure D.2.1-2. Considerations for Determining Coastal Hazards and BFEs

The Atlantic and Gulf coastlines can be generalized into five distinct geological classifications: glaciated, barrier and drowned valley, coral and mangrove, wetland mangrove and barrier, and barrier coasts. The coral and mangrove coasts are found on both the Atlantic and Gulf coasts, while the glaciated and barrier and drowned valley coasts are found primarily on the Atlantic coast. The wetland mangrove and barrier and barrier coasts are found primarily on the Gulf coast, as shown in Figures D.2.1-4 and D.2.1-5 (USACE, 2003). Information in the following subsections, taken from the *Coastal Engineering Manual* (CEM), prepared by the USACE, Coastal and Hydraulics Laboratory, provides a detailed explanation of each of the five classifications.

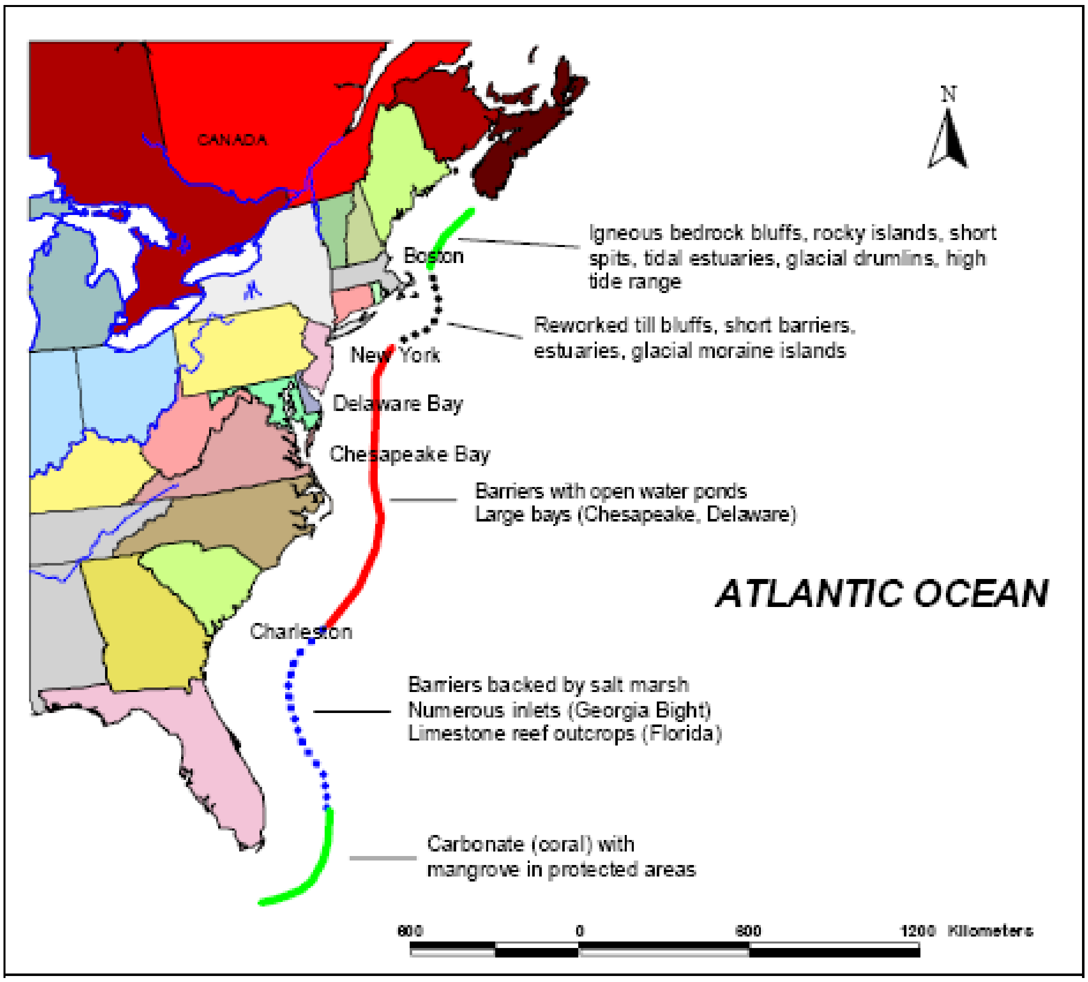

Figure D.2.1-3. Atlantic Coast Geological Characteristics

- *Atlantic North: Glaciated coast*

  These coasts are normally deeply indented and bordered by numerous rocky islands. The embayments usually have straight sides and deep water as a result of erosion by glaciers. Uplifted terraces may be common along these coasts that were formerly weighted down by ice. Abrupt changes in coastal character occur where glacial deposits, particularly glacial outwash, play a dominant role; while in some rocky areas, few glacial erosion forms can be found. Moraines, drumlins, and sand dunes, the result of reworking outwash deposits, are common features. Glaciated coasts in North America extend from the New York City area north to the Canadian Arctic; on the west coast, from Seattle, Washington, north to the Aleutian Islands and in the Great Lakes (Shepard, 1982).

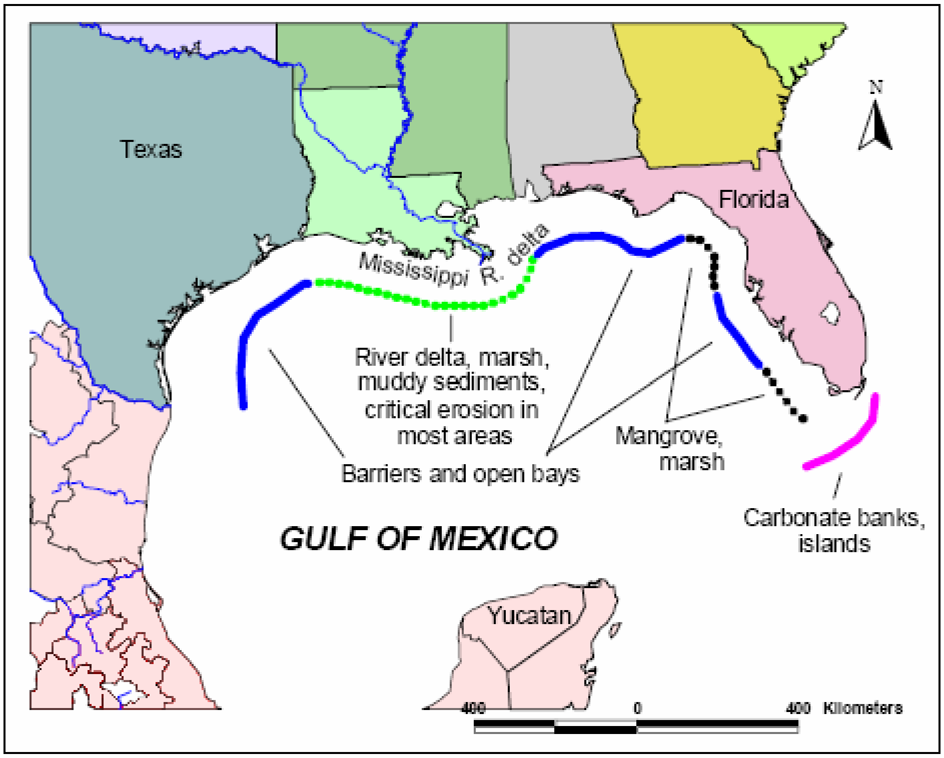

Figure D.2.1-4. Gulf Coast Geological Characteristics

- *Atlantic Central and South: Barrier and Drowned Valley Coasts*

South of the glacial areas begins the coastal Atlantic plain, featuring almost continuous barrier islands interrupted by inlets and by large embayments with dendritic drowned river valleys, the largest being the Delaware and Chesapeake Bays. Extensive wetlands and marshes mark much of the coast, where sediment and marsh vegetation have partly filled the lagoons behind the barriers. Some coasts have inland ridges of old barrier islands, formed during interglacial epochs, separated from the modern barrier islands by low marshes or lagoons. The best exhibit of cuspate forelands in the world extends from the mouth of the Chesapeake Bay to Cape Romain, South Carolina. The coast is much straighter south of Cape Romain and the only cuspate foreland is that of Cape Canaveral, Florida. Barrier Islands and drowned valleys continue south to Miami, Florida, except for a brief length of coast near Myrtle Beach, South Carolina, where the barriers are attached to the coastal plain. Much of the southeast coast of Florida was extensively filled, dredged, and reshaped in the early 20th century to support development (Lenček and Bosher, 1998). From Miami, around the tip of Florida, through Alabama, Mississippi, and eastern Louisiana, coastal characteristics alternate between swampy coast and white sand barriers (Shepard 1982).

- *The Atlantic and Gulf of Mexico: Coral and Mangrove Coasts*

  South of Miami, the barrier islands change from quartz sand to carbonate-dominated sand, eventually transforming into coral keys and mangrove forest. The Florida Keys are remnants of coral reefs developed during a higher sea level stage of the last interglacial period. Live reefs now grow along the east and south side of the keys and the shallows of Florida Bay are studded with mangrove islands extending north and west into the Everglades and the Ten Thousand Islands area that comprises the lower Florida Gulf Coast (Shepard 1982).

- *Gulf of Mexico East: Wetland Mangrove and Barrier Coasts*

  On Florida's Gulf Coast, barrier islands begin at Cape Romano and extend north as far as Cedar Keys. Enclosed bays usually have an abundance of mangrove islands and the topography is low with many lakes and marshes. North of Cedar Keys, the barrier islands end. They are replaced by a vast marsh dotted with small vegetated islands. The rock strata in this area are limestone, which, along with the low river gradients and numerous ponds or sinkholes, accounts for the absence of sand in the region. Because of its location and the large shallow water area offshore, little wave energy is present except during hurricanes. Some 130 kilometers (km) to the northwest, the swamp coast ends. Here the coastal trend changes direction from north-south to east-west, and Ochlockonee Bay, with drainage from the southern Appalachian Mountains, provides quartz sand for redevelopment of barrier islands. These sandy islands, with their various openings for access to the lowland port cities, continue westward as far as the Mississippi River delta.

  Studies of the Mississippi River delta indicate that the river has built a series of deltas into the Gulf of Mexico during postglacial times and that the Balize Delta (Bird Foot) is the latest, with an age of about 1,500 years. The Bird Foot delta is southeast of New Orleans, lying among a series of old passes that extend for 300 km (186 miles) along the coast. Most of the greater Mississippi River Delta is marshland and mud flats, with numerous shallow lakes and intertwining channels. Aquatic plants cover the marshland, which is renowned for the huge population of waterfowl it supports. The principal rivers have built natural levees along their course. These natural levees are about a meter above the normal water level, but many of them have been artificially raised to provide flood protection to towns and cities. In the areas of old delta lobes, subsidence has left only the natural levees above water, in some instances.

- *Gulf of Mexico West: Barrier Coast*

  From western Louisiana, west of the Mississippi River Delta marsh coast, toward the southwest, barrier islands become the dominant coastal features. Some of the longest barrier islands in the world are located along the Texas coast. Padre Island and Mustang Island, combined, extend for 208 km and feature extensive dune fields behind the broad beaches. The dunes rarely rise more than 10 meters in height, and many marshy wash-over deltas have extended into the large lagoons behind the barriers. The lagoons and estuaries decrease in depth toward Mexico. A large part of Laguna Madre is only inundated during flood periods or when the wind blows water from Corpus Christi Bay onto the flats. River deltas are responsible for much of this infilling, resulting in large differences between recent chart depths and those of 100 years ago (Shepard 1982).

D.2.1.2 Atlantic and Gulf Coast Flood Map Projects

This subsection briefly introduces Atlantic and Gulf coast studies through a discussion of general study considerations, including special considerations for sheltered waters and unique study conditions. Descriptions of typical project scoping activities, flood insurance risk zone definitions, and reporting requirements are also provided. Additional information on flood insurance risk zone mapping and study documentation is provided in Subsections D.2.11 and D.2.12, respectively.

D.2.1.2.1 Project Scoping

Project scoping is defined as the process of determining the extent of a particular coastal study and defining the fundamental methodologies to be used in completing the study. As presented in this subsection, this process includes two major tasks.

The first task is designed to assess the need for flood hazard mapping for communities and to assign priorities. Mapping Partners should evaluate the study area, prioritize study reaches, assign rankings and designate funds for specific aspects of the study according to the needs of the community and FEMA.

The second task involves determining general study methodologies based on study area setting, morphology, and coastal processes. This step also includes practical considerations of data availability and data collection needs, as well as study time and budget requirements. Subsections D.2.2 and D.2.3 on study methodology and analysis methods shall be consulted by Mapping Partners to determine which methods are appropriate for a particular coastal study setting and their general requirements for data and flooding analysis. In some complex study areas, a scoping phase of the coastal Flood Map Project may be needed to determine the availability of data and define a study methodology that combines a number of analysis methods and mapping procedures. When scoping for coastal redelineation studies, the Mapping Partner shall consult Subsection D.2.11 on mapping procedures in order to become familiar with potential datum conversion and other redelineation issues.

The following general procedures shall be followed for scoping the study methodology:

1. Define the objectives of the project based on information from the communities, and information from the FEMA Study Representative.
2. Review prior flood studies at the site or in the vicinity.
3. Review the study area setting exposure and shoreline morphology.

4. Make an initial assessment of the probable types and extent of hazard zones in the study area.
5. Identify subregions and reaches based on onshore conditions (e.g., shore geometry, structures), nearshore conditions (e.g., local exposure, profile morphology), and offshore conditions (e.g., depth contours, geometry of sheltered waters).
6. Define potentially applicable study methodologies using Subsections D.2.2 and D.2.3 as guidance.
7. Determine data requirements and data availability to support various analysis methods.
8. Assess the probable study methods in terms of level of complexity and probable accuracy of results – in general, the simplest methodology that provides reliable results shall be chosen. Incremental benefits of more sophisticated or detailed analysis may be assessed in this step.
9. Refine selection of analysis methods based on data requirements and reliability to synthesize an overall study methodology that effectively combines multiple analysis methods. For some studies, alternatives to the methods described in this subsection may be required to address specific situations.
10. Confirm that the study methodology is adequate to support development of anticipated flood insurance risk zones and produce required mapping.
11. Estimate time and budget requirements.
12. Adjust study extent, data collection, analysis methods, or overall methodology, if necessary, to meet study time and budget constraints.

Some flexibility is desirable in selecting study methodologies with respect to the procedures defined in these guidelines. Overarching considerations in selecting study methodologies shall include a basis in physical processes and quality-assured data, use of technically reliable and current analysis methods, reproducibility using standard engineering methods, verification of results using sensitivity tests and simple checks, and consistency with this appendix and other FEMA guidance.

D.2.1.2.2 Regional Versus Local Studies

Flood Insurance Studies were traditionally been performed for a single political jurisdiction, most commonly a community, with the FIS reports and FIRMs/DFIRMs being specifically developed for that community. Adjacent communities have been addressed only insofar as necessary to ensure that Base Flood Elevations (BFEs) match at the community boundaries. The hydrologic and hydraulic efforts have also typically stopped at the community boundaries, or have extended only so far beyond them as to encompass complete hydrologic units, such as drainage basins, which are necessary to determine conditions within the study community.

This local study approach has been followed, in part, due to the demanding computational effort necessary to encompass large regions within the analysis. For example, storm surge calculations require large computational grids, which in turn require large computer capacity and long execution time. To model more than a limited coastal region was difficult or impossible with the computer capabilities of only a few years ago. Similarly, ocean wave simulations have been restricted to limited zones in past studies. Although this community-by-community approach proved tractable, it also introduces some compromise into the studies. For example, a long length of coast that is simulated by breaking it into small sections means that boundary conditions must be specified for each segment, with some probable loss in both efficiency and accuracy.

A second compromise in local studies is that different Mapping Partners may make different assumptions that lead to differences between adjacent studies. Furthermore, not all Mapping Partners have the necessary tools and experience to perform some types of coastal flooding analyses.

The idea of regional studies is to perform large-scale regional analyses for certain portions of the engineering tasks needed in a community study and to make these analyses available as input to the local studies. For example, Subsection D.2.4 of these guidelines describes large regional databases (e.g., Global Reanalysis of Ocean Waves [GROW] data) of wave hindcast data. These data can be transformed to the nearshore area, just outside the surf zone, as part of a regional study effort covering a very large portion of the Atlantic and/or Gulf Coasts, using a single, consistent, state-of-the-art methodology. The advent of modern computational abilities makes these regional efforts feasible and more cost-effective than community-by-community repetition of a similar effort.

Regional studies can be implemented to varying degrees. Regional studies need not be as large as an entire coastline or a statewide analysis, but instead might cover a limited number of counties. This would be the case if there is a physical characteristic of a region that makes it logical to treat it as a unit, instead of breaking it up into smaller areas. For example, wave studies might be accomplished regionally according to directional exposure, island sheltering, breadth of shelf, or other physical factors. In general, processes that originate in the far field – such as storm surge – are candidates for regional analysis because a single coherent source might affect a large coastal reach. In an event-selection analysis, the selected event might be adopted regionally, controlling behavior within a multi-community basin such as a large sound.

The extent to which regional studies, perhaps focused on particular coastal processes, are available and can be used in local studies depends on planning and implementation of these studies by FEMA. The Mapping Partner shall consult with FEMA Study Representatives during the project scoping to determine if relevant regional information or analysis is available and should be incorporated into the study methodology.

D.2.1.2.3 Sheltered Waters

A generally accepted definition for “sheltered waters,” which is taken here to include inland waters, enclosed basins, fetch-limited waters, and low-energy beaches, does not exist (Jackson et al., 2002). For the purposes of these guidelines, “sheltered” is assumed to imply a significant sheltering effect on the inland propagation of storm surge, waves, and wind by land masses and vegetation. “Sheltered waters” are water bodies or regions that experience diminished forces from wind and/or wave action relative to the open coast due to the presence of physical barriers, both natural and human, either on land or under water.

Sheltered water areas are exposed to the same flood-causing processes as are open coastlines (i.e., high winds, wave setup, runup, overtopping), but sheltering effects reduce the wave energy and flood potential. The Mapping Partner shall evaluate these potential sheltering effects at both a regional scale and a local site scale.

At a regional scale, wind-generated waves in sheltered water areas are highly dependent on the shape and orientation of the surrounding terrain to prevailing wind directions. Wave generation and transformation in sheltered waters are usually limited by the open water fetch distance, complex bathymetry, and often the presence of in-water and shoreline coastal structures. Other processes, such as the effects of flood discharges from rivers, can modify local tidal and storm surge elevations, and relatively strong tidal and/or fluvial currents can combine to create tidal and hydrodynamic conditions only found in sheltered water areas. (See Subsection D.2.4 for details on statistical determination of flood levels in areas with multiple flooding sources.)

Bays and estuaries often display significant spatial variability in tidal still water elevations as a result of the combined effects of complex tidal hydraulics, residual currents, local winds, and river runoff. Oceanic storm surge can also be modified in estuaries, with surge heights sometimes uniformly additive to local tidal datums throughout an estuary, or amplified or muted within a given region of a large estuary.

The Mapping Partner shall review bathymetric and topographic maps and aerial photographs, and make field observations to determine if a coastal flood study site is located within sheltered waters and to assess the degree of sheltering from swell, waves, and wind. The Mapping Partner shall investigate local site scale features contributing to sheltering from wind and waves and affecting flooding at the study site. It is important to note that sheltered water characteristics and processes viewed at a regional scale may be different at a local scale due to site-specific controls (Jackson and Nordstrom, 1992). In general, a more detailed examination of local conditions will be required in sheltered waters than on the open coast.

General wave transformation conditions within a sheltered water body may be inferred from wave patterns observed on vertical aerial photographs. During field reconnaissance, the Mapping Partner shall make field observations to identify conditions that affect selection of a study approach. Jackson et al. (2002) have identified characteristics of sheltered water shorelines that may be useful as a guide for field reconnaissance.

The Mapping Partner shall define a general approach to a sheltered water study at the scoping phase of the project. Because sheltered water areas experience the same flood-causing processes as open coast areas, guidance for performing coastal flood studies in sheltered waters is integrated throughout the remainder of these guidelines. Where procedures apply specifically to sheltered waters, they are identified in the individual subsections.

Beyond the initial effort to determine if a study site is located within a sheltered water area, as described above, a general approach to sheltered water studies shall address the following topics:

- **Topography/Bathymetry:** The Mapping Partner shall obtain backshore topography to define hazard zones, obtain nearshore bathymetry to define beach profiles, and define the geometry (size and volume) of the sheltered water body to evaluate hydrodynamic conditions. Detailed bathymetric data will likely be required in tidal inlets to assess their hydrodynamic characteristics, which may control the magnitude and timing of flood components, such as tidal stillwater levels and wave propagation.

- **Wind:** The climate in sheltered waters is dependent on localized wind conditions, and wave data are typically unavailable at a suitable resolution. The study approach will typically focus more on the identification of appropriate wind data sources rather than wave data (as may be relied upon for open-coast studies). Accordingly, the Mapping Partner shall identify, obtain, and review available wind data from the nearest appropriate sources; augment long-term data from established weather stations with available short-term data from local governments, industries, or private landowners to verify local wind conditions; and define characteristics related to fundamental wind parameters, such as wind source, seasonal direction, duration, magnitude, and vertical velocity distribution.

- **Tide and Currents:** The Mapping Partner shall identify, obtain, and review available tide gage data to define fundamental tide characteristics, such as astronomical tide, storm surge, tidal amplification, wind setup, and tidal and fluvial currents. Long-term tidal elevation data from established tide stations may need to be augmented with data from other sources. In some cases, estimates of natural tidal datums from landscape features, such as mud and vegetation lines, may provide verification of estimated extreme tidal elevations.

- **Waves:** The Mapping Partner shall obtain available data on observed wave height, wave length, and wave period, and shall assess probable extreme wave conditions given potential bathymetric and vegetative effects on wave energy.

These general topics can define the forcing functions, boundary conditions, and constraints necessary for analytical and/or numerical modeling approaches to flood determination. Sheltered water physical processes can be complex and may require detailed numerical modeling to adequately define the flood hazards. Given the availability and relative ease of use of modern numerical models, the Mapping Partner shall consider a numerical modeling approach to a sheltered water study where simpler methods do not appear reliable.

Model selection shall be made with consideration of the level of complexity of physical processes, data available for calibration, flood risk, and available study budget. If the physical scale of the sheltered water coastal flood study is small and the geographic setting and physical processes are relatively well understood and simple, the Mapping Partner shall confer with the FEMA Study Representative about the feasibility of using simplified analytical approaches instead of numerical models. A limited analytical approach may also be appropriate to obtain a quick assessment of physical conditions and/or to provide a check of the results from a numerical modeling approach.

D.2.1.2.4 Debris

Debris entrained in tidal floodwaters and cast inland by storm surge and wave propagation may occur along parts of the Atlantic and Gulf coasts. Natural debris consists of floating woody debris, such as drift logs, branches, cut firewood, and other natural floatable materials. Wave-cast beach sediments, such as cobbles and gravel, also constitute natural debris.

Debris from human sources may originate from flood damage. This debris may include broken pieces of shore revetment cast inland by extreme surge and wave attack, or floatable materials, such as construction materials, building materials, and home furnishings.

Debris hazards depend on the beach type and configuration, debris sources, the inland extent of wave propagation, the proximity of insured structures to the shoreline, and the height of the structures above the BFE. At present, debris hazards are not explicitly included in FEMA flood insurance risk zones and therefore a detailed debris analysis is not required. However, the Mapping Partner shall note significant debris hazards in a study area, document the hazards in the "Principal Flood Problems" section of the FIS report, and confer with the FEMA Study Representative so relevant information may be shared with community floodplain managers.

D.2.1.2.5 Beach Nourishment and Constructed Dunes

Current FEMA policy is not to consider the effects of beach nourishment projects in flood hazard mapping. Beach nourishment, in effect, is treated as a temporary shoreline disturbance, or an "uncertified" coastal structure (a structure not capable of withstanding the 1-percent-annual-chance flood event and/or a structure without an approved maintenance plan).

However, given that beach nourishment is conducted by more and more communities in response to coastal erosion, it is becoming increasingly difficult to obtain recent topographic data that do not reflect prior beach nourishment. In many communities, beach nourishment has been ongoing for a decade or more (predating the NFIP in some cases). Mapping Partners should be aware that flood hazard mapping of coastal areas could potentially be affected by various types of beach nourishment, and that current topographic data may reflect beach nourishment efforts.

The Mapping Partner shall determine whether beach nourishment affects a study area, research any beach nourishment projects identified, identify any available data that would allow the performance of the beach nourishment project(s) to be assessed, and determine whether the beach nourishment is likely to persist and have an effect on flood hazard mapping. If it is determined that beach nourishment will likely affect flood insurance risk zones or BFEs, the Mapping Partner shall contact the FEMA Study Representative to determine whether an exception to current FEMA policy should be considered.

The presence of constructed dunes in the study area may raise similar questions. For all practical purposes, the Mapping Partner shall treat constructed or reconstructed dunes (i.e., "artificial" dunes) as natural dunes during the study process if they meet the criteria set forth in the NFIP regulations. Paragraph 65.11(a) of the NFIP regulations does not allow an artificial dune to be considered an effective barrier against coastal flooding unless it has well-established, longstanding vegetative cover, regardless of its size and cross section.

D.2.1.2.6 Data Requirements

To conduct a study for a coastal community, the Mapping Partner shall first collect the wide variety of quantitative data and other site information required to perform the required analyses. Some data are entered directly into computer models of flood effects, while other data are used to interpret and integrate the calculated results.

Each computer model of a separate flood effect is executed along transects, which are cross sections taken perpendicular to the mean shoreline to represent a segment of coast with similar characteristics. Thus, collected data are compiled primarily for transects, which, in turn, are situated on work maps at the final scale of the DFIRM. Work maps are used both to locate and develop the transects and to interpolate and delineate the flood zones and elevations.

In addition to the necessary quantitative information, the Mapping Partner shall collect descriptions of previous flooding and the community in general to aid in the evaluation of flood hazards and for inclusion in the FIS report. The Mapping Partner shall begin this data collection effort at the community level and then turn to county, State, and Federal data sources. The Mapping Partner also shall contact private firms specializing in topographic mapping and/or aerial photography at the suggestion of government agencies.

D.2.1.2.6.1 Stillwater Elevations

The Mapping Partner performing the analysis shall determine the SWELs in a rational, defensible manner and shall not include contributions from wave action either as a result of the mathematics of the predictive model or of the data used to calibrate the model. Only the 1-percent-annual-chance SWEL is required for the coastal analyses, although 10-, 2-, and 0.2-percent-annual-chance elevations are provided in the FIS report and the 0.2-percent-annual-chance floodplain boundary is mapped on the DFIRM.

SWELs may be defined by statistical analysis of available tide gage records or by calculation using a storm surge computer model. FEMA also has specified procedures and documentation for coastal flood studies using a storm-surge model, as presented previously in Subsections D.2.3 and D.2.4. Of particular importance in this determination, the surge model study can provide estimates of the wind and water levels likely to occur over the course of the 1-percent-annual-chance flood.

D.2.1.2.6.2 Selected Transects

The Mapping Partner performing the analysis shall locate transects with careful consideration of the physical and cultural characteristics of the land so that the transects will closely represent conditions in their locality. The transects shall be placed closer together in areas of complex topography, dense development, unique flooding, and areas where computed wave heights and runup may be expected to vary significantly. Wider spacing may be appropriate in areas with more uniform characteristics. For example, a long stretch of undeveloped shoreline with a continuous dune or bluff of fairly constant height and shape and similar landward features may require a transect every 1 to 2 miles. However, a developed area with various building densities, protective structures, and vegetation cover may require a transect every 1,000 feet.

If good judgment is exercised in placing required transects, the Mapping Partner will avoid excessive interpolation of elevations between transects, while also avoiding unnecessary study effort. In areas where runup may be significant, the proper location of transects is governed by variations in shore slope or gradient. On coasts with sand dunes, the Mapping Partner shall site transects according to major variations in the dune geometry and the upland characteristics. In areas where dissipation of wave heights may be most significant in the computation of flood hazards, the Mapping Partner shall base transect location on variations in land cover (i.e., buildings, vegetation, and other factors). The Mapping Partner should site a separate transect at each flood protection structure. However, if areas with similar characteristics are scattered throughout a community and have the same SWEL, the Mapping Partner may apply the results from one transect at various locations within this common area. This is to be done only after careful consideration is given to topographic and cultural features to assure accurate representation of coastal hazards.

The Mapping Partner shall locate transects on the work map and compile the input data on a separate sheet for each transect. The data for each transect should not be taken directly along the line on the work map. Rather, they should be taken from the area, or length of shoreline, to be represented by each particular transect so that the input data depict the average characteristics of the area. Because of this, the Mapping Partner may find it is useful to divide the work map into transect areas for purposes of data compilation.

D.2.1.2.6.3 Topography

The topographic data must have a contour interval no greater than 5 feet or 1.5 meters. More information regarding topographic data can be found in Appendix A of these Guidelines. The topographic data, usually in the form of maps, must be recent and reflect current conditions or, at a minimum, conditions at a clearly defined time. Transects need not be specially surveyed unless available topographic data are unsuitable or incomplete. The Mapping Partner shall examine the topographic data to confirm that the information to be used in the analysis and mapping represents the actual planimetric features that might affect identification of coastal hazards.

If possible, the Mapping Partner shall field-check shore topography to note any changes caused by construction, erosion, coastal engineering, or other factors. The Mapping Partner shall document any significant changes with location descriptions, drawings, and/or photographs. The community, county, and State are usually the best sources for topographic data. The Mapping Partner shall examine U.S. Geological Survey (USGS) 7.5-minute series topographic maps. If the contour interval of the USGS maps are greater than 5 feet, they still may prove useful as reference or base maps.

D.2.1.2.6.4 Land Cover

The land-cover data include information on buildings and vegetation. Stereoscopic aerial photographs can provide the required data on structures and some of the data on vegetation. The Mapping Partner shall ensure that aerial photographs are not more than 5 years old unless they can be updated by surveys. Local, county, or State agencies may have the coastline photographed on a periodic basis and may provide photographs or permission to obtain them from their source.

Aerial photographs can provide the required data on tree- and bush-type vegetation. However, although they are useful in identifying areas of grass-like vegetation, they cannot identify specific types. National Wetland Inventory maps from the U.S. Fish and Wildlife Service and color infrared aerial photographs can provide some more specific data required for marsh plants. Ground-level photographs are also useful in providing information on plants. State offices of coastal zone management, park and wildlife management, and/or natural resources should be able to provide information.

The Mapping Partner also may contact local universities with coastal studies and/or Sea Grant programs. The Mapping Partner may conduct field surveys in lieu of obtaining data from the above sources, but field surveys are more cost effective when used only to supplement or verify data.

D.2.1.2.6.5 Bathymetry

The Mapping Partner may acquire bathymetric data from National Ocean Service nautical charts, although any reliable source may be used. The bathymetry must extend far enough offshore to include the breaker location for the 1-percent-annual-chance flood. Although that depth may not be exactly known during the data collection phase, the Mapping Partner may assume that a mean water depth of 40 feet will encompass all typical breaker depths. Bathymetry further offshore also may be useful in interpreting likely differences between nearshore and offshore wave conditions and may be necessary where offshore waves are more readily specified.

D.2.1.2.6.6 Storm Meteorology

The 1-percent-annual-chance flood elevations represent a statistical summary and likely do not correspond exactly to any particular storm event. However, the meteorology of storms believed to have been approximations of the 1-percent-annual-chance flood can be useful information in selecting recurrence intervals for historical events and in assessing wave characteristics likely associated with the 1-percent-annual-chance flood. An important distinction of the flood source from Delaware to Maine is whether the 1-percent-annual-chance flood is more likely to be caused by a hurricane or by a Northeaster. The Mapping Partner must make this distinction in the course of defining SWELs because the time history of water levels can be radically different in each case.

D.2.1.2.6.7 Storm Wave Characteristics

The basic presumption in conducting coastal wave analyses is that wave direction must have some onshore component, so that wave hazards occur coincidentally to the 1-percent-annual-chance flood. This presumption appears generally appropriate for open coasts and along many mainland shores of large bays, where the 1-percent-annual-chance SWEL must include some contribution from direct storm surge and thus requires an onshore wind component. However, an assumption of onshore waves coincident with a flood may require detailed justification along the shores of connecting channels, in complex embayments, near inlets, and behind protective islands. Once it is confirmed that sizable waves likely travel onshore at a site during the 1-percent-annual-chance flood, the storm wave condition must be defined for assessments of coastal structure stability, sand dune erosion, wave runup and overtopping, and overland elevations of wave crests.

It is important to recognize that somewhat different descriptions of storm waves (Table D.2.1-1) can be appropriate in assessing each distinct flooding effect. This depends mainly on the formulation of an applicable empirical or analytical treatment for each effect. In Flood Map Project models and analyses, the different wave descriptions include the following:

- Various wave statistics (e.g., mean wave condition for runup elevations, but an extreme or controlling height for overland waves);

- Various dominant parameters (e.g., incident wave height for overtopping computation, but incident wave period for overland crest elevations); and
- Various specification sites (e.g., deep water for estimating runup elevations, but transformation of waves actually reaching a structure in shallow water for most stability or overtopping considerations).

To proceed with general orientation, the Mapping Partner may develop storm wave conditions from actual wave measurements, wave hindcasts or numerical computations based on historical effects, and specific calculations based on assumed storm meteorology. Where possible, the Mapping Partner shall pursue two or all three of these possibilities in estimating wave conditions expected to accompany the 1-percent-annual-chance flood at a study site. Using all available information can improve the level of certainty in estimated storm wave characteristics.

Table D.2.1-1. Some Commonly Used Specifications of Irregular Storm Waves

Symbol	Name	Description
Wave Heights (water depth must be given)		
H_s	Significant	average over highest one third of waves
H_c	Controlling	defined as (1.6 H_s) in NAS (1977)
$\overline{H}$	Mean	average over all waves
H_{mo}	zero moment	defined by the variance of water surface (about equal to H_s in deep water)
Wave Periods (basically invariant with water depth)		
T_s	significant	associated with waves at significant height
T_p	peak	represents the maximum in energy spectrum
$\overline{T}$	mean	average over all waves

D.2.1.2.6.8 Coastal Structures

The Mapping Partner shall obtain documentation for each coastal structure that may provide protection from the 1-percent-annual-chance flood. That documentation shall include the following:

- Type and basic layout of the structure;
- Dominant site particulars (e.g., local water depth, structure crest elevation, and ice climate);
- Construction materials and present integrity;

- A historical record for the structure, including construction date, maintenance plan, responsible party, repairs after storm episodes; and.
- Clear indications of the effectiveness or ineffectiveness of the structure as protection.

The Mapping Partner shall develop much of this information through office activity, including a careful review of aerial photographs. In some cases, site inspection would be advisable for major coastal structures to confirm preliminary judgments.

D.2.1.2.6.9 Historic Floods

While not required as input to any of the FEMA coastal models, local information regarding previous storms and flooding can be very valuable in developing accurate assessments of coastal flood hazards and validation of storm-surge models. General descriptions of flooding are useful in determining what areas are subject to flooding and in obtaining an understanding of flooding patterns. More specific information, such as the location of buildings flooded and damaged by wave action, can be used to verify the results of the coastal analyses. Detailed information on pre- and post-storm beach or dune profiles is valuable in checking the results of the erosion assessment.

When quantitative data are available on historical flooding effects, the Mapping Partner shall make a special effort to acquire all recorded water elevations and wave conditions for the vicinity. This information can be used in estimating recurrence intervals for SWELs and for wave action during a flood event and in assisting an appropriate comparison to the 1-percent-annual-chance flood.

Local, county, and State agencies are good sources of historical data, especially more recent events. It is becoming common practice for these agencies to record significant flooding with photographs, maps, and/or surveys. Some Federal agencies (e.g., USACE, U.S. Geological Survey (USGS), and the National Research Council) prepare post-storm reports for more severe storms. Local libraries and historical societies may provide useful data.

D.2.1.2.7 Hazard Zone Definitions and Use by FEMA

Coastal flood insurance risk zones shown on the FIRM are generally divided into three categories: 1) VE zone (the coastal high hazard area); 2) AE zone (and other A zones, where flood hazards are not as severe as in VE zones); and 3) X zone (which is only subject to flooding by floods more severe than the base flood). AH zone and AO zone designations are used in special situations.

Delineation of flood insurance risk zones involves a set of analyses (waves, water levels, wave effects, and shoreline response) combined into a methodology for a particular study area. The criteria for establishing flood insurance risk zones are briefly described below. The reader should refer to subsequent subsections for a detailed description of the mapping parameters and their derivation.

D.2.1.2.7.1 VE Zone

VE zones are coastal high hazard areas where wave action and/or high-velocity water can cause structural damage during the base flood. They are subdivided into elevation zones with BFEs assigned. VE zones are identified using one or more of the following criteria for the base flood conditions:

1. The ***wave runup zone*** occurs where the (eroded) ground profile is 3.0 feet or more below the 2-percent wave runup elevation
2. The ***wave overtopping splash zone*** is the area landward of the crest of an overtopped barrier, in cases where the potential 2-percent wave runup exceeds the barrier crest elevation by 3.0 feet or more($\Delta R > 3.0$ feet). (See Subsection D.2.8.2.)
3. The ***breaking wave height zone*** occurs where 3-foot or greater wave heights could occur (this is the area where the wave crest profile is 2.1 feet or more above the total stillwater level).
4. The ***primary frontal dune zone***, as defined in 44 CFR Section 59.1 of the NFIP regulations.

D.2.1.2.7.2 AE Zone

AE zones are areas of inundation by the 1-percent-annual-chance flood, including areas with the 2-percent wave runup elevation less than 3.0 feet above the ground and areas with wave heights less than 3.0 feet. These areas are subdivided into elevation zones with BFEs assigned. The AE zone will generally extend inland to the limit of the 1-percent-annual-chance flood SWEL.

D.2.1.2.7.3 AH Zone

AH zones are areas of shallow flooding or ponding with water depths generally limited to 1.0 to 3.0 feet. These areas are usually not subdivided, and a BFE is assigned.

D.2.1.2.7.4 AO Zone

AO zones are areas of sheet-flow shallow flooding where the potential runup is less than 3.0 feet above an overtopped barrier crest ($\Delta R < 3.0$ feet). The sheet flow in these areas will either flow into another flooding source (AE zone), result in ponding (AH zone), or deteriorate because of ground friction and energy losses and merge into the X zone. AO areas are designated with 1-, 2-, or 3-foot depths of flooding.

D.2.1.2.7.5 X Zone

X zones are areas above the 1-percent-annual-chance flood level. On the FIRM, a shaded X zone area is inundated by the 0.2-percent-annual-chance flood, and an unshaded X zone area is above the 0.2-percent-annual chance flood.

Detailed guidance on hazard zone mapping is provided in Subsection D.2.11.

D.2.1.2.8 Reporting Requirements

Reporting requirements for coastal studies shall follow guidance provided in Appendix M for the preparation of a Technical Support Data Notebook (TSDN). The TSDN shall consist of the following four major sections, which are more specifically described in Appendix M:

- General documentation;
- Engineering analyses;
- Mapping information; and
- Miscellaneous reference materials.

In general, the material compiled for these sections of a coastal study TSDN will be similar to a riverine study, with the exception of the engineering analyses section. The engineering analyses section of a TSDN for a coastal study shall be formatted to reflect the required intermediate data submissions, together with the subsequent correspondence from FEMA and any other subsequent documentation related to a particular intermediate data submission. The purpose and content of individual intermediate data submissions are briefly described below.

Due to the differences between coastal and riverine flood studies and the complexity of coastal studies, intermediate data submissions are required from the Mapping Partner. Intermediate data submissions provide defined milestones in the coastal flood study process where independent reviews are conducted to confirm that the methods and findings are acceptable to FEMA. The primary purpose of this submission and review process is to minimize revisions to analysis methods late in the study.

Coastal analyses involving hydrodynamic modeling for development of water levels and wave processes (transformation, refraction, and diffraction) are highly specialized and complex. Changing or correcting the water-level and wave analyses after they have been used in analysis of shoreline processes and in flood insurance risk zone mapping is expensive and time consuming. Therefore, FEMA has established intermediate data submission requirements to facilitate review of analysis methods and results at appropriate milestones. Additional specific information on reporting requirements is provided in Subsection D.2.12. In general, the Mapping Partner shall submit the data for FEMA review in accordance with the sequence discussed below.

D.2.1.2.8.1 Intermediate Submission No. 1 – Scoping and Data Review

In this phase of reporting, the Mapping Partner provides the background information on the study setting and available data relevant to the study area. Any new data needed for the detailed coastal analyses in subsequent phases should be identified in this phase. The study should not proceed until all of the information is available and incorporated in the scoping document for approval.

D.2.1.2.8.2 Intermediate Submission No. 2 – Storm-surge Model Calibration and Storm Selection

Documentation of this phase shall include a description of the calibration, validation and sensitivity analysis of the storm-surge model to be used in the generation of surge elevations for flood frequency-of-occurrence analysis. It shall also include a description of the selection and definition of storm events to be used in the statistical analysis.

D.2.1.2.8.3 Intermediate Submission No. 3 – Storm-surge Modeling and Flood-Frequency Analysis

Documentation shall be provided on the methods used to estimate the 1- and 0.2-percent–annual-chance coastal flooding conditions. Documentation may include response-based and simulation methods (e.g., JPM, Monte Carlo, or EST), depending on study setting. Methods of extrapolation of hindcast and/or measured data to 1- and 0.2-percent–annual-chance values should be documented, including comparisons between alternate procedures, if appropriate. In cases for which extreme value analyses of wave, wind, water level, and residual tides are used, the submission shall include documentation of the analyses to develop frequency relationships, including a description of the data sets and analytical assumptions.

D.2.1.2.8.4 Intermediate Submission No. 4 – Nearshore Hydraulics

This submission shall be completed before flood hazard mapping is conducted and shall document the analyses related to the following four classes of coastal processes: water level and wave analyses to develop base flood conditions at the shoreline, including wave modeling for transformation, refraction, diffraction, and shoaling; wave runup, setup, and overtopping assessments in the surf zone; coastal structure and erosion analyses; and inland and overland water level and wave propagation analyses. This submission should include data on control, field, aerial, and bathymetric surveys. It should also include validation of results with available historical flood data, and discussion of modeling results by transect (as needed for interpretation of flood hazards). Where riverine sources influence coastal flood insurance risk zones in the study area, this submission shall include analysis of riverine flood stages and frequencies.

D.2.1.2.8.5 Intermediate Submission No.5 – Hazard Mapping

This submission will be prepared at the completion of draft delineations of flood insurance risk zones. The Mapping Partner shall document the analysis results used in the determination of hazard zone limits and BFEs and provide draft work maps for the study area showing all flood insurance risk zone boundaries.

The Mapping Partner will receive review comments within 30 days of the receipt of each data submission. The Mapping Partner shall include the interim review in the project schedule and shall plan the study work to minimize the effect of the reviews on the overall schedule for FIS report and DFIRM production.

D.2.2 Study Methodology

This subsection provides guidance for selecting and combining specific technical methods and data into a study methodology. The selection of methods depends upon the coastal setting and the available data.

D.2.2.1 Overview

In this appendix, "methods" means the individual techniques used to make specific computations. "Study methodology" is the combination of appropriate methods and data necessary to develop flood insurance risk zones for depiction on a FIRM. A variety of technical methods are available for application in the unique settings of each coast, with those most appropriate for the Atlantic and Gulf coasts presented in Subsections D.2.3 through D.2.10 of this appendix. In most cases, several methods may apply to a specific coastal setting, and in some cases, methods used for the Atlantic coast will differ from those used for the Gulf Coast region. This would be expected for the coastal areas dominated by northeaster coastal flood events, as opposed to those influenced primarily by hurricanes. The objective of this subsection is to provide guidance for developing an appropriate methodology based on coastal settings and available data.

A significant portion of Appendix D is devoted to the presentation of technical methods that were established in previous guidance dating back to 1989. It is important to remember that the objective of this document is to provide updated guidance for developing flood insurance risk zones and maps. These updates are based on recent advancements in coastal engineering and recommendations from the technical panel of coastal experts convened to evaluate existing methodologies and new technical approaches for analyzing the Atlantic and Gulf coasts, previously presented in the 2003 version of Appendix D. The updates from the more developed of the coastal panel recommendations were presented in FEMA Procedure Memorandum No. 37 and serve as the primary basis for the updates in the subsections to follow. When considering the technical approach for a coastal setting, the Mapping Partner must keep in mind that the level of technical analysis should remain consistent with the objective of this document. It is only necessary to obtain the data and conduct the analyses that are required to accomplish this objective. Because there are often several methods available to conduct similar analyses, the Mapping Partner must choose methods that are technically consistent, are applicable for the study setting, use available data, and are appropriate for project resources.

The recommended generalized study methodology is summarized below. There are many well-established methods for Mapping Partners to follow in developing flood insurance risk zones and maps, and they provide the approach that best suits the objectives of this document. It is important, though, to consider all the coastal processes that occur during the base (1-percent-annual-chance) flood event, and to consider what data and technical methods are appropriate for application and update, and what existing data is still valid to use in the determination of flood insurance risk zones and BFEs. At the outset of the study process, the Mapping Partner should begin the onshore analysis by identifying the information that is required to develop the flood insurance risk zones and mapping. This involves identifying all of the physical coastal processes that are likely to contribute to flood hazards in the study area, and their interaction with particular coastal settings in the onshore, nearshore, and offshore environments of the study area. In some cases, this initial review will not resolve all of the questions related to coastal processes and hazard zones. However, the review should identify the data requirements for one or more methods that can be applied to make these determinations.

New or additional data may be required to perform analyses with the updated methodologies presented in this volume. FEMA recently adopted changes to wave runup analyses that require Mapping Partners to analyze directly or convert from the mean to the 2-percent wave runup depth. The conversion of the 2-percent to mean wave runup approximately doubles the total runup depth for hazard zones and BFE determination. Shorelines where previous analyses using the mean wave runup method did not predict runup depths greater than the wave height effects may now warrant further consideration. Further discussion of the 2-percent wave runup analyses can be found in Subsection D.2.8.1 of these guidelines. When a coastal protection structure is present in a study site the Mapping Partner will need to identify the data and methods needed to determine whether the structure will withstand the forces associated with the base flood, or if it requires the application of newly adopted methods to predict failed structure conditions. These are just two examples of the types of changes in the study process that must now factor into the data requirements and technical approach of Mapping Partners, based on the update to Appendix D.

After a review of probable hazards at the shoreline, the Mapping Partner should proceed offshore, considering what data and analyses are required at each level and for each setting within the study area to accomplish the onshore analysis. This will establish the limit of the offshore data and computations necessary to conduct the analyses. In most cases, this limit will correspond to offshore conditions. Once the offshore data requirements for the study are established, the wave data and other information will be brought back onshore to determine the information needed to develop the hazard zones. In other words, the mapping needs are established by progressing from the hazard map to the offshore area, but the analysis proceeds in the direction of the physics — from offshore to onshore.

Different data requirements are associated with different analysis methods. For example, if methods are based on the deep water, unrefracted, significant wave height and peak wave period, it is not necessary to examine the details of the spectrum. If it is not necessary to transform the waves across the surf zone, the surf zone bathymetry is not required for this method. More advanced methods generally require additional data. New methods that have been developed for wave setup and evolving guidance on wave runup methods will require a higher level of data requirements, depending on their significance in the detailed coastal analyses.

It should be noted that the two initial and significant phases of a full coastal restudy to determine the SWELs, the hydrodynamic storm surge inundation modeling and statistical methods for flood frequency analysis, have changed with the evolving world of coastal science and engineering since the first guidance was presented in 1981 for the Tetra Tech surge model (TTSURGE) and the Joint Probability Method (JPM). This surge and statistical guidance was developed and maintained separately from the detailed coastal analyses of wave effects, erosion, and mapping criteria that were initially published in 1989. This update begins to merge the two by including a generalized discussion on data submission requirements of a storm-surge modeling effort

(Subsections D.2.1.2.6 and D.2.12.2) and flood frequency analysis methods and statistical theories (Subsection D.2.3). The results that the Mapping Partner obtains from new hydrodynamic and statistical methods will influence the study methodology for wave effects, erosion assessments, and final hazard zone mapping. Although well documented and applied along floodprone coastal regions, many of the surge modeling and statistical methods have not been fully resolved and documented as NFIP methodologies in a comprehensive set of guidelines. At this time, user guidance for a specific model or method setup and application to a FEMA Flood Map Project are not available in this guidance and will generally be documented and supported independently by the author and/or developer of the model or method.

In selecting analysis methods, logic must be applied to both the overall study (study methodology) and to the selection of methods for each major coastal process to be analyzed in developing flood insurance risk zones (Figure D.2.2-1). The basic logic begins with the definition of objectives, which should focus on the development of flood insurance risk zones at an appropriate resolution and level of accuracy, considering potential damages, the inherent uncertainty in the analyses, schedule, and budget. The geomorphic setting is a key factor in identifying the dominant physical processes that must be analyzed and the appropriate methods for analysis. The potential methods applicable to a given setting may have different data requirements, and the availability of data may influence the selection of methods. Once a methodology (combination of methods and data) has been defined, the Mapping Partner must confirm that the methodology satisfies the study objectives, including time and budget constraints.

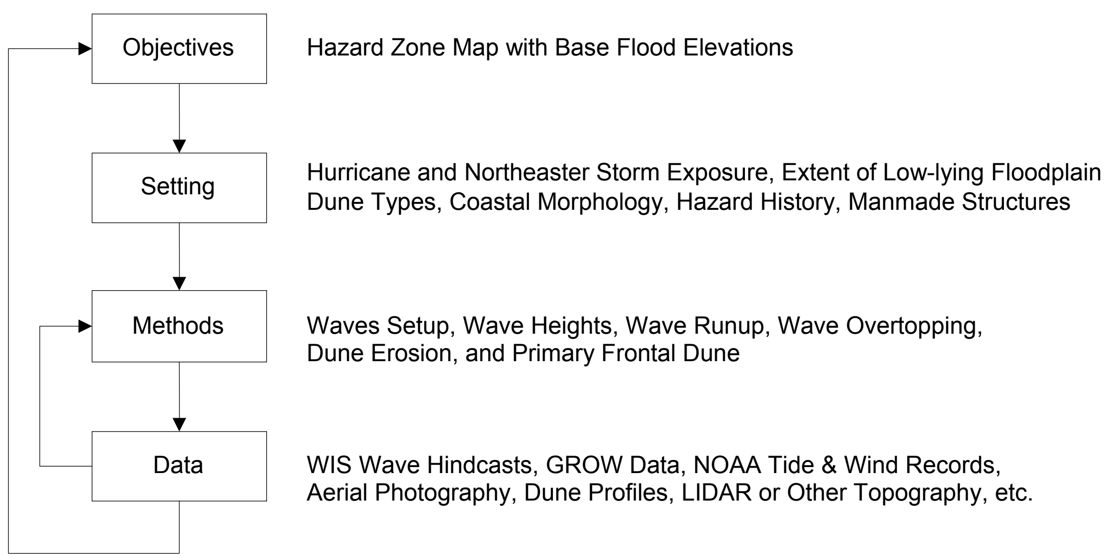

Figure D.2.2-1. Study Methodology Development Considerations

D.2.2.2 Setting

The study area setting and hazard history will determine which methods and data are necessary and/or appropriate. Important considerations include the coastal exposure to hurricanes and northeaster storm events (both on open ocean coasts and in inland bays or other sheltered waters), the shoreline morphology (small or large dune fields, barrier islands, inlets and rivermouths, coastal bluffs or cliffs, etc.), and the shore conditions (topography, development, etc.). Consideration of each of these conditions frames the data requirements and the appropriate analysis methods.

D.2.2.2.1 Open Ocean Coasts, Inland Bays, and Sheltered Waters

A primary consideration is the exposure of the shoreline, which can be classified into three groups: open ocean coasts, inland bays, and other sheltered waters. Open ocean coasts are exposed to the full influence of the Atlantic Ocean or the Gulf of Mexico and include processes such as prolonged and substantial hurricane and northeaster storm surges, large fluctuations of astronomical tides, and large wave effects accompanying storm surges. For most inland bays, storm surge and wave effects result from an exposure to open ocean processes and weather conditions, combined with local processes and weather conditions. In more isolated portions of sheltered waters and inland bays, the waves and flood levels may be primarily caused by local weather and tide conditions and require special hazard analysis considerations, such as timing of peak surge levels with peak wind driven wave effects.

On the open ocean coast, the interrelationships between storm surge and wave processes, such as the influence wave setup exerts upon storm surge flood levels, may be quite complex. Depending upon the scope of the coastal restudy, the simultaneous assessment of wave setup processes is recommended during the hydrodynamic modeling of wind setup from hurricanes and northeasters, to avoid underestimating the contribution of wave setup processes to total stillwater flood levels. This is a key point for the Atlantic and Gulf coasts that can not be addressed properly in Appendix D. The methodologies presented here will be for independent assessments of each process, until the time when Appendix D can be cross-referenced to new guidance for the complex processes involved with hydrodynamic modeling and statistical analyses prior to the more simplified analyses for wave effects and erosion assessments.

In sheltered waters, the waves are typically generated by local weather, which simplifies the interrelationships. As a result, statistical or simulation techniques may be used to analyze these processes. However, tidal amplification, currents, and the effects of river inflows must be considered differently in sheltered waters than larger inland bays. While most methods for open ocean coasts and inland bays are also applicable for sheltered waters, a number of special considerations exist for sheltered waters which have been addressed in the Pacific Coast guidelines.

D.2.2.2.2 Shoreline Profile Settings

The shoreline morphology determines which analysis tools are appropriate for estimating shoreline responses. The general shoreline settings on the Atlantic and Gulf coasts include:

- Sandy beach backed by low or high sand barrier dune formations
- Sandy beach backed by coastal shore protection structures
- Cobble, gravel, shingle, or mixed-grain-size beach
- Erodible bluffs and plateau

- Nonerodible bluffs and cliffs
- Tidal marshes and wetlands.

Details of the specific methods for each setting are given in Subsection D.2.9. Other special considerations for the detailed analyses, due to unique coastal nearshore features, would include the presence of fringing submerged or exposed reefs and rock outcrops, breakwaters, and shore protection structures such as groins and jetties.

Figure D.2.2-2 summarizes key considerations for each of these six settings. In all settings, the existing shoreline conditions must be determined. These are required to determine the present location of the shoreline, the condition of structures, etc. For settings in which seasonal adjustments to beach profiles are needed, the initial profile from which storm-induced changes are calculated should be determined. Profile changes not completed using established erosion assessment methods (such as the 540-square-foot erosion criteria; see Subsection D.2.9) are estimated with appropriate methods or historical documentation to yield a feasible and technically justified eroded profile. If the eroded profile results in dune breaching, structure failure, or bluff recession, then an adjusted final profile must be determined. Wave setup, wave runup, wave overtopping, and wave height overland propagation are determined for the final profile. These results are then used for mapping the flooding hazards.

- For a sandy beach backed by a low sand berm or high sand dune, the 540 ft^2 methodology is appropriate. If the dune is overtopped or breached, then the profile may require additional adjustments to construct a final erosion-adjusted profile before continuing with the wave effect analyses.

- For a sandy beach backed by shore protection structures, the eroded profile is determined from data or other overtopping considerations rather than the 540 SF erosion methodology. The structure may cause local scour and the structure may fail. The final profile based on these processes is then examined for overtopping depths and possibly ponding. In overtopping cases with structures, profile adjustments should consider FEMA policy for stability of fill placed landward of a coastal structure. Fill placed landward of a coastal structure is considered to be stable only to the crest of the structure, and fill placed in excess (or above) the crest of the structure should be eroded and not included in the adjusted profile. This would hold true for the failed structure scenario as well. If the structure in its certified or failed condition has crest elevations at or below the seaward toe of a dune (located adjacent and landward of the structure) or 10-percent-annual-chance flood elevation, then the 540 SF methodology criteria should be considered in making final adjustments to the beach profile.

- For a cobble beach, little analytical guidance is available because many of the cobble beaches on the northeast-Atlantic coast are mixed grain sizes and are difficult to model, while there are no cobble beaches in the mid-Atlantic coast south and into the Gulf Coast region. As a result, observed profiles during large events are used as the basis for determining wave runup, wave overtopping, and possibly ponding.

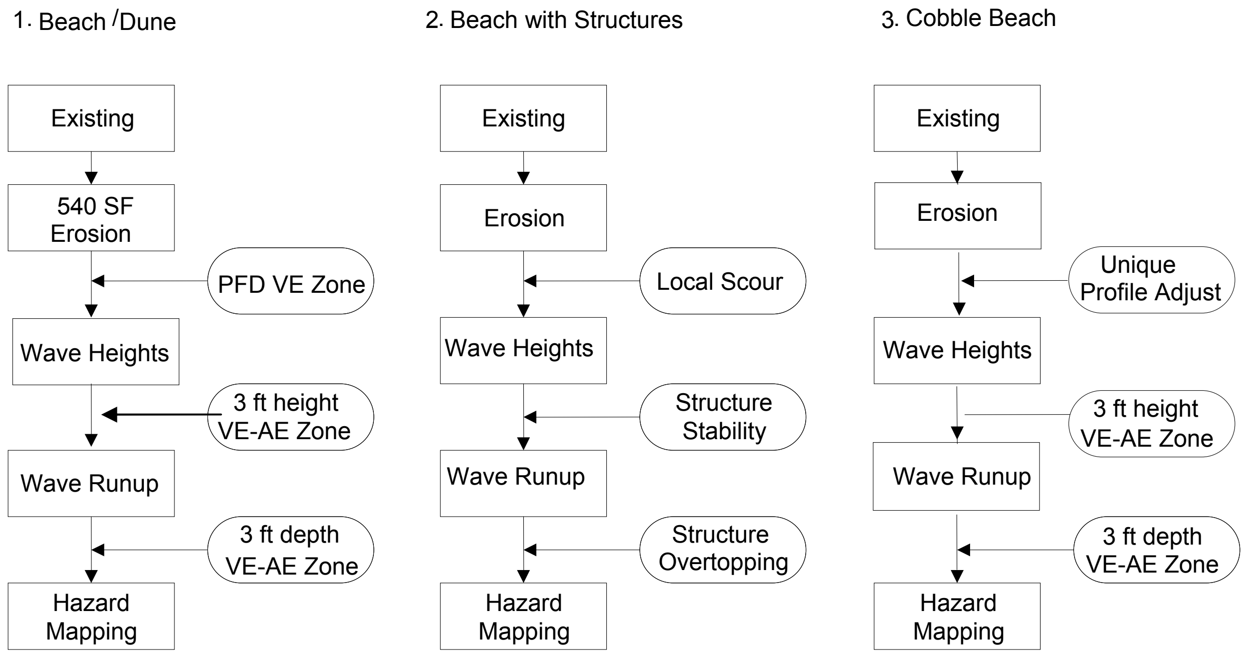

Figure D.2.2-2a. Shoreline Profile Setting Nos. 1 - 3

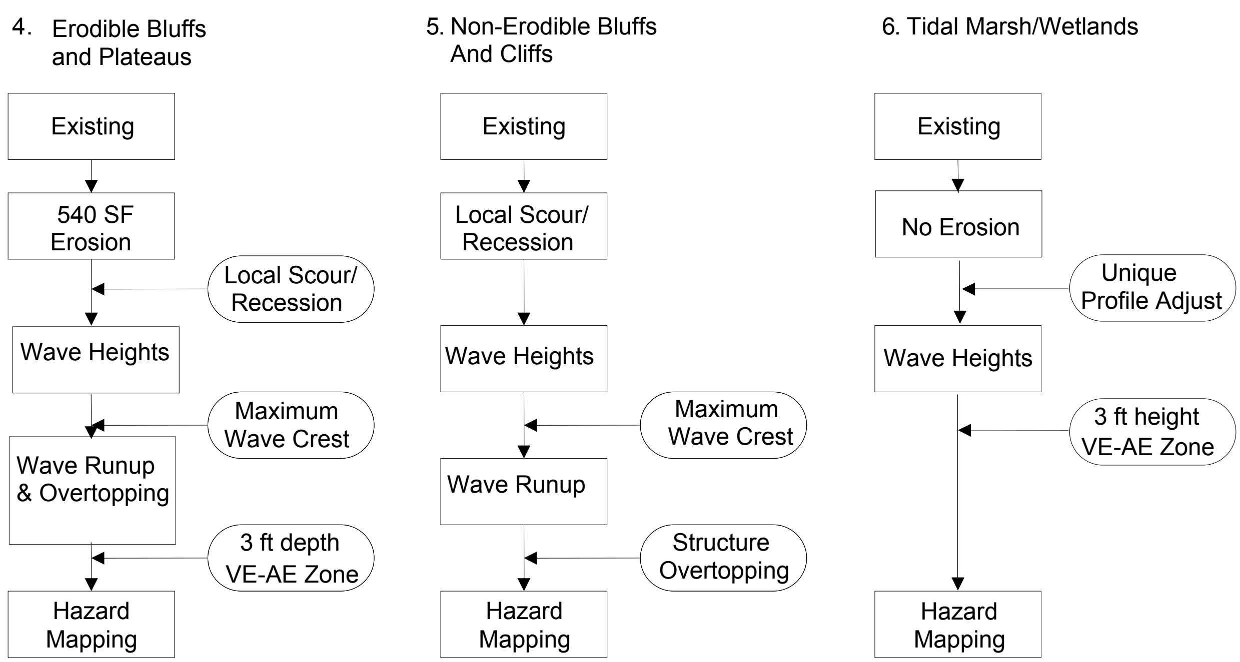

Figure D.2.2-2b. Shoreline Profile Setting Nos. 4 - 6

- For erodible bluffs, the eroded beach profile is determined from use of the 540 SF methodology, if applicable (see Subsection D.2.9), local bluff recession assessments, or historic measurements of storm-induced erosion. The bluff recession is estimated with a bluff erosion model, and bluff toe scour should be considered for possible collapse. The resulting profile is then used to determine the wave runup, wave overtopping, and possibly ponding.

- For non-erodible bluffs, the eroded beach profile is determined from historic measurements of storm-induced erosion or local bluff recession assessments if any are applicable to the bluff-type. This profile is then used to determine the wave runup, wave overtopping, and and possibly ponding.

- For tidal flats and wetlands, it is assumed that there is no erosion over the duration of the base flood event. If historic measurements or data indicate any type of unique profile adjustments (such as in coastal Louisiana marshes), then the profile adjustment should be applied before being used for overland wave propagation modeling of wave effects.

D.2.2.3 Coastal Zones

Figure D.2.2-3 shows the cross-shore divided into four zones. The offshore zone is the area influenced by waves and water levels that are not substantially influenced by bathymetry or topography. The dominant processes in this zone include swell, seas, astronomical tides, and storm surge. The shoaling zone is the area outside the surf zone, where offshore conditions are transformed by interaction with bathymetry or topography. This typically includes the refraction, diffraction, dissipation, and generation of waves, but along the Atlantic and Gulf coasts this area is characterized by fully developed sea conditions.

In these coastal regions, storm waves are local, caused by storms that pass close to or make landfall at the shore, resulting in predominantly shore-perpendicular wave propagation. The surf zone is where waves break as they interact with the bottom. The dominant processes include wave setup, runup, overtopping, erosion, and interaction with structures. The backshore zone is the area outside the normal surf zone (under normal weather conditions), which may be subject to inundation during coastal flooding events. This area has hazards characterized by wave effects such as wave runup, wave overtopping, and overland wave height propagation. The backshore zone is subject to development and is the critical area for determining flood hazards.

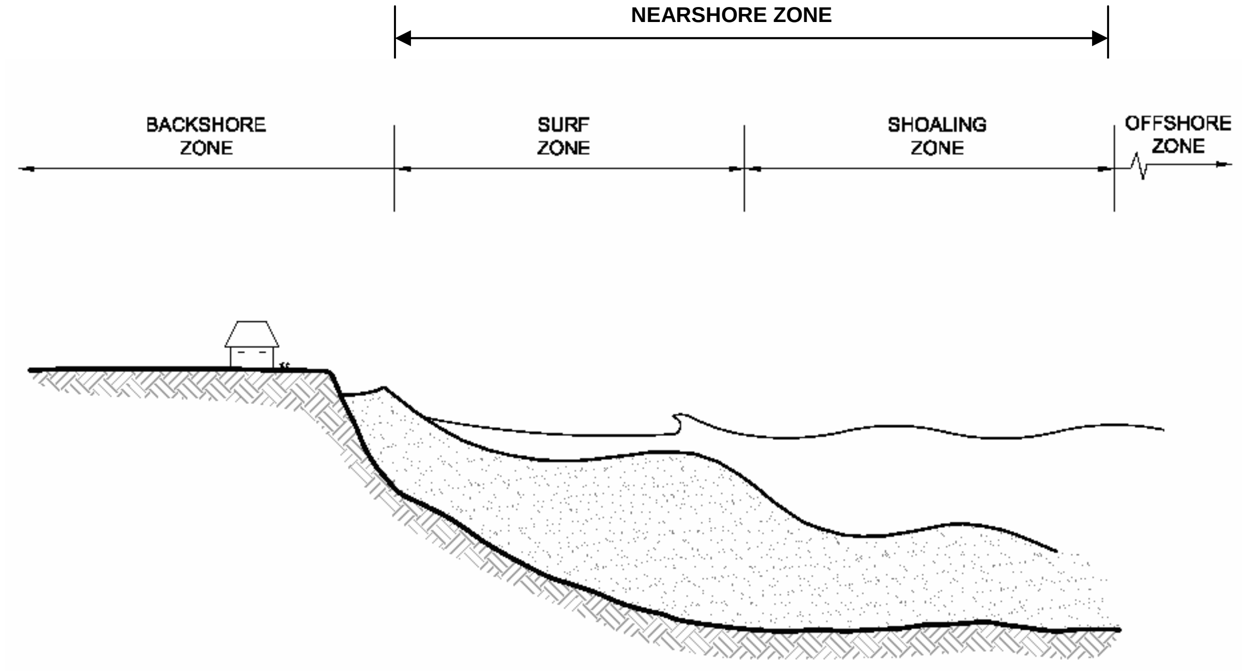

Figure D.2.2-3. Coastal Zones

Figure D.2.2-4 shows the coastal processes as they are referred to in the analysis methods given in Subsections D.2.3 through D.2.10. It should be noted that "offshore" does not necessarily imply deep water conditions, which are defined according to water depth and wave length. Although this deep water condition is typical, an "offshore" designation might only mean that the processes being considered are outside the surf zone. If the offshore zone is not in deep water, then the offshore and shoaling zones are combined.

Except for storm surge elevation determinations (statistical flood levels), the computations made in each zone use data from the preceding zone and pass the results to the next zone. Computations generally start in the offshore zone. Wave information is determined from measurements or hindcasts. Stillwater levels are determined from previously completed flood reports or storm-surge modeling, and are derived from methods described in other guidance materials. The resulting estimates for waves and stillwater levels are then passed to the shoaling zone. The definition of shoaling zone above indicates that this is a zone where bottom friction affects wave movement. Thus, waves will change from deep water to shallow water waves and wave height, length, and period should change.

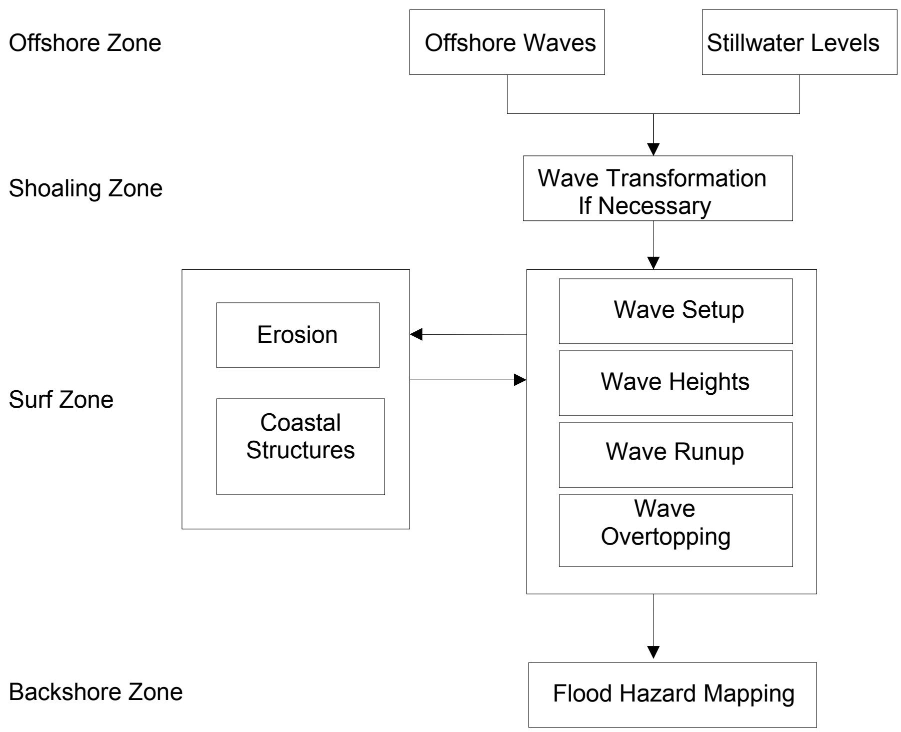

Figure D.2.2-4. Coastal Zones and Processes

In the shoaling zone, the offshore waves are transformed onshore to a water depth outside the breaker line. This requires information for the bathymetry and possibly other factors, such as dissipation over submerged reefs, rock outcrops or barrier islands, mud flats, or wetlands. Several of the surf zone analysis methods require unrefracted deep water wave conditions. After the waves have been transformed across the shoaling zone, the corresponding unrefracted deep water conditions may also be determined. These results are then passed on to the surf zone. Surf zone computations use nearshore bathymetry and either the wave conditions determined outside the breaker line or the unrefracted deep water conditions. Wave setup, wave runup, wave overtopping, wave heights, and erosion are estimated at the shoreline, based on the specific shoreline conditions. These results are then, as appropriate, passed to the backshore zone to determine flood hazards, mostly in the form of overland sheet flow and "splash zone" effects from wave runup, and overland wave height propagation through vegetation and buildings. In the backshore, information from the surf zone is combined with topography and land-use type to calculate the hazard zones and BFEs needed to develop a hazard map.

D.2.2.4 Event and Response Analysis Considerations

On the Atlantic and Gulf coasts, the 1-percent-annual-chance flood has typically been associated with a 1-percent storm event condition defined offshore and transformed to the surf zone. Because increased wave heights and water levels are both associated with the same forcing event, typically a hurricane, this association is reasonable. Statistical tools such as JPM, Monte Carlo, and empirical simulation methods are easily applied in these areas. In the coastal areas of the Northeastern United States, an alternative is to use the statistical relationships of measured water-level conditions for a large coastal region (e.g., Long Island Sound to the U.S.-Canadian border).

An event corresponds to a set of time-dependent wave and water-level conditions taken as a paired data set with a specific duration. This type of analysis is not generally applicable in the Atlantic and Gulf coasts. However, the concept of using a set of conditions to define responses and performing statistical analysis on the responses may be applied in sheltered water. The 1-percent response may be determined at the boundary of any one of the zones shown in Figure D.2.2-3. For example, a 1-percent-annual-chance combination of waves and water levels might be statistically determined in the offshore zone by examining the joint occurrence of waves and water levels. This condition could be transformed onshore, the setup and runup estimated, and the flood insurance risk zone mapped. However, it is unlikely that this single combination of waves and water levels with a 1-percent-annual-chance storm event in the offshore zone corresponds to the 1-percent-annual-chance flood hazard in the backshore zone.

Other combinations of waves and water levels that have a lower probability of occurrence may result in higher levels of flood hazard because of differing responses in the form of runup, setup, erosion, or coastal structure interaction in the backshore zone. These responses are dependent on variables such as wave period and event duration in the sheltered water. The 1-percent-annual-chance flood is defined as the basis for hazard zone mapping by FEMA; thus, the response at the backshore is the condition of interest.

Although the response-based approach is reasonable theoretically, it may not be practical to include all coastal processes in the computations before statistical analysis in the backshore. However, the further the response-based approach can practically be carried onshore, the better the estimate of the 1-percent-annual-chance flood hazard in the backshore zone will be. As a standard methodology, it is recommended that the 1-percent-annual-chance determination be made on total SWELs (consideration of astronomical tide, wind setup for storm surge response, and wave setup). If overtopping occurs, then the determination of the overtopping rate and overtopping volume should be made using the 1-percent-annual-chance runup and the associated storm. These are the most significant hydrodynamic parameters influencing flood hazards. This standard methodology may require modification where processes in the backshore (ponding, riverine flows, etc.) influence the flood hazards (Subsection D.2.4.1.5.4 offers guidance on riverine flow considerations).

D.2.2.5 Selection of Events

Offshore wave conditions, as either measured data or hindcasts, are available for most of the open-coast shorelines of the Atlantic and Gulf coasts. The methods for selecting events are documented in Section D.4.2.5 of the Pacific Coast guidelines and should be consulted for use and application for open ocean coasts, inland bays, or sheltered water. The determination of the 1-percent-annual-chance flood hazards based on a 1-percent-annual-chance response at the shoreline (as opposed to estimating 1-percent-annual-chance storm conditions offshore) provides a more direct connection between the actual causal events and the flooding response and is presented in more detail in the above-referenced subsection.

The 0.2-percent-annual-chance conditions (500-year conditions) are used to map the X zones. The determination of the 0.2-percent conditions and the associated flood hazards is completely analogous to the methods used to determine the 1-percent conditions. These SWELs are generally computed in previously completed storm surge studies or modeling.

D.2.2.6 Summary of Methods

Table D.2.2-1 is a summary of methods presented in Section D.2. This table provides an overview of the available methods and a reference to the appropriate subsection of the document.

Table D.2.2-1. Summary of Methods Presented in Section D.2

Zone/Process	Method	Comments
All Zones	**Statistics (D.2.3)** 1-percent condition – Generalized Extreme Value (GEV) and maximum likelihood fit Peak over threshold with Pareto distribution	Annual maxima are used to determine the 1-percent condition.
	Joint Probability Methods (JPM), Monte Carlo, Empirical Simulation Technique (EST)	JPM, Monte Carlo, or EST are only used in hydrodynamic storm-surge modeling for flood frequencies
Offshore Zone	**Water Level (D.2.4)** Measured Stillwater Level NOAA or USACE tide gauging, storm-surge modeling	In most cases, the measured stillwater level does not include wave setup, represents regional conditions, and is used in all the analyses.
	Sheltered Waters Seiche, tidal amplification, rivers	A number of other factors can influence the water level in sheltered waters.

Table D.2.2-1. Summary of Methods Presented in Section D.2

Zone/Process	Method	Comments
Offshore Zone	**Waves (D.2.5)** Measured NDBC, CDIP Hindcast GROW, WIS, WAVEWATCH III	The use of significant wave conditions (height, period, direction, storm duration) or directional spectra depends upon the choice of the methods selected for determining setup, runup, and overtopping. The wave record must be long enough (30 years or longer) to reasonably estimate the 1-percent- annual-chance condition.
	Wave Generation 2-D models CEM parametric model	Wave generation methods are only applicable in combined surge modeling, sheltered water, or a regional-scale offshore model.
Shoaling Zone	**Wave Transformations (D.2.5)** Straight and parallel contours shoaling and Snell's Law Spectral methods transformation coefficients, CDIP Nearshore transformations 2-D spectral and time domain models Sheltered waters seiching, inlets	Numerical models are typically only required for complex bathymetry

Table D.2.2-1. Summary of Methods Presented in Section D.2

Zone/Process	Method	Comments
Surf Zone	**Wave Setup, Overland Wave Heights, and Wave Runup (D.2.6-D.2.8)** Beaches DIM parametric or numerical for wave setup Wave setup using advanced wave models - Boussinesq	DIM methods combine wave setup and runup. Parametric method only requires significant wave height. Advanced models are only necessary for complex conditions.
	Structures van der Meer, CEM Overland WHAFIS 3.0 Runup RUNUP 2.0 at 2-percent runup SPM for vertical structures/bluffs TAW for sloped structures	
Surf Zone and Backshore Zone	**Erosion (D.2.9)** Beaches Geometric Models – 540 SF method Shore Protection Structures CEM local scour equations Cobble Beaches Observed storm profiles Erodible Bluffs 540 SF method or bluff recession Non-Erodible Bluffs and Cliffs No erosion, local scour or bluff recession Tidal Flats and Wetlands No erosion – unique profile adjustments if known	Atlantic and Gulf Coast “540 Rule” is primary methodology recommended for the Atlantic and Gulf coasts.

Table D.2.2-1. Summary of Methods Presented in Section D.2

Zone/Process	Method	Comments
Backshore Zone	**Overtopping (D.2.8)** Beaches CEM Structures CEM, Besley	30 foot splash zone minimum requirement if no mean overtopping rates known 1.0 CFS mean overtopping rate needed for splash zone Wave runup overtopping limited to 3 foot above crest of dune ridge or structure regardless of runup elevation
Backshore Zone	**Overland Flow (D.2.5)** Cox and Machemehl, WHAFIS	New velocity hazard zone considered for sheet flow greater than 200 CFS
Backshore Zone	**Hazard Indicators (D.2.11)** 3-foot wave runup depth Overtopping splash distance 3-foot wave height Primary frontal dune landward heel/limit	

D.2.3 Flood Frequency Analysis Methods

This subsection outlines general features of statistical methods used in a coastal study, including basic statistical tools that are frequently needed. When an extreme value analysis of annual maxima is performed, it is recommended that the Generalized Extreme Value (GEV) Distribution be adopted, with parameters estimated by the Method of Maximum Likelihood. The discussion in this subsection is illustrative only; guidelines for application of these tools in specific instances are provided in other sections of this appendix.

Performing flood frequency analyses requires a good understanding of probability theory and statistics, and in most instances sound engineering judgment. This subsection summarizes some basic concepts in probability and statistics and also throws light on analytical methods relevant to flood life-cycle studies. This is by no means exhaustive and users should consult the texts and articles referred to in this subsection for detailed coverage. There is no cookbook method for determining storm return periods; flood frequency analyses should be done by experienced modelers and in close collaboration with FEMA Study Representatives. At the outset of each study, the Mapping Partner should document all available data for discussion with the FEMA Study Representative, and for review and comparison with other studies of a similar nature. This should include the extent of available data in both space and time. In particular, it is noted that at the time of this writing a great deal of work is ongoing as a result of the Katrina disaster, which is expected to lead to development of improved data sets. The Mapping Partner should research recent work post-dating these guidelines, sponsored by the U.S. Army Corps of Engineers, the National Oceanic and Atmospheric Administration (NOAA), FEMA, and other agencies, to help establish the best input for a study.

D.2.3.1 The Base Flood

The primary goal of a coastal study is to determine the flood levels throughout the study area that have a 1-percent chance of being exceeded in any given year. The level that is exceeded at this rate at a given point is called the base flood level, and has a probability of 0.01 to be equaled or exceeded in any year; on the average, this level is exceeded once in 100 years and is commonly called the 100-year flood.

The base flood might result from a single flood process or from a combination of processes. For example, astronomic tide, storm surge, and storm waves may combine to produce the total high water runup level. There is no one-to-one correspondence between the BFE and any particular storm or other flood-producing mechanism. The level may be produced by any number of mechanisms, or by the same mechanism in different instances. For example, an incoming wave with a particular height and period may produce the base flood runup, as might a quite different wave with a different combination of height and period.

Furthermore, the flood hazard maps produced as part of a FEMA Flood Map Project do not necessarily display, even locally, the spatial variation of any realistic physical hydrologic event. For example, the base flood levels just outside and just inside an inlet will not generally show the same relation to one another as they would during the course of any real physical event, because the inner waterway may respond most critically to storms of an entirely different character from those that affect the outer coast. Where a flood hazard arises from more than one source, the mapped level is not the direct result of any single process, but is a construct derived from the statistics of all sources. Note then that the base flood level is an abstract concept based as much on the statistics of floods as on the physics of floods.

Because the base flood level cannot be rigorously associated with any particular storm, it is a mistake to think of some observed event as having been the base flood event. A more intense storm located at a greater distance might produce the same flood level, or the same flood level might be produced by an entirely different mechanism. Furthermore, if a particular storm were, in fact, the so-called "100-year event," it might not cause base flood effects everywhere, but only at a particular point.

The base flood level is a consequence solely of the areawide flooding mechanisms recognized for a particular location. That is, there may be mechanisms that are not taken into account, but that could also produce water levels comparable to the base flood level or that could contribute to the base flood level. For example, tsunamis are not recognized as areawide flood sources for the Atlantic coast. However, they occur in all oceans, and even the Atlantic coast is vulnerable to tsunami attack at some frequency. The Great Lisbon earthquake of 1755 (with magnitude approaching 9) produced a large Atlantic tsunami that was felt in the New World. Similarly, advances in science may from time to time reveal new flood mechanisms that had not previously been recognized.

D.2.3.2 Event vs. Response Statistics

The flood level experienced at any coastal site is the complicated result of a large number of interrelated and interdependent factors. For example, coastal flooding by wave runup depends upon both the local waves and the level of the underlying still water on which they ride. That stillwater elevation (SWEL), in turn, depends upon the varying astronomic tide and the contribution of the transient storm surge. The wave characteristics that control runup and crest elevation include amplitude, period, and direction, all of which depend on the meteorological characteristics of the generating storm, including its location and its time-varying wind and pressure fields. Furthermore, the resulting wave characteristics are affected by variations of water depth over their entire propagation path, and thus depend also on the varying local tide and surge. Still further, the beach profile is variable, changing in response to wave-induced erosion and causing variation in the wave transformation and runup behavior; catastrophic erosion of a barrier might also cause a fundamental change in stillwater surge elevations. All of these interrelated factors may be significant in determining the coastal base flood level. Whatever methods are used, simplifying assumptions are inevitable, even in the most ambitious response-based study, which attempts to simulate the full range of important processes over time.

These guidelines offer insight and methods to address the complexity of the coastal flood process in a reasonable way. However, the inevitable limitations of the guidance must be kept in mind. No fixed set of rules or "cookbook procedures" can be appropriate in all cases, and the Mapping Partner must be alert to special circumstances that violate the assumptions of the methodology.

D.2.3.2.1 Event-Selection Method

A great simplification is made if one can identify a single event (or a small number of events) that produces a flood thought to approximate the base flood. This might be possible if, for example, a single event parameter is believed to dominate the final elevations, so the 1-percent value of that particular item might suffice to determine the base flood. In its simplest form for wave runup, for example, one might identify a significant wave height thought have only a 1-percent chance of being exceeded, and then to follow this single wave as it would be transformed in propagation and as it would run up the beach. This is the *event-selection method*. Used with caution, this method may allow reasonable estimates to be made with minimal cost. It is akin to the concept of a design storm, or to constructs such as the standard project or probable maximum storms.

The inevitable difficulty with the event-selection method is that multiple parameters are always important, and it may not be possible to assign a frequency to the result with any confidence, because unconsidered factors always introduce uncertainty. In the case of runup, for example, smaller waves with longer periods might produce greater runup than the largest waves selected for study. A slight generalization of the event-selection method, often used in practice, is to consider a small number of parameters – say wave height, period, and direction – and attempt to establish a set of alternative, "100-year" combinations of these parameters. The RUNUP 2.0 program, for example, considers variations of height and period around nominal mean values. More general alternatives might be, say, pairs of height and period from each of three directions, with each pair thought to represent the 1-percent-annual-chance threat from that direction, and with each direction thought to be associated with independent storm events. Each such combination would then be simulated as a selected "event", with the largest flood determined at a particular site being chosen as the base flood. The probable result of this procedure would be to seriously underestimate the true base flood level by an unknown amount. This can be seen easily in a hypothetical case in which all three directional wave height and period pairs resulted in about the same flood level. Rather than providing reassurance that the computed level represents a good approximation of the base flood level, such a result would show the opposite – the computed flood would not be at the base flood level, but would instead approximate the 33-year level, having been found to result once in 100 years from each of three independent sources, for a total of three times in 100 years. It is not possible to salvage this general scheme in any rigorous way – say by choosing three 300-year height and period combinations, or any other finite set based on the relative magnitudes of their associated floods – because there always remain other combinations of the multiple parameters that will contribute to the total rate of occurrence of a given flood level at a given point, by an unknown amount.

D.2.3.2.2 Response-based Approach

With the advent of powerful and economical computers, a preferred approach that considers all (or most) of the contributing processes has become practical; this is the *response-based approach*. In the response-based approach, one attempts to simulate the full complexity of the physical processes controlling flooding, and to derive flood statistics from the results (the local response) of that complex simulation. For example, given a knowledge of local hurricane climatology, one might simulate a large number of either historical or hypothetical storms in such a way as to create an equivalent long period of record, from which the statistics of storm surge elevations could be derived. In a wave-dominated environment, if given the history of offshore waves in terms of height, period, and direction, one might compute the runup response of the entire time series, using all of the data and not prejudging which waves in the record might be most important. With knowledge of the astronomic tide, the entire processes could be repeated with different assumptions regarding tidal amplitude and phase. Further, with knowledge of the erosion process, storm-by-storm erosion of the beach profile might also be considered, so that its feedback effect on wave behavior could be taken into account.

At the end of this process, one would have developed a long-term simulated record of surge or runup, or both, at the site, which could then be analyzed to determine the base flood level. Clearly, successful application of such a response-based approach requires a tremendous effort to characterize the individual component processes and their interrelationships, and a great deal of computational power to carry out the intensive calculations.

The response-based approach is preferred for all Gulf and Atlantic coast studies.

D.2.3.3 General Statistical Methods

This subsection summarizes the statistical methods that will be most commonly needed in the course of a FEMA Flood Map Project to establish the BFE. Two general approaches can be taken, depending on the availability of observed flood data for the site. The first, preferred, approach is used when a reasonably long observational record is available, such as 30 years or more of flood records or other data. In this *extreme value analysis* approach, the data are used to establish a *probability distribution* that is assumed to describe the flooding process, and that can be evaluated by using the data to determine the flood elevation at any frequency. This approach can be used for the analysis of wind and tide gage data, for example, or for a sufficiently long record of a computed parameter such as wave runup.

The second approach is used when an adequate observational record of flood levels does not exist. In this case, it may be possible to simulate the flood process using hydrodynamic models driven by meteorological or other processes for which adequate data exist. That is, the hydrodynamic model (perhaps describing storm surge or waves) provides the link between the known statistics of the generating forces, and the desired statistics of flood levels. These *simulation* methods are relatively complex and will be used when no acceptable, more economical alternative exists. Only a general description of these methods is provided here; full documentation of the methods can be found in the user's manuals provided with the individual simulation models. The manner in which the base flood level is derived from a simulation will depend upon the manner in which the input forcing disturbance is defined. If the input is a long time series, then the base flood level might be obtained using an extreme value analysis of the simulated process. If the input is a set of empirical storm parameter distributions, then the base flood level might be obtained by a method such as joint probability or Monte Carlo, as discussed later in this subsection.

The present discussion begins with the basic ideas of probability theory and introduces the concept of a continuous probability distribution. Distributions important in practice are summarized, including the extreme value family in particular. Methods to fit a distribution to an observed data sample are discussed, with specific recommendations for FEMA Flood Map

Project applications. A list of suggested additional information resources is included at the end of the subsection.

D.2.3.3.1 Elementary Probability Theory

Probability theory deals with the characterization of random events and, in particular, with the likelihood of the occurrence of particular outcomes. The word "probability" has many meanings, and there are conceptual difficulties with all of them in practical applications such as flood studies. The common frequency notion is assumed here: the probability of an event is equal to the fraction of times it would occur during the repetition of a large number of identical *trials*. For example, if one considers an annual storm season to represent a trial, and if the event under consideration is occurrence of a flood exceeding a given elevation, then the annual probability of that event is the fraction of years in which it occurs, in the limit of an infinite period of observation. Clearly, this notion is entirely conceptual, and cannot truly be the source of a probability estimate.

An alternate measure of the likelihood of an event is its expected *rate of occurrence*, which differs from its probability in an important way. Whereas probability is a pure number and must lie between zero and one, a rate of occurrence is a measure with physical dimensions (reciprocal of time) that can take on any value, including values greater than one. In many cases, when one speaks of the probability of a particular flood level, one actually means its rate of occurrence; thinking in terms of physical rate can help to clarify an analysis.

To begin, a number of elementary probability rules are reviewed. If an event occurs with probability P in some trial, then it fails to occur with probability $Q = 1 - P$. This is a consequence of the fact that the sum of the probabilities of all possible results must equal unity, by the definition of total probability:

$$\sum_i P(A_i) \equiv 1 \qquad \text{(D.2.3-1)}$$

in which the summation is over all possible outcomes of the trial.

If *A* and *B* are two events, the probability that either *A* or *B* occurs is given by:

$$P(A \text{ or } B) = P(A) + P(B) - P(A \text{ and } B) \qquad \text{(D.2.3-2)}$$

If *A* and *B* are mutually exclusive, then the third term on the right-hand side is zero, and the probability of obtaining either outcome is the sum of the two individual probabilities.

If the probability of *A* is contingent on the prior occurrence of *B*, then the *conditional probability* of *A* given the occurrence of *B* is defined to be:

$$P(A \mid B) \equiv \frac{P(AB)}{P(B)} \qquad \text{(D.2.3-3)}$$

in which *P(AB)* denotes the probability of both *A* and *B* occurring.

If *A* and *B* are *stochastically independent*, *P(A|B)* must equal *P(A)*. Then the definition of conditional probability just stated gives the probability of occurrence of both *A* and *B* as:

$$P(AB) = P(A)P(B) \tag{D.2.3-4}$$

This expression generalizes for the joint probability of any number of independent events, as:

$$P(ABC...) = P(A)P(B)P(C)... \tag{D.2.3-5}$$

As a simple application of this rule, consider the chance of experiencing at least one base flood (P = 0.01) in 100 years. This is 1 minus the chance of experiencing no such flood in 100 years. The chance of experiencing no such flood in 1 year is 0.99, and if it is granted that floods in different years are independent, then the chance of not experiencing such a flood in 100 years is 0.99^{100} according to Equation D.2.3-5, or 0.366. Consequently, the chance of experiencing at least one base flood in 100 years is 1 – 0.366 = 0.634, or only about 63 percent.

D.2.3.3.2 Distributions of Continuous Random Variables

A continuous random variable can take on any value from a continuous range, not just a discrete set of values. The instantaneous ocean surface elevation at a point is an example of a continuous random variable; so, too, is the annual maximum water level at a point. If such a variable is observed a number of times, a set of differing values *distributed* in some manner over a range is found; this fact suggests the idea of a *probability distribution*. The observed values are a data *sample*.

We define the *probability density function*, PDF, of *x* to be *f(x)*, such that the probability of observing the continuous random variable x to fall between *x* and *x + dx* is *f(x) dx*. Then, in accordance with the definition of total probability stated above:

$$\int_{-\infty}^{\infty} f(x)\ dx = 1 \tag{D.2.3-6}$$

If we take the upper limit of integration to be the level L, rather than infinity, then we have the definition of the *cumulative distribution function*, CDF, denoted by *F(x)*, which specifies the probability of obtaining a value of L or less:

$$F(x \le L) \equiv \int_{-\infty}^{L} f(x)\ dx \tag{D.2.3-7}$$

It is assumed that the observed set of values, the sample, is derived by random sampling from a *parent distribution*. That is, there exists some unknown function, *f(x)*, from which the observed sample is obtained by random selection. No two samples taken from the same distribution will be exactly the same. Furthermore, random variables of interest in engineering cannot assume values over an unbounded range, as suggested by the integration limits in the expressions shown above. In particular, the lower bound for flood elevation at a point can be no less than ground level, windspeed cannot be less than zero, and so forth. Upper bounds also exist, but they cannot be precisely specified; whatever occurs can be exceeded, if only slightly. Consequently, the usual approximation is that the upper bound of a distribution is taken to be infinity, while a lower bound might be specified.

If the nature of the parent distribution can be inferred from the properties of a sample, then the distribution provides the complete statistics of the variable. If, for example, one has 30 years of annual peak flood data, and if these data can be used to specify the underlying distribution, then one can easily obtain the 10-, 2-, 1-, and 0.2-percent-annual-chance flood levels by computing x such that $F(x)$ is 0.90, 0.98, 0.99, and 0.998, respectively.

The entirety of the information contained in the PDF can be represented by its moments. For the normal distribution, the *mean*, μ, specifies the *location* of the distribution and is the first moment about the origin:

$$\mu = \int_{-\infty}^{\infty} x \, f(x) \, dx \tag{D.2.3-8}$$

Two other common measures of the location of the distribution are the mode, which is the value of x for which f is maximum, and the median, which is the value of x for which F is 0.5.

The spread of the distribution is measured by its variance, σ^2, which is the second moment about the mean:

$$\sigma^2 = \int_{-\infty}^{\infty} (x-\mu)^2 f(x) \, dx \tag{D.2.3-9}$$

The *standard deviation*, σ, is the square root of the variance.

The third and fourth moments are called the skew and the kurtosis, respectively; still higher moments fill in more details of the distribution shape, but they are seldom encountered in practice. If the variable is measured about the mean and is normalized by the standard deviation, then the *coefficient of skewness*, measuring the asymmetry of the distribution about the mean, is:

$$\eta_3 = \int_{-\infty}^{\infty} \left(\frac{x-\mu}{\sigma}\right)^3 f(x) \, dx \tag{D.2.3-10}$$

and the *coefficient of kurtosis*, measuring the peakedness of the distribution, is:

$$\eta_4 = \int_{-\infty}^{\infty} \left(\frac{x-\mu}{\sigma}\right)^4 f(x) \, dx \tag{D.2.3-11}$$

These four parameters are properties of the unknown distribution, not of the data sample. However, the sample has its own set of corresponding parameters. For example, the *sample mean* is:

$$\overline{x} = \frac{1}{n} \sum_i x_i \tag{D.2.3-12}$$

which is the average of the sample values. The *sample variance* is:

$$s^2 = \frac{1}{n-1}\sum_i (x_i - \bar{x})^2 \qquad \text{(D.2.3-13)}$$

while the sample skew and kurtosis are:

$$C_S = \frac{n}{(n-1)(n-2)s^3}\sum_i (x_i - \bar{x})^3 \qquad \text{(D.2.3-14)}$$

$$C_K = \frac{n(n+1)}{(n-1)(n-2)(n-3)s^4}\sum_i (x_i - \bar{x})^4 \qquad \text{(D.2.3-15)}$$

Note that in some literature the kurtosis is reduced by 3, so the kurtosis of the normal distribution becomes zero; it is then called the *excess* kurtosis.

D.2.3.3.3 Stationarity

Roughly speaking, a random process is said to be stationary if it is not changing over time, or if its statistical measures remain constant. Many statistical tests can be performed to help determine whether a record displays a significant trend that might indicate nonstationarity. A simple test that is very easily performed is the Spearman Rank Order Test. This is a nonparametric test operating on the ranks of the individual values sorted in both magnitude and time. The Spearman *R* statistic is defined as:

$$R = 1 - \frac{6\sum_i (d_i)^2}{n(n^2-1)} \qquad \text{(D.2.3-16)}$$

in which *d* is the difference between the magnitude rank and the sequence rank of a given value. The statistical significance of *R* computed from Equation D.2.3-16 can be found in published tables of Spearman's *R* for *n — 2* degrees of freedom.

D.2.3.3.4 Correlation Between Series

Two random variables may be statistically independent of one another, or some degree of interdependence may exist. Dependence means that knowing the value of one of the variables permits a degree of inference regarding the value of the other. Whether paired data (*x,y*), such as simultaneous measurements of wave height and period, are interdependent or correlated is usually measured by their *linear correlation coefficient*:

$$r = \frac{\sum_i (x_i - \bar{x})(y_i - \bar{y})}{\sqrt{\sum_i (x_i - \bar{x})^2}\sqrt{\sum_i (y_i - \bar{y})^2}} \qquad \text{(D.2.3-17)}$$

This correlation coefficient indicates the strength of the correlation. An *r* value of +1 or -1 indicates perfect correlation, so a cross-plot of *y* versus *x* would lie on a straight line with positive or negative slope, respectively. If the correlation coefficient is near zero, then such a plot would show random scatter with no apparent trend.

D.2.3.3.5 Convolution of Two Distributions

If a random variable, *z*, is the simple direct sum of the two random variables *x* and *y*, then the distribution of *z* is given by the convolution integral:

$$f_z(z) = \int_{-\infty}^{\infty} f_x(T) f_y(z - T)\ dT \qquad \text{(D.2.3-18)}$$

in which subscripts specify the appropriate distribution function. This equation can be used, for example, to determine the distribution of the sum of storm surge and tide under the assumptions that surge and tide are independent and that they add linearly without any nonlinear hydrodynamic interaction.

D.2.3.3.6 Important Distributions

Many statistical distributions are used in engineering practice. Perhaps the most familiar is the *normal* or *Gaussian* distribution. We discuss only a small number of distributions, selected according to probable utility in a FEMA Flood Map Project. Although the normal distribution is the most familiar, the most fundamental is the uniform distribution.

D.2.3.3.6.1 Uniform Distribution

The uniform distribution is defined as constant over a range, and zero outside that range. If the range is from *a* to *b*, then the probability distribution function (PDF) is:

$$f(x) = \frac{1}{b-a}, \quad a \le x < b, \quad 0 \text{ otherwise} \qquad \text{(D.2.3-19)}$$

which, within its range, is a constant independent of *x*; this is also called a *top-hat* distribution.

The uniform distribution is especially important because it is used in drawing random samples from all other distributions. A random sample drawn from a given distribution can be obtained by first drawing a random sample from the uniform distribution defined over the range from 0 to 1. Then set *F(x)* equal to this value, where *F* is the cumulative distribution to be sampled. The desired value of *x* is obtained by inverting the expression for *F*.

Sampling from the uniform distribution is generally done with a random number generator returning values on the interval from 0 to 1. Most programming languages and spreadsheets have such a function built in, as do many calculators. However, not all such standard routines are satisfactory. While adequate for drawing a small number of samples, many widely used standard routines fail statistical tests of uniformity. If a critical application requires a large number of samples, as might be the case when performing a large Monte Carlo simulation (see Subsection D.2.3.6.3), these simple standard routines may be inadequate. A good discussion of this matter, including examples of high-quality routines, can be found in the book *Numerical Recipes*, included in Subsection D.2.3.7, *Additional Resources*.

D.2.3.3.6.2 Normal or Gaussian Distribution

The normal or Gaussian distribution, sometimes called the bell-curve, has a special place among probability distributions. Consider a large number of large samples drawn from some unknown distribution. For each large sample, compute the sample mean. The distribution of those means tends to follow the normal distribution, a consequence of the *central limit theorem*. Despite this, the normal distribution does not play a central direct role in hydrologic frequency analysis. The standard form of the normal distribution is:

$$f(x) = \frac{1}{\sigma(2\pi)^{1/2}} e^{-\frac{(x-\mu)2}{2\sigma^2}}$$

$$F(x) = \frac{1}{2} + \frac{1}{2}\operatorname{erf}\left(\frac{x-\mu}{\sqrt{2}\sigma}\right) \tag{D.2.3-20}$$

D.2.3.3.6.3 Rayleigh Distribution

The Rayleigh distribution is important in the theory of random wind waves. Unlike many distributions, it has some basis in theory; Longuet-Higgins (1952) showed that with reasonable assumptions for a narrow banded wave spectrum, the distribution of wave height will be Rayleigh. The standard form of the distribution is:

$$f(x) = \frac{x}{b^2} e^{-\frac{x^2}{2b^2}}$$

$$F(x) = 1 - e^{-\frac{x^2}{2b^2}} \tag{D.2.3-21}$$

The range of x is positive, and the scale parameter $b > 0$. In water wave applications, $2b^2$ equals the mean square wave height. The mean and variance of the distribution are given by:

$$\mu = b\sqrt{\frac{\pi}{2}}$$

$$\sigma^2 = b^2\left(2 - \frac{\pi}{2}\right) \tag{D.2.3-22}$$

The skew and kurtosis of the Rayleigh distribution are constants (approximately 0.63 and 3.25, respectively) but are of little interest in applications here.

An application of the Rayleigh distribution in coastal flood studies is the estimation of the 2-percent-annual-chance runup level, given the mean level computed by the RUNUP 2.0 program. There is empirical evidence that the runup is Rayleigh distributed, so that the ratio between the

2-percent and 50-percent runup levels can be computed from Equation D.2.3-21. This topic is discussed in section D.2.8.

D.2.3.3.6.4 Extreme Value Distributions

Many distributions are in common use in engineering applications. For example, the log-Pearson Type III distribution is widely used in hydrology to describe the statistics of precipitation and stream flow. For many such distributions, there is no underlying justification for their use other than flexibility in mimicking the shapes of empirical distributions. However, there is a particular family of distributions that is recognized as most appropriate for extreme value analyses and that has some theoretical justification. This group consists of the so-called *extreme value distributions*.

Among the well-known extreme value distributions are the *Gumbel* distribution and the *Weibull* distribution. Both of these are candidates for FEMA Flood Map Project applications and have been widely used with success in similar applications. Significantly, these distributions (and others, including the Rayleigh) are subsumed under a more general distribution, the *generalized extreme value* (GEV) distribution, given by:

$$f(x)=\frac{1}{b}\left\{1+c\left(\frac{x-a}{b}\right)\right\}^{-\frac{1}{c}-1} e^{-(1+c(x-a)/b))^{-1/c}}$$

$$\text{for } -\infty < x \le a-\frac{b}{c} \text{ with } c<0 \text{ and } a-\frac{b}{c} \le x < \infty \text{ with } c>0$$

$$f(x)=\frac{1}{b}e^{-e^{-(x-a)/b}}e^{-(x-a)/b} \qquad \text{for } -\infty \le x < \infty \qquad \text{with } c=0 \tag{D.2.3-23}$$

The cumulative distribution is given by the expressions:

$$F(x)=e^{-(1+c(x-a)/b))^{-1/c}}$$

$$\text{for } -\infty < x \le a-\frac{b}{c} \text{ with } c<0 \text{ and } a-\frac{b}{c} \le x < \infty \text{ with } c>0$$

$$F(x)=e^{-e^{-(x-a)/b}} \qquad -\infty \le x < \infty \qquad \text{with } c=0 \tag{D.2.3-24}$$

In these expressions, a, b, and c are the *location*, *scale*, and *shape* factors, respectively. This distribution includes the *Frechet* (Type 2) distribution for $c > 0$ and the Weibull (Type 3) distribution for $c < 0$. If the limit of the exponent of the exponential in the first forms of these distributions is taken as c goes to 0, then the simpler second forms are obtained, corresponding to the Gumbel (Type 1) distribution. Note that the Rayleigh distribution is a special case of the Weibull distribution, and so is also encompassed by the GEV distribution.

The special significance of the members of the extreme value family is that they describe the distributions of the extremes drawn from other distributions. That is, given a large number of samples drawn from an unknown distribution, the extremes of those samples tend to follow one of the three types of extreme value distributions, all incorporated in the GEV distribution. This is analogous to the important property of the normal distribution that the means of samples drawn from other distributions tend to follow the normal distribution. If a year of water levels is considered to be a sample, then the annual maximum, as the largest value in the sample, is an extreme and may tend to be distributed according to the statistics of extremes.

D.2.3.3.6.5 Pareto Distribution

If for some unknown distribution the sample extremes are distributed according to the GEV distribution, then the set of sample values exceeding some high threshold tends to follow the Pareto distribution. Consequently, the GEV and Pareto distributions are closely related in a dual manner. The Pareto distribution is given by:

$$F(y) = 1 - \left(1 + \frac{cy}{\tilde{b}}\right)^{-1/c} \quad \text{for} \quad y = x - u$$
$$\text{with} \quad \tilde{b} = b + (u - a) \tag{D.2.3-25}$$

where u is the selected threshold. In the limit as c goes to zero, this reduces to the simple expression:

$$F(y) = 1 - e^{-y/\tilde{b}} \qquad \text{for} \quad y > 0 \tag{D.2.3-26}$$

The Pareto distribution is useful in describing situations where equilibrium can be found between the occurrence of large and small values; for example, there are few category 5 storms but many tropical storms.

D.2.3.3.6.5 Poisson Distribution

The Poisson distribution – a discrete distribution – is especially important in some applications, because it describes a process in which events occur at a known average rate, but with no memory of the last occurrence. Examples include such processes as radioactive decay and, of interest here, might include the number of hurricanes occurring in a year at some site. If hurricanes occur at some long-term average rate (storms per year at the site), and if the occurrence of one storm is independent of any other occurrence, then the process may be Poisson. The Poisson distribution is given by:

$$f(k:\lambda) = \frac{e^{-\lambda}\lambda^k}{k!} \tag{D.2.3-27}$$

where f is the probability of experiencing exactly k occurrences in an interval if λ is the number expected to occur in that interval, equal to the interval length multiplied by the average rate. The Poisson distribution is important in coastal applications of the Empirical Simulation Technique (EST) method for estimation of surge frequency.

D.2.3.4 Data Sample and Estimation of Parameters

Knowing the distribution that describes the random process, one can directly evaluate its inverse to give an estimate of the variable at any recurrence rate; that is, at any value of 1-F. If the sample consists of annual maxima (see the discussion in Subsection D.2.3.5), then the 1-percent-annual-chance value of the variable is that value for which F equals 0.99, and similarly for other recurrence intervals. To specify the distribution, two things are needed. First, an appropriate form of the distribution must be selected from among the large number of candidate forms found in wide use. Second, each such distribution contains a number of free parameters (generally from one to five, with most common distributions having two or three parameters) that must be determined.

It is recommended that the Mapping Partner adopt the GEV distribution for FEMA Flood Map Project applications for reasons outlined earlier: extremes drawn from other distributions (including the unknown parent distributions of flood processes) may be best represented by one member of the extreme value distribution family or another. The remaining problem, then, is the determination of *a*, *b*, and *c*, the three free parameters of the GEV distribution.

Several methods of estimating the best values of these parameters have been widely used, including, most frequently, the methods of plotting positions, moments, and maximum likelihood. The methods discussed here are limited to point-site estimates. If statistically similar data are available from other sites, then it may be possible to improve the parameter estimate through the method of *regional frequency analysis*; see Hosking and Wallis (1997) for information on this method. Note that this sense of the word *regional* is unrelated to the geographical sense (as in *regional studies*) discussed elsewhere in these guidelines, but instead relates to numerical regions of statistical parameters.

D.2.3.4.1 Plotting Positions

Widely used in older hydrologic applications, the method of plotting positions is based on first creating a visualization of the sample distribution and then performing a curve-fit between the chosen distribution and the sample. However, the sample consists only of the process variable; there are no associated quantiles, and so it is not clear how a plot of the sample distribution is to be constructed. The simplest approach is to rank-order the sample values from smallest to largest, and to assume that the value of *F* appropriate to a value is equal to its fractional position in this ranked list, *R/N*, where *R* is the value's rank from 1 to *N*. Then, the smallest observation is assigned *plotting position 1/N* and the largest is assigned *N/N=1*. This is clearly unsatisfactory at the upper end, because instances larger than the largest observed in the sample can occur. A more satisfactory and widely used plotting position expression is *R/(N+1)*, which leaves some room above the largest observation for the occurrence of still larger elevations. A number of such plotting position formulas are encountered in practice, most involving the addition of constants to the numerator and denominator, *(R+a)/(N+b)*, in an effort to produce improved estimates at the tails of specific distributions.

Given a plot produced in this way, one might simply draw a smooth curve through the points, and visually extend it to the recurrence intervals of interest. This constitutes an entirely empirical approach and is sometimes made easier by using a transformed scale for the cumulative frequency to construct the plot. The simplest such transformation is to plot the logarithm of the cumulative frequency, which flattens the curve and makes extrapolation easier.

A second approach would be to choose a distribution type and to adjust its free parameters so that a plot of the distribution matches the plot of the sample. This is commonly done by least-squares fitting. Fitting by eye is also possible if an appropriate *probability paper* is adopted, on which the transformed axis is not logarithmic but is transformed in such a way that the corresponding distribution plots as a straight line; however, this cannot be done for all distributions.

These simple methods based on plotting positions, although widely used, are problematic. Two fundamental difficulties with the methods are seldom addressed. First, it is inherent in the methods that each of N quantile bins of the distribution is occupied by one and only one sample point, an extremely unlikely outcome. Second, when a least-squares fit is made for an analytical distribution form, the error being minimized is taken as the difference between the sample value and the distribution value, whereas the true error is not in the value but in its frequency position.

It is noted here that the plotting position approach is an important component of the EST simulation method, to be discussed in a later subsection.

D.2.3.4.2 Method of Moments: Conventional Moments

An alternate method that does not rely upon visualization of the empirical distribution is the method of moments, of which there are several forms. This is an extremely simple method that generally performs well. The methodology is to equate the sample moments and the distribution moments, and to solve the resulting set of equations for the distribution parameters. That is, the sample moments are simple functions of the sample points, as defined earlier. Similarly, it may be possible to express the corresponding moments of an analytical distribution as functions of the several parameters of the distribution. If this can be done, then empirical estimates of those parameters can be obtained by equating the expressions to the sample values.

D.2.3.4.3 Method of Moments: Probability-weighted Moments and Linear Moments

Ramified versions of the method of moments overcome certain difficulties inherent in conventional methods of moments. For example, simple moments may not exist for a given distribution or may not exist for all values of the parameters. Higher sample moments may not be able to adopt the full range of possible values; for example, the sample kurtosis is constrained algebraically by the sample size.

Alternate moment-based approaches have been developed, including probability-weighted moments and the newer method of linear moments, or L-moments. L-moments consist of simple linear combinations of the sample values that convey much the same information as true moments: location, scale, shape, and so forth. However, being linear combinations rather than powers, they have certain desirable properties that make them preferable to normal moments. The theory of L-moments and their application to frequency analysis has been developed by Hosking; see, for example, Hosking and Wallis (1997).

D.2.3.4.4 Maximum Likelihood Method

A method based on an entirely different idea is the method of maximum likelihood. Consider an observation, x, obtained from the density distribution $f(x)$. The probability of obtaining a value close to x, say within the small range dx around x, is $f(x)\,dx$, which is proportional to $f(x)$. Then, the *posterior* probability of having obtained the entire sample of N points is assumed to be proportional to the product of the individual probabilities estimated in this way, in consequence of Equation D.2.3-5. This product is called the *likelihood* of the sample, given the assumed distribution:

$$L = \prod_{1}^{N} f(x_i) \tag{D.2.3-28}$$

It is more common to work with the logarithm of this equation, which is the *log-likelihood*, *LL*, given by:

$$LL = \sum_{1}^{N} \log f(x_i) \tag{D.2.3-29}$$

The simple idea of the maximum likelihood method is to determine the distribution parameters that maximize the likelihood of the given sample. Because the logarithm is a monotonic function, maximizing the likelihood is equivalent to maximizing the log-likelihood. Note that because $f(x)$ is always less than one, all terms of the sum for LL are negative; consequently, larger log-likelihoods are associated with smaller numerical values.

Because maximum likelihood estimates commonly show less bias than other methods and are also conceptually appealing, they are preferred. However, they usually require iterative calculations to locate the optimum parameters, and a maximum likelihood estimate may not exist for all distributions or for all values of the parameters for a particular distribution. If the Mapping Partner considers alternate distributions or fitting methods, the likelihood of each fit can still be computed using the equations given above, even if the fit was not determined using the maximum likelihood method. The distribution with the greatest likelihood of having produced the sample could then be chosen.

D.2.3.5 Extreme Value Analysis in a FEMA Flood Map Project

For FEMA Flood Map Project extreme value analysis, the Mapping Partner may adopt the annual maxima of the data series (runup, SWEL, and so forth) as the appropriate data sample, and then fit the GEV distribution to the data sample using the method of maximum likelihood. Also acceptable is the peak-over-threshold (POT) approach, fitting all observations that exceed an appropriately high threshold to the generalized Pareto distribution. The POT approach is generally more complex than the annual maxima approach, and only needs to be considered if the Mapping Partner believes that the annual series does not adequately characterize the process statistics. Further discussion of the POT approach can be found in references such as Coles (2001). The Mapping Partner can also consider distributions other than the GEV for use with the annual series. However, the final distribution selected to estimate the base flood level should be based on the total estimated likelihood of the sample. In the event that methods involve different numbers of points (e.g., POT vs. annual maxima), the comparison might be made on the basis of average likelihood per sample point.

As an example of this process, consider the extraction of a surge estimate from tide data. As discussed in Subsection D.2.4, the tide record includes the astronomic component and a number of other components, such as storm surge. For this example, all available hourly tide observations for a typical coastal tide gage were obtained from the NOAA tide data website. These observations covered the years from 1924 to the present. To work with full-year data sets, the period from 1924 to 2003 was chosen for analysis.

The corresponding hourly tide predictions were also obtained. The predictions represent only the astronomic component of the observations based on summation of the 37 local tidal constituents, so a departure of the observations from the predictions represents the anomaly or residual. A simple utility program was written to determine the difference between corresponding high waters (observed minus predicted) and to extract the maximum such difference found in each year. Levels at corresponding peaks were chosen because small-phase displacements between the predicted and observed data can cause spurious apparent amplitude differences. The Mapping Partner should inspect the observed and predicted values to determine the best means of defining the residuals; if the surge is large, for example, corresponding peaks may not be identifiable, and simple differences at fixed times may be preferred.

The resulting data array consisted of 80 annual maxima. Inspection of the file showed that the values were generally consistent except for the 1924 entry, which had a peak anomaly of over 12 feet, much larger than expected for the site. Inspection of the file of observed data showed that a large portion of the file was incorrect, with erroneous observations reported for long periods. Although the NOAA file structure includes flags intended to indicate data outside the expected range, these points were not flagged. Nevertheless they were clearly incorrect, and so were eliminated from consideration. The remaining abridged file for 1924 was judged to be too short to be reliable, and so the entire year was eliminated from further consideration.

Data inspection of this sort is critical for any such frequency analysis. Data are often corrupted in subtle ways, and missing values are common. Years with missing data may be acceptable if the fraction of missing data is not excessive, say not greater than one quarter of the record, and if there is no reason to believe that the missing data are missing precisely because of the occurrence of an extreme event, which is not an uncommon situation. Gages may fail during extreme conditions and the remaining data may not be representative and so should be discarded, truncating the total period.

The remaining 79 data points in the sample for this example were used to fit the parameters of a GEV distribution using the maximum likelihood method. The results of the fit are shown in Figure D.2.3-1 for the cumulative and the density distributions. Also shown are the empirical sample CDF, displayed according to a plotting position formula, and the sample density histogram. Neither of these empirical curves was used in the analysis; they are shown only to provide a qualitative idea of the goodness-of-fit.

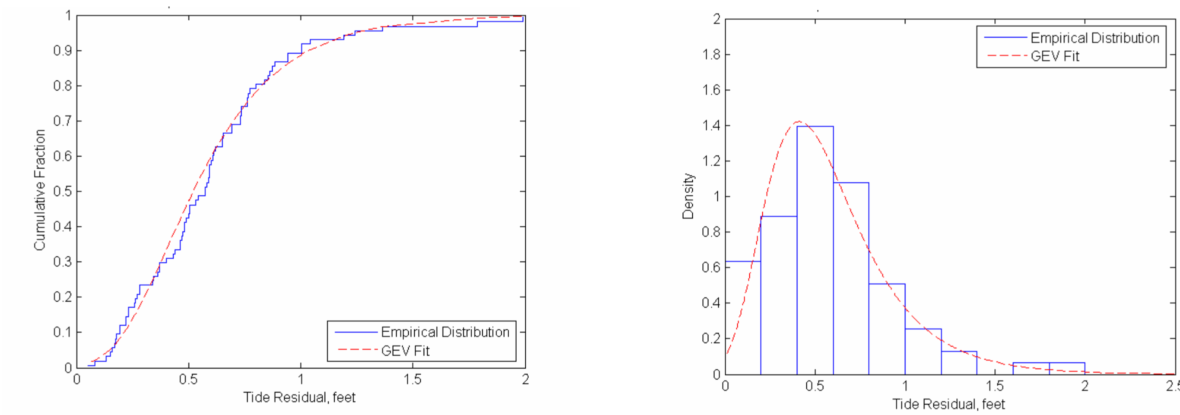

Figure D.2.3-1. Examples of Cumulative and Density Distributions for the Tide Residual

The GEV estimate of the 1-percent annual chance residual for this example was 1.74 feet, with a log-likelihood of -19.7. The estimate includes the contributions from all nonastronomic processes, including wind and barometric surge, and from wave setup to the degree that it might be incorporated in the record at the gage location.

D.2.3.6 Simulation Methods

In some cases, flood levels must be determined by numerical modeling of the physical processes, by simulating a number of storms over a long period of record and then deriving flood statistics from that simulation. FEMA Flood Map Project flood statistics have been derived through four types of simulation methods. Three of these methods involve storm parameterization and random selection: the JPM, the EST, and the Monte Carlo method. These methods are described briefly below. In addition, a direct simulation method may be used in some cases. This method requires the availability of a long, continuous record describing the forcing functions needed by the model (such as windspeed and direction in the case of surge simulation using the one-dimensional [1-D] BATHYS model described elsewhere). The model is used to simulate the entire record, and flood statistics are derived in the manner described previously.

D.2.3.6.1 Joint Probability Method (JPM)

JPM has been applied to flood studies in two distinct forms. First, joint probability has been used in the context of an event-selection approach to flood analysis. In this form, JPM refers to the joint probability of the parameters that define a particular event, such as wave height and water level. In this approach, one seeks to select a small number of such events thought to produce flooding approximating the base flood level. This method usually requires a great deal of engineering judgment and should only be used with the permission of the FEMA Study Representative.

FEMA has adopted a second sort of JPM approach for hurricane surge modeling on the Atlantic and Gulf coasts, which is generally acceptable for any site or process for which the forcing function can be parameterized by a small number of variables (such as storm size, intensity, and kinematics). If this can be done, one estimates cumulative probability distribution functions for each of the several parameters using storm data obtained from a sample region surrounding the study site. Each of these distributions is approximated by a small number of discrete values, and all combinations of these discrete parameter values, representing all possible storms, are simulated with the chosen model. The rate of occurrence of each storm simulated in this way is calculated from the total rate of storm occurrence at the site, estimated from the record, multiplied by each of the discrete parameter probabilities. If the parameters are not independent, then a suitable computational adjustment must be made to account for this dependence. For example, if hurricane central pressure and radius to maximum wind are thought to be correlated, then one might adopt distributions of radius that are contingent upon pressure. There is no limitation of the JPM method requiring the assumption of independence between parameters, despite some perception to the contrary.

The peak flood elevations for each storm are saved for subsequent determination of the flood statistics. This is done by establishing a histogram for each point at which data have been saved, using a small bin size of about 0.1 foot. The rate contribution of each storm, determined as described above, is summed into the appropriate elevation bin at each site. When this is done for all storms, the result is that the histograms approximate the density function of flood elevation at that site. The cumulative distribution is obtained by summing across the histogram from top down; the BFE is found at the point where this sum equals 0.01. Full details of this procedure are provided in the user's manual accompanying the FEMA storm-surge model (FEMA, 1988), and in other JPM studies performed by agencies such as NOAA.

D.2.3.6.2 Empirical Simulation Technique (EST)

The U.S. Army Corps of Engineers has developed a newer technique, EST, that FEMA has approved for the FIS; a full discussion can be found in Scheffner et al. (1999). At the heart of the technique is the empirical estimation of the cumulative distribution using nonparametric plotting position methods. Alternate life-cycle simulations are based on bootstrap resampling-with-replacement from a historical data set, perhaps supplemented by a random walk variation. The random sampling of the finite-length historical-event database generates a larger long-period database, which is especially useful in assessing the importance of variability. The only assumption is that future events will be statistically similar in magnitude and frequency to those particular storms that constitute the database.

The EST begins with an analysis of historical storms that have affected the study area. The selected events are then parameterized to define relevant input parameters that are used to define the dynamics of the storms (the components of the so-called input vectors) and factors that may contribute to the total response of the storm, such as tidal amplitude and phase. Associated with the storms are the response vectors that define the storm-generated effects. Input vectors are sets of selected parameters that define the total storm; response vectors are sets of values that summarize the effects. Basic response vectors are determined from observational data, or numerically by simulating the historical storms using the selected hydrodynamic model.

These sets of input and response vectors are used subsequently as the basis for the long-term surge history estimations. These are made by repeatedly sampling the space spanned by the input vectors in a random fashion, and estimating the corresponding response vectors. The number of storms occurring in a particular year is assumed to be governed by a Poisson distribution. Moderate variation from the historical record is permitted by the use of random displacements in the input vector space. The final step of the procedure is to extract statistics from the simulated long records by performing an extremal analysis as though the simulated records were physical records. This has commonly been done using plotting position methods. More recent work has focused attention on the need to extend the upper tails in an appropriate way in order to reach the extreme levels of interest in a coastal study, in cases for which the historical database is limited.

D.2.3.6.3 Monte Carlo Method

As discussed above for the JPM approach, the Monte Carlo method is based on probability distributions established for the parameters needed to characterize a storm. Unlike JPM, however, these probability distributions are not discretized. Instead, storms are constructed by randomly choosing values for each parameter by generating random values uniformly distributed between 0 and 1, and then entering the cumulative distributions at those values and selecting the corresponding parameter values. Each storm selected by this Monte Carlo procedure is simulated with the hydrodynamic model, and shoreline elevations are recorded. Simulating a large number of storms in this way is equivalent to simulating a long period of history, with the frequency connection established through the rate of storm occurrence estimated from a local storm sample. The Monte Carlo method has been used extensively in concert with a 1-D surge model by the Florida Department of Environmental Protection to determine coastal flood levels; see the 1-D surge discussion in Subsection D.2.4 for additional information.

D.2.3.6.4 Period of Record and Data Sample Area

Important issues to be addressed in any simulation study are the period of record from which governing data is taken, and the geographic sample area. Unfortunately, data sets are generally limited, and one must compromise between assembling a large sample in order to minimize sample error, and a small local sample to minimize population error.

No hard rules can be presented here. Instead, the Mapping Partner should evaluate the sources and limitations of the data available for the process at hand, and assess likely limits and uncertainties. The available record may be enhanced by the construction of hypothetical storms similar to historical storms (possibly important in EST work) or by the inclusion of storms that passed outside the area but appear to be homogeneous with the history at the site (again, an EST issue to augment a small local sample).

The derivation of storm parameters used in a JPM study, for example, should be based on very high quality data., The period since the World War II includes the best data for hurricane parameters, although earlier data (back to about 1900) may also be acceptable, depending upon its source. The quality of data will also depend upon the particular parameter of interest. For example, data from the HURDAT tropical storm database may be acceptable for storm occurrence and tracks as far back as the mid-to-late 19th century, but they may not be deemed acceptable for storm intensity and wind information during the same era. Furthermore, data should generally be direct and not inferred secondarily from other sources.

D.2.3.7 Additional Resources

The foregoing discussion has been necessarily brief; however, the Mapping Partner may consult the extensive literature on probability, statistics, and statistical hydrology. Most elementary hydrology textbooks provide a good introduction. For additional guidance, the following works might be consulted:

Probability Theory:

An Introduction to Probability Theory and Its Applications, Third Edition, William Feller, 1968 (two volumes). This is a classic reference for probability theory, with a large number of examples drawn from science and engineering.

The Art of Probability for Scientists and Engineers, Richard Hamming, 1991. Less comprehensive than Feller, but it provides clear insight into the conceptual basis of probability theory.

Statistical Distributions:

Statistics of Extremes, E.J. Gumbel, 1958. A cornerstone reference for the theory of extreme value distributions.

Extreme Value Distributions, Theory and Applications, Samuel Kotz and Saralees Nadarajah, 2000. A more modern and exhaustive exposition.

Statistical Distributions, Second Edition, Merran Evans, Nicholas Hastings, and Brian Peacock, 1993. A useful compendium of distributions, but lacking discussion of applications; a formulary.

An Introduction to Statistical Modeling of Extreme Values, Stuart Coles, 2001. A practical exposition of the art of modeling extremes, including numerous examples. Provides a good discussion of POT methods that can be consulted to supplement the annual maxima method.

Statistical Hydrology:

Applied Hydrology, Ven Te Chow, David Maidment, and Larry Hays, 1988. One of several standard texts with excellent chapters on hydrologic statistics and frequency analysis.

Probability and Statistics in Hydrology, Vujica Yevjevich, 1972. A specialized text with much pertinent information for hydrologic applications.

General:

Numerical Recipes, Second Edition, William Press, Saul Teukolsky, William Vetterling, and Brian Flannery, 1992. A valuable and wide ranging survey of numerical methods and the ideas behind them. Excellent discussions of random numbers, the statistical description of data, and modeling of data, among much else. Includes well-crafted program subroutines; the book is available in separate editions presenting routines in FORTRAN and C/C++.

D.2.4 Water Levels

This subsection provides guidance for the determination of water levels, including tide and wind setup. New guidance on special considerations in sheltered waters is provided. This subsection also includes guidance on 1-percent stillwater levels (SWELs), combined effects of surge and riverine runoff, and consideration of nonstationary processes.

D.2.4.1 Overview and Definitions

The two fundamental components of the BFE are *water levels*, discussed in this subsection, and waves, discussed in a subsequent subsection. The stillwater level (SWL), also known as the stillwater elevation (SWEL), is the base elevation upon which the waves ride. It consists of several parts including mean sea level (MSL), the astronomic tide that fluctuates around MSL, and storm surge. All storm wave contributions are excluded; static and dynamic wave setup (Subsection D.2.6) is included in the *mean water level* (MWL), which is somewhat higher than the SWEL (which does not include wave setup).

The Mapping Partner performing the flood analysis shall adopt previously documented SWEL analyses (by others) or determine the SWELs in a rational, defensible manner, and shall not include contributions from wave action either as a result of the assumptions of the predictive model or of the data used to calibrate the model. Only the 1-percent-annual-chance SWEL is currently required for determination of coastal BFEs, although 10-, 2-, and 0.2-percent-annual-chance elevations are tabulated in the FIS report, and the 0.2-percent-annual-chance floodplain boundary is mapped on the FIRM as the limit of the shaded X Zone.

SWELs may be defined by statistical analysis of available tide gage records or by calculation using a storm surge computer model. A minimum of 30 years of recorded tide data is needed if the SWEL is to be based on tide gage records alone. Measured tide levels are preferred over models, provided they have an adequate period of continuous record and can accurately represent the geographic area of the study. FEMA previously prescribed the use of its hurricane storm-surge model (FEMA, August 1988) and a northeaster model that simulates the wind and pressure fields of an extratropical storm (Stone & Webster Engineering Corporation, 1978). The FEMA storm-surge model as well as other FEMA-accepted hydrodynamic models that meet the NFIP regulatory requirements can be used for storm surge studies, including the Advanced Circulation Model (ADCIRC) and the DHI MIKE-21 model. For the northeast Atlantic coasts from Long Island Sound to the Maine border with Canada, FEMA Regional offices have adopted the USACE New England District tide profile analysis of the 1-percent-annual-chance elevations (based on long-term tide gage data throughout the region), which has superseded use of the Stone & Webster northeaster model.

The Mapping Partner shall use these or other approved computer models for complex shorelines where gage records are limited, nonexistent, non-representative, or which otherwise indicate appreciable variations in flood elevations from point to point within a community. FEMA also has specified procedures required for intermediate reviews and documentation of coastal flood studies using a storm-surge model, as discussed separately in section D.2.9.

The following subsections discuss each of the stillwater components in turn, including an outline of methods to determine water-level statistics. Also included is a discussion of nonstationarity in the processes that control relative water levels.

D.2.4.2 Astronomic Tide

The astronomic tide is the regular rise and fall of the ocean surface in response to the gravitational influence of the moon, the sun, and the Earth. Because the astronomic processes are entirely regular, the tides, too, behave in an entirely regular, though complex, manner. A useful overview of tidal physics is presented in a small booklet published by the National Ocean Service (NOS), *Our Restless Tides*, now out of print, but available in electronic form from the NOAA website (<http://tidesandcurrents.noaa.gov/pub.html>) where many other documents of related interest can be found.

D.2.4.2.1 Tides and Tidal Datums

The tides along the Atlantic are semidaily or semidiurnal, meaning that there are two highs and two lows each day, while in the Gulf of Mexico the tides are mix of diurnal, meaning that there is only one high and low each day, and semidiurnal. The average of all the highs is denoted as mean high water, MHW, while the average of all the lows is mean low water (MLW). Averages are taken over the entire tidal datum *epoch*, which is a particular 19-year period explicitly specified for the definition of the datums; a full astronomic tidal cycle covers a period of 18.6 years. The average of all hourly tides over the epoch is the MSL.

The daily highs are generally unequal, as are the lows, and are identified as Higher High, Lower High, and so forth. At a given coastal location, each of these has a mean value identified as mean higher high water (MHHW), mean lower high water (MLHW), mean higher low water (MHLW), and mean lower low water (MLLW). In addition to these, one speaks of the mean tide level, MTL, which is the average of MHW and MLW, and which is also called the half-tide level.

These several levels are important because they constitute the datums to which tide data have traditionally been referred. Local charts and recorded tide gage data are generally referenced to local MLLW or MLW. This introduces some ambiguity because MLLW and MLW vary from place to place and from epoch to epoch. For use in FEMA Flood Map Projects, then, these tidal datums are insufficient in themselves, and must be related to a standard vertical datum such as the North American Vertical Datum of 1988 (NAVD88) or the National Geodetic Vertical Datum of 1929 (NGVD29); it is not always straightforward to make this connection. However, NOAA maintains tidal benchmarks for many stations that are now tied to a standard vertical datum. Benchmark sheets for active and historic stations are available at the NOAA Website, <http://tidesandcurrents.noaa.gov/station_retrieve.shtml>. The following example is extracted directly from the Galveston, Texas benchmark sheet for Galveston Pleasure Pier:

Tidal datums at GALVESTON PLEASURE PIER, GULF OF MEXICO based on:

LENGTH OF SERIES: 5 Years
TIME PERIOD: January 1997 - December 2001
TIDAL EPOCH: 1983-2001
CONTROL TIDE STATION:

Elevations of tidal datums referred to Mean Lower Low Water (MLLW), in METERS:

HIGHEST OBSERVED WATER LEVEL (09/11/1961)	= 2.805
MEAN HIGHER HIGH WATER (MHHW)	= 0.622
MEAN HIGH WATER (MHW)	= 0.563
MEAN TIDE LEVEL (MTL)	= 0.341
MEAN SEA LEVEL (MSL)	= 0.338
NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)	= 0.186
MEAN LOW WATER (MLW)	= 0.119
MEAN LOWER LOW WATER (MLLW)	= 0.000
LOWEST OBSERVED WATER LEVEL (02/12/1985)	= -1.487

National Geodetic Vertical Datum (NGVD 29)

Bench Mark Elevation Information In METERS above:

Stamping or Designation	MLLW	MHW
NO 43 1957	7.539	6.976
WALL 1933 ELEV 15.279 FT	4.536	3.973
E 168 1936 ELEV 15.502 FT	4.586	4.023
WALL NO 1 1942	4.508	3.945
NO 44 1957	4.565	4.002
NO 10 1973	4.555	3.992
NO 45 1975	4.573	4.010
NO 46 1975	4.560	3.997
NO 47 1975	4.525	3.962
S 449 5	4.574	4.011

In this example, NAVD88 is shown to be at 0.186 meters above MLLW for the specified 1983-2001 epoch, fixing the tidal datums. Not all NOAA benchmark sheets include NAVD88 (or NGVD29) as this example does, but most include surveyor's benchmark information as shown above, through which the tidal datums can usually be tied to a standard vertical datum as needed in FEMA Flood Map Projects; these benchmark sheets include full descriptions of the benchmarks and exact locations. If other tide gages without surveyed bench marks are used as part of the FIS, these need to be surveyed and tied into other established bench marks to determine the NGVD29 or NAVD88 elevation reference levels.

D.2.4.2.2 Tide Observations

The tide is recorded at a large number of gages maintained by NOAA, with records dating back over 100 years in many cases. Much of this data is available at NOAA's website for the National Water Level Observation Network, <http://tidesandcurrents.noaa.gov/nwlon.html>, as either six-minute or hourly time series over the particular site's entire period of record. Additional data may be available from other sources.

The tide observations include the total water level at the gage, suitably filtered to suppress high frequency wave components, leaving the long period components associated not only with astronomic tide, but also with sea-level variations caused by atmospheric pressure fluctuations, wind setup (storm surge), riverine rainfall runoff into a relatively confined region, low frequency tsunami elevation, and wave setup to the degree that it occurs at the gage site. In general, little wave setup is reflected in tide gage data because gages are often located in protected areas not subject to much setup, or in open areas outside the surf zone, and so seaward of the largest setup values (see subsection D.2.5.1 for discussion of the physics of wave setup).

The fact that the tide gage record includes all of these non-astronomic low frequency components makes it possible to extract stillwater statistics from gage data, subject to the setup limitation noted. A general method to do this is discussed below.

D.2.4.2.3 Tide Predictions

The Mapping Partner should obtain NOAA's tide prediction computer program, NTP4, or an equivalent, and generate tide predictions as needed. The advantages include not only convenience, but more importantly, the ability to use other constituent values than those currently adopted. This is important because the local tide depends not only on the astronomic forcing, but also on the response of the local basins. The response can, and does, change with time owing to deposition and dredging, construction of coastal structures such as breakwaters, changes in inlet geometry, and so forth. Consequently, the astronomic tide observed at a fixed location may not be stationary, but may have changed over the period of record. NOAA can provide prior estimates of the tidal constituents for a site, and these should be used with the NOAA computer program to produce more realistic estimates of historical tides than would be achieved using the current data for a prior period. In order to verify tide prediction validity, a set of observed tide data for a period without any significant influence from local or regional weather conditions (periods of prolonged droughts are preferred) is required. Once determined, the data can be used to compare measured/observed tide levels to the predicted tidal fluctuations. Any differences in predicted versus observed can then be accounted for in model adjustments if necessary.

NOAA's tide prediction program, NTP4, is not available online, but can be purchased from NOAA at nominal cost, including both source code, an executable file (a DOS console program), and two manuals that thoroughly document the theory and practice of tide prediction: U.S. Department of Commerce Special Publication 98, *Manual of Harmonic Analysis and Prediction of Tides* (1940, 1958), and a 1982 supplement updating certain numerical factors to 21st century values.

D.2.4.2.4 Tidal Constituents

The astronomic component of the observed tide gage record is considered to be well-known in principle, consisting of the summation of 37 tidal constituents that are simply sinusoidal components with established periods, and with site-dependent amplitudes and phases. These constituents are available for most gage locations from the NOAA site, <http://tidesandcurrents.noaa.gov/products.html>.

The NOAA website also provides tide predictions for any date in the past or future, limited however to download of one year of predictions at a time. Note that these predictions are computed using the currently adopted values of the 37 tidal constituents for the site, not prior values.

D.2.4.2.5 Tide Gage Analysis (Surge Anomaly) and Extraction of Non-astronomic Stillwater from Gage Records

As discussed above, both observed data and a method to predict the purely astronomic component of those observations are available. By subtracting the predictions from the observations, one arrives at a time series of the non-astronomic contribution to the measured stillwater (the tide residual or tide anomaly), including surge and meteorological effects, rainfall runoff, and tsunamis – in fact, all non-astronomic components termed stillwater. As a practical matter, the static setup will not usually be present in the record to a significant degree, for reasons already mentioned. Figure D.2.4-1 shows observed, predicted, and residual tides (observed minus predicted) at Panama City, Florida for a five-day period in August 2005 during Hurricane Katrina's approach and landfall to the west in Louisiana and Mississippi. As shown, a slowly varying residual component approximately 2.5 feet in amplitude is superimposed on the fluctuating astronomic tide.

The recommended procedure to extract the residual stillwater as the difference between the observed and predicted data is extremely simple in concept, assuming that the period of record is adequate (30 years or more) and that the older predictions were made using the appropriate set of tidal constituents, not necessarily those in current use. It would also be assumed that the observed and predicted data has been adjusted to the same vertical datum prior to extraction. One first determines the differences between the observed and predicted elevations (either for all points or only for the highs and lows, as appropriate), and then scans these to locate the annual peaks. These annual peaks are used to fit an extreme value distribution from which the 1-percent-annual-chance elevation can be found.

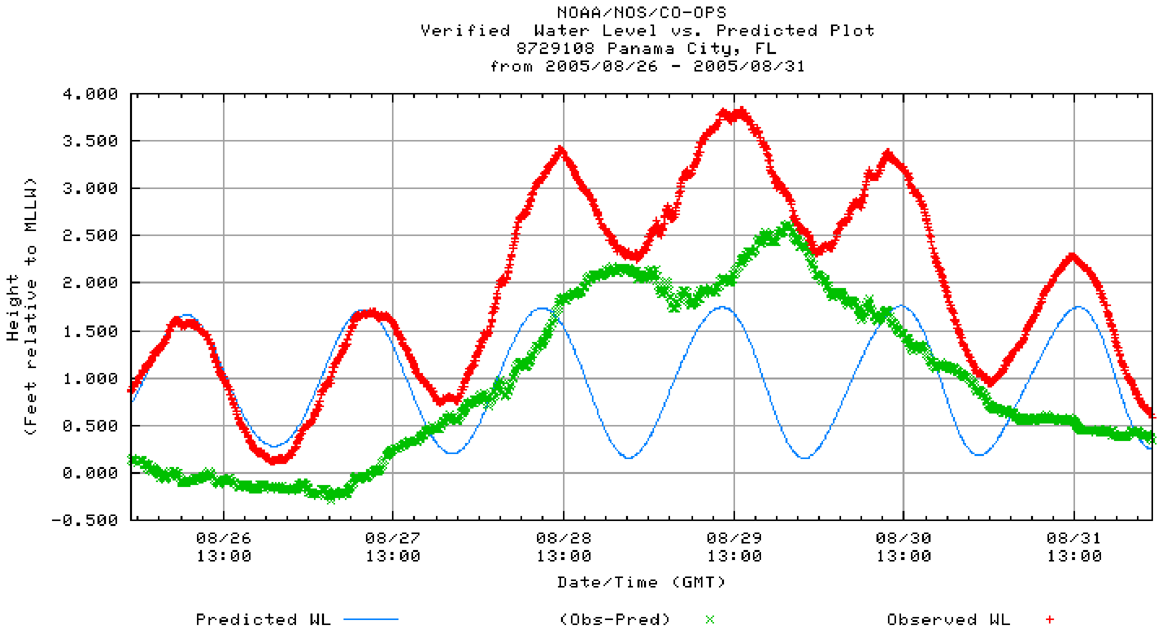

Figure D.2.4-1 Predicted, Observed, and Residual Tides at Panama City, Florida

As discussed in Subsection D.2.3.3, an acceptable approach for the extreme value analysis is to adopt the GEV Distribution, and to determine the distribution parameters by the method of maximum likelihood. The Mapping Partner may consider other distributions and other fitting techniques as may be necessary to adequately fit the observations, although the particular result with the greatest likelihood value among all of the considered distribution types should be adopted, unless otherwise approved by the FEMA Study Representative.

This recommended procedure is based upon the annual maxima of the residual rather than the annual maxima of the raw data because the underlying astronomic tide is not a random variable, but is deterministic and is limited to a known maximum (less than or equal to the sum of the 37 tidal constituent amplitudes). For these reasons, it may not be appropriate to extrapolate the bounded and deterministic portion of the record out to the upper tail of an unbounded distribution. Subsequent determination of the combined effects of the separated tide and the residual stillwater can be made as discussed in the following Storm Surge section.

Finally, it is emphasized that although this procedure is straightforward in concept, it can be complicated in practice. One complicating factor – changes in the tidal constituents over time – has already been mentioned. Another is the fact that tidal predictions are made with respect to tidal datums, and these may have changed over time, even when referenced to a fixed standard such as NAVD88. Changes in the constituents are one source of datum shift, while changes in relative sea level (including sea-level rise and land subsidence) are another. The Mapping Partner should carefully review the history of the tide gage, the history of the tidal datums, the history of the published constituents, and the local history of relative sea level to ensure that at each step, the residual is properly defined.

D.2.4.3 Storm Surge

D.2.4.3.1 General Considerations

Storm surge is the rise of the ocean surface that occurs in response to barometric pressure variations (the inverse barometer effect) and to the stress of the wind acting over the water surface (the wind setup component). Wave setup is excluded by this definition. Setup is not incorporated in the common procedures for storm-surge modeling, nor is it present to a significant degree in tide gage data owing to the typical configuration of gages with respect to the zone of large setup; consequently, it must be taken into account separately as discussed in Subsection D.2.5.1.

Storm simulation models must be capable of adequately prescribing and implementing wind, pressure, and tidal conditions into the physics represented by the model if the model-generated spatial and temporal distribution of surge and circulation are to be physically realistic. Models of differing complexity are in wide use, including both 1-D and 2-D models. The Mapping Partner should consult FEMA's list of accepted models to select an appropriate model for a given study. Should a model that is not on the list appear advantageous, the Mapping Partner shall discuss the possibility of its use with the FEMA Study Representative.

Some of the factors that must be considered in selection and application of a model are enumerated below. Specific guidance regarding each factor is not given here. Instead, guidance for complex 2-D modeling is best obtained from the user's manual for a particular model, and from review of prior studies which have successfully used that model. A good general overview of surge modeling for flood insurance studies can also be found in the 3-volume documentation of the FEMA Surge Model (1988), although specifics of application will differ for other models.

Modeling factors that shall be considered in any full storm surge study include:

- The *governing equations* of the model, typically the nonlinear long wave equations accounting for conservation of mass and momentum, with surface wind and barometric pressure terms representing the influence of the storm

- The *numerical scheme* used by the model, whether finite differences computed on a grid of rectangular cells (commonly of fixed size) or in curvilinear coordinates, or finite elements represented by triangular or quadrilateral cells (of varying sizes). The numerical scheme may also be explicit or implicit, affecting time step constraints, and so affecting study cost

- The *flooding / drying* treatment of cells as the surge and tides advance onto land and then recedes

- The *storm representation*, such as a planetary boundary layer model for a hurricane, or a simpler empirical/parametric description, including both wind and pressure; the storm representation will be quite different for hurricanes and northeasters although the modeling principles remain the same in each case; on-land filling will be significant for sheltered waters; winds and pressure representations must be appropriate 10 meter elevation, averaged winds

- The *wind stress coefficient* which relates the windspeed at the surface to the stress felt by the fluid; consideration must be given to the possibility that the wind stress is capped under the most extreme conditions

- The *sheltering* treatment, adjusting the effective wind stress to account for partial reduction by tall vegetation, terrain, and structures (especially significant for sheltered waters)

- The offshore *bottom friction* treatment over the relatively smooth ocean or bay bottom, which retards the flow

- The onshore *flow resistance* treatment accounting for bottom friction and resistance offered by tall vegetation and structures; critical for sheltered waters

- The source and quality of *bathymetric* data, defining the varying depths at the site

- The source and quality of *topographic* data, such as traditional quad sheets or newer LIDAR data

- The manner in which *normal storm erosion* alters the topography used in the model

- The manner in which *catastrophic erosion* might affect the modeling assumptions, in the event of loss of a major barrier to inland flooding

- The representation of the bathymetry and topography in the model *grid* system, which depends upon the numerical scheme

- The *faithfulness* of the grid to the irregular bathymetry and terrain, including conformance to boundary shapes and inclusion of small sub-grid barriers which may control the local variation of overland flow

- The *resolution* of the grid, whether fixed or varying through the study area

- The *boundary conditions* which impose approximate rules along the edges of the model area, both offshore and onshore, permitting termination of the calculations at the expense of accuracy

- The treatment of *astronomic tide* which might be handled as part of the simulation through the boundary conditions and tidal potentials, or which might be treated as an added effect separate from the surge simulations

- The types and limits of *calibration* which might be done, including small amplitude astronomic tide reproduction for which calibration data is reliable

- The role of *verification hindcasts* to confirm the apparent reasonableness of the final model when compared with historical surge records
- The role of *wave setup* (a separate topic in these guidelines), especially in the interpretation of high-water marks used for hindcast verification

These factors have been listed here to alert the Mapping Partner to the numerous and complex issues which must be addressed during the course of a full storm surge study. For each, the Mapping Partner must review model documentation and user's manuals, as well as recent studies accepted by FEMA using the selected model, to discern the appropriate level of effort for a new study.

D.2.4.3.2 Simplified One-Dimensional Surge Modeling

While specific guidance for large-scale 2-D surge modeling is beyond the scope of these guidelines, a simplified one dimensional tool has been specially developed for restricted use in flood insurance studies.

There are several reasons a Mapping Partner might wish to make simplified estimates where detailed 2-D modeling is either not needed or is inappropriate: the Mapping Partner may wish to determine SWEL in regions of sheltered waters where an absence of tide gage data makes it impractical to extract stillwater data from the tide residual; the Mapping Partner might wish to compare the surge level from a wind of a certain magnitude with the 1-percent-annual-chance wave event; the 1-percent-annual-chance wave event might be accompanied by strong onshore winds and the Mapping Partner might wish to include this contribution or to evaluate the significance of neglecting it; or the Mapping Partner may wish to explore the sensitivity of locally generated surge levels to windspeed or direction, or to variations in bathymetry and topography.

For such approximate and/or diagnostic purposes, a computer program (BATHYS) has been developed based on the so-called Bathystrophic Storm Tide (BST) theory formulated originally by Freeman, Baer, and Jung (1954). The BST theory accounts for the onshore component of wind stress and the Coriolis force associated with the Earth's rotation. The assumptions of the model are that the onshore forces are in static balance; however, the longshore component includes inertia and requires some time to achieve a balance. A user's manual describing the program and its use in much greater detail is available separately from FEMA.

D.2.4.3.2.1 The System of Interest and Governing Equations

The system of interest is shown in Figure D.2.4-2. A wind with speed W is directed at an angle, θ, to the x-axis that is parallel to the shoreline. The surge distribution is $\eta(y)$, where y is the cross-shore direction. The wind obliquity induces a mean current, $U(y,t)$, which varies with time, t.

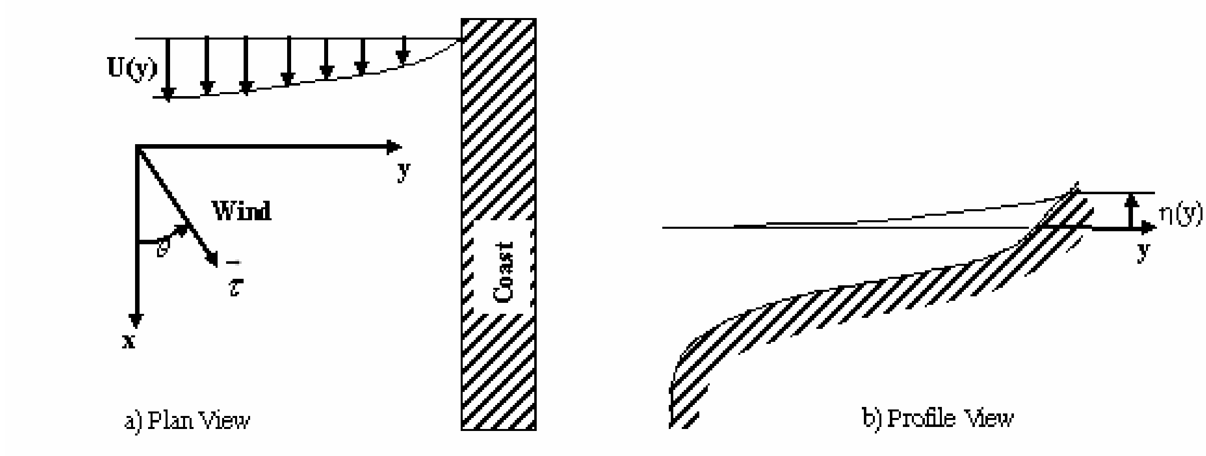

Figure D.2.4-2 Definition Sketch for the BST Formulation

The governing equations are:

y Direction

$$\frac{\partial \eta}{\partial y} = \frac{1}{g}\left(\frac{n\tau_y}{\rho(h+\eta)} - f_c U\right) \tag{D.2.4-1}$$

x Direction

$$\frac{\partial U}{\partial t} = \frac{1}{h+\eta}\left(\frac{\tau_x}{\rho} - \frac{fU^2}{8}\right) \tag{D.2.4-2}$$

In these equations, n ($\approx$1.05 to 1.1) is a factor that augments the onshore component of the wind stress, τ_y, to account for the bottom frictional effect because of return flow; τ_x is the longshore component of wind stress; ρ is the mass density of water ($\approx$1.99 slugs/ft^3); and f_c is the Coriolis coefficient (= 2Ω sinφ) where Ω and φ are the rotational speed of the Earth in radians per second and latitude, respectively. The quantity f is the Darcy-Weisbach friction factor ($\approx$0.08 to 0.16).

The longshore and onshore components of the wind stress are specified in terms of a wind stress coefficient, k, and the wind direction, θ, relative to a shore normal

$$\begin{Bmatrix} \tau_x \\ \tau_y \end{Bmatrix} = \begin{Bmatrix} \cos\theta \\ \sin\theta \end{Bmatrix} k|W|W \tag{D.2.4-3}$$

where the wind stress coefficient, k, is that developed by Van Dorn (1953):

$$k = \begin{cases} 1.2x10^{-6}, & |W| \le W_c \\ 1.2x10^{-6} + 2.25x10^{-6}\left(1 - \dfrac{W_c}{|W|}\right)^2 & |W| > W_c \end{cases} \tag{D.2.4-4}$$

D.2.4.3.2.2 BATHYS Program Input and Output

The input quantities to the program are the bathymetry along the shore normal transect, $h(y)$, and the windspeed and direction , $W(t)$ and $\theta(t)$, which can be specified so as to vary linearly with time between specified pairs of windspeeds and directions at selected times. The output of the program is the wind surge at the shore, η_s, as a function of time. To incorporate the effects of astronomic tide, the program permits specification of a time-dependent condition at the seaward boundary of the transect.

Because the longshore current varies as a function of time, the surge, η_s, also varies with time. This reflects the contribution of the Coriolis force; for fixed wind conditions, the surge approaches a constant value as the longshore current approaches its constant equilibrium value for a given windspeed and direction.

The program is extremely efficient and easy to use, with minimal input requirements. The necessary bathymetric data can be obtained from available charts, and wind data can usually be extracted from common sources or from parametric storm descriptions. Users need to be certain that model output is for 1-percent-annual-chance surge elevations, and make any necessary data or input changes to obtain the desired results.

A second simplified tool, the DIM program discussed in Subsection D.2.5, is also available. It was developed especially for the computation of setup over a shore-normal transect similar to that used here by BATHYS. DIM requires additional input, however, because its primary purpose is wave setup simulation. The user's manuals for these programs should be consulted for additional details and examples of use.

D.2.4.3.3 Surge Estimation from Tide Data

A procedure was outlined in Subsection D.2.4.2.5 to extract the total stillwater, exclusive of astronomic tide, from a tide gage record. It is in general difficult or impossible to distinguish among the several components of the residual, which include surge, and there is usually no need to do so. Consequently, the tide residual methodology can be considered equivalent to the estimation of surge from tide data, for all practical purposes. What one generally wants is the 1-percent-annual-chance level of the total flood, irrespective of mechanism.

D.2.4.3.4 Aspects of 2-D Surge Modeling

As noted before, a detailed exposition of 2-D surge modeling is beyond the scope and intent of these guidelines; model documentation and user's manuals as well as detailed documentation of prior studies must be consulted by the Mapping Partner. However, some general guidance is offered in this subsection. The goal of this guidance summary is to identify and discuss important model features and capabilities that should be part of any numerical model that is to be used to simulate tropical or extratropical storms, and the general procedures to be followed.

Grid Considerations:

A primary consideration in numerical modeling is that the modeled domain is adequately represented by the computational grid. This includes not only the shoreline and island land boundaries in the area of interest, but also the offshore boundary. For example, the grid should extend far enough offshore of the project area to allow the storm to generate a fully developed surge as a function of tides and wind/pressure. If the grid is too small, the surge will not develop completely, resulting in an under prediction of the surge. Additionally, if the model boundaries are too close to the project area and located in shallow water, i.e., on the continental shelf, nonlinearities and numerical instabilities may develop, or the boundary computations may be otherwise inaccurate. In general, outer boundary conditions should be prescribed as far from the study area and in as deep water as possible.

A second grid consideration concerns irregular boundaries along open coasts and within estuaries or embayments. Although curvilinear coordinate structured grids can be made to adequately represent irregular boundaries, it may be difficult and time consuming to develop an acceptable structured grid, especially if currents as well as surface elevations are a concern. There is also a computational burden associated with structured grids because the high resolution in the area of interest must be extended to the grid boundary. Therefore, unstructured grids are often preferable to structured grids in large domain modeling applications. Unstructured grids are usually easier to generate and provide the necessary flexibility to define offshore boundaries that are well removed from the project area. Simple structured grids are acceptable, however, and may consist of a succession of nested grids of increasing resolution, rather than a single grid.

A third consideration in the development of any computational grid is the compatibility with the maps used to generate the grid. Many grids are referenced to Latitude and Longitude; however, some are referenced to X- and Y-coordinate systems that may in turn be referenced to one or more state-plane coordinate systems. If the grid overlaps multiple systems, the modeler needs to ensure that all data is compatible. Compatibility includes not only the numerical values of common but translated or rotated nodes at a specific topographic/bathymetric feature, but also the projection used to determine horizontal distances. If different projection methods are used in adjacent regions, the derived model grid may be correspondingly skewed.

Finally, the selected grid must be populated with the most up to date and accurate bathymetric and topographic data available. Even the best numerical model will give erroneous results if depths and land elevations are in error. It is very important to ensure that all data are referenced to the same vertical datum, so that the geometry of the basin is faithfully reflected in the model. Depths are often referred to Mean Lower Low Water (MLLW) on navigational charts, not to some mean level; in many locations, the difference between MSL and MLLW is significant, perhaps as much as a few meters. Topography will generally be referenced to either NAVD88 or NGVD29. These potential differences must be resolved to ensure consistency among bathymetry, topography, ocean surface elevation, and tidal forcing.

Boundary forcing:

Tropical storms require both pressure and windfields in order to simulate the storm surge; northeasters and extratropical events may only require wind. However, both applications may require the storm to be simulated with the influence of tides. In this case, tidal boundary conditions on the open-coast boundaries as well as tidal potential terms over the entire computational grid must be specified. The model selected for the storm surge computations may have these capabilities as part of the model package. If a model does not contain these features, the user will have to provide the necessary boundary conditions. Global tidal boundary conditions and extratropical windfields are available from a variety of sources. Tropical storm input for a model may be based on a separate wind and pressure model.

Tidal elevation boundary conditions can be obtained from the global tidal constituent data bases of Schwiderski (1979) or LeProvost (1995, 1998) or from domestic data bases such as the tidal constituent data base of Mukai, et al (2002). For small domain applications, it is possible specify a single tidal time series along the open water boundary of the grid and neglect tidal potential terms. For applications in which the surge is not large with respect to the tide or there is not a significant amount of overland wetting and drying or barrier island overtopping, it may also be acceptable to model the surge without tides and then to linearly add a reconstructed tidal time series to the surge time series.

Tropical storm models are available from both the public domain and commercial sources. These models range from simple empirical/parametric representations of a hypothetical storm, to more complex planetary boundary layer models that incorporate some of the essential physics of an actual storm event. Regardless of the model selected, output is in the form of windspeed and atmospheric pressure over the computational grid.

Extratropical windfields can be obtained from a variety of sources, including commercial sources. The U.S. Navy Fleet Numerical Meteorological and Oceanographic Center provides hindcast, nowcast, and forecast wind over the world on a fixed delta-Latitude/Longitude basis at fixed time intervals. Wind and weather archives are also available from the National Weather Service (NWS) at specific station locations. For large domain modeling (regional scale), a spatially and temporally variable wind is required. However, for small domain (local scale) projects, a time varying single point windspeed over the full grid may be sufficient. Such data might be obtained from a local airport or from the NWS database.

Regardless of the type of storm event (tropical or extratropical) and the origin of the data used to drive the hydrodynamic model, the data *must be compatible* with the wind drag formulation used in the hydrodynamic model (see below). For example, windfield databases may specify winds at 20-m heights while the selected model formulation may assume 10-m heights. Database units of windspeed can be in ft/sec, m/s, or possibly knots. Care must be taken to assure compatibility in units and in the particular convention chosen for windspeed definition.

Model-Specific Capabilities:

The following list of model capabilities repeats some features of the grid and boundary forcing items mentioned above. They are reiterated below because some aspects of the grid and boundary forcing are also model specific, and are treated differently by different models.

- Governing equations: All two-dimensional (2-D) numerical long wave storm-surge models should solve essentially the same governing equations – the nonlinear equations representing the conservation of mass and linear momentum. For large domain applications, the governing equations must also include the coriolis parameter. For storm surge, it is generally acceptable to assume hydrostatic conditions and constant density in the governing equations.

- The surface stress distribution resulting from storm induced wind is computed through application of some specified wind drag formulation that includes empirical coefficients. Selection of an appropriate formula and coefficients is extremely important because the shear stress is proportional to the square of the windspeed. A formulation/coefficient that is appropriate for extratropical events may not be appropriate for tropical storms. A versatile numerical model should provide the user with capability to specify parameters for a given formulation, including reductions in the presence of vegetation. Based on current research progress, it may soon be a standard procedure to consider allowing the wind stress to be capped at the highest windspeeds, owing to reduction in sea surface roughness during extreme events.

- The bottom drag must also be specified. As for the surface shear stress, a versatile numerical model should allow the user a variety of friction options. These options should include linear or quadratic options as well as provisions for variable resistance to account for the effects of all manner of vegetation and structures which might be encountered in overland flood propagation.

- Wave radiation stress forcing can be included in the numerical model if wave setup is of concern. At the time of this writing, appropriate methods for this are being developed and tested. Although wave setup is usually estimated separately and added to the computed stillwater surge, a coupled approach might be used. Primary drawbacks would be additional modeling complexity and cost; in practice, simpler approaches may be sufficiently accurate and considerably more efficient.

- The need for wetting and drying elements has been noted. Such a capability is mandatory for regions of low relief and large surge, although a model implementation with a fixed shoreline boundary would be acceptable if the terrain rises rapidly, limiting inland flood penetration.

Model Verification:

Verification of the hydrodynamic model is critical to ensure that grid resolution, bathymetry, topography, and boundary conditions are adequately defined. Verification includes elements commonly thought of as *calibration*, although, in general, surge models should not be calibrated in a traditional sense to reproduce observations. Fundamental model parameters such as wind stress coefficients, overland friction factors, and the like, should be based on published best-estimates, and are not free parameters for calibration.

For tides, verification can be achieved by comparing computed data to either measured prototype data collected for a specific time period at the location of interest, or to a multi-constituent (i.e., M2, S2, N2, N1, K1, O1, Q1, and P1) tidal time series reconstructed from published harmonic constituents. These constituents are available from the tidal data base sources such as the NOS or the International Hydrographic Center (IHC). Tidal verification should be achieved to better than 10 percent in both amplitude variation throughout the domain, and phase variation; generally, even better results should be possible. Failure to achieve tidal verification might indicate inadequate grid resolution, especially at inlets and other critical points. Any subsequent model calibration efforts to adjust bottom friction should be limited to values within published ranges for the local hydraulic conditions.

Verification for storm events is more complex. In order to achieve a meaningful result, both the storm conditions (winds and pressures) and the response conditions (such as high-water marks) must be known with accuracy. This is seldom achieved. Actual storm winds and pressures do not faithfully follow simple models, for example, and observed high-water marks may be contaminated by very local wave effects, and may include varying proportions of the local wave setup. Tide gage observations are more reliable, although, again, it is necessary to assess to what degree the record might incorporate setup; it also often happens that tide gages fail prior to the surge peaks of major storms.

In any case, the Mapping Partner shall undertake a thorough verification/hindcast effort for all significant storms that have affected the study area for which high quality data is available. Special hydrodynamic simulations using best wind and pressure estimates are required; such wind and pressure data may be available from Federal agencies, or may be obtained from commercial sources specializing in meteorological data. High-water marks and tide gage records must be evaluated to account for the possible contributions of setup and runup. It should not be expected that an exact comparison will be achieved for any storm. However, given several storms, the observed data should scatter around the model simulations, and not show any large, consistent bias. If certain areas of the grid produce consistently poor comparisons, this may suggest that the grid definition should be carefully reviewed to ensure that area-wide features, such as elongated road embankments, have been accounted for. Of course, it is also important to ensure that the grid represents conditions which prevailed at the time of the storm; barrier island erosion or inlet alterations from prior storms may produce sizeable alterations in a simulation. Consequently, it may be necessary to develop different grid versions for hindcasts, in order to obtain valid results.

No hard and fast rules regarding acceptable model verification are possible, although a careful verification study should permit a reasonable conclusion to be drawn. Should the verification effort be inconclusive, or should poor results be consistently obtained for the historical storm set, the Mapping Partner shall confer with the FEMA Study Representative and with FEMA's technical representatives in order to resolve the issue prior to proceeding with further modeling.

D.2.4.3.5 Storm Climatology

The general topic of storm climatology includes issues of storm data sources and questions of statistical inference needed for storm surge studies. This is an area undergoing rapid development at the time of this writing. Consequently, it is inappropriate to offer firm guidelines at this time. Instead, only some general observations are collected below. The Mapping Partner shall consult the more recent literature at the time of a new study, and shall confer with the FEMA Study Representative and technical representatives for updated guidance.

The historical storm record is needed for two purposes: first, for definition of the characteristics of particular historical storms necessary for hindcast modeling and model verification as discussed above; second, for estimation of storm frequency and frequencies of such storm parameters as may be needed in the statistical simulation effort.

As already noted, extratropical data may be obtained from knowledgeable and experienced Federal agencies, as well as from commercial sources. Similar data for tropical storms and hurricanes is also available. The latter data is more problematic, however, owing to the sporadic quality of hurricanes and to their relatively large spatial gradients in winds and surge (compared with area-wide northeasters, for example). Important northeasters are of such dimension and duration as to affect large coastal areas, including numerous tide gages, and do not generally result in loss of gage data as is often the case for major hurricanes. Consequently, difficulties and limitations of storm climatology are more acute for tropical storms and hurricanes than for extratropical systems.

Critical data for historical storms and model verification studies should be prepared by experienced meteorologists. Such data may be available for significant storms within knowledgeable Federal agencies; new data for historical storms can also be obtained from commercial sources.

For synthetic storm definition as might be needed in a statistical simulation study, other data is needed. Common hurricane data sources include, especially, the HURDAT database of tropical storms in the north Atlantic. This data file purports to include all tropical storms since the mid-19th century, including eye position at six hour intervals, along with the corresponding peak winds and central pressures, as available.

It has become evident in recent studies, however, that the HURDAT data must be used with caution. In particular, the reported winds should not be used for FIS applications. In no event should they be used to back-estimate central pressures using standard empirical relationships between pressure and maximum windspeed. Central pressures given in HURDAT may be used, but are probably of highly variable quality. As a general rule, the highest quality track and pressure data extends back only to the 1970s, when satellite and other modern sensing technology became common.

A prior break in historical quality occurred during World War II, when aircraft and military reconnaissance began to contribute to improved data quality. Consequently, the period between about 1944 and 1970 might be regarded as a transitional period of good, but not best, data quality.

The HURDAT data quality deteriorates as one continues to move back in time. Despite this, the entire record back to the 19th century may be useful for estimation of storm frequency and, more guardedly, for determination of track characteristics. It is the pressures and, especially, the winds which may not be appropriate for FEMA coastal flood insurance studies.

A secondary data source which contains information regarding both central pressure and radius to maximum winds is NOAA's NWS 38, prepared in 1987 especially for FEMA FIS applications. Although this document is now somewhat outdated, it includes a useful table of best estimates of storm pressures and radii for both the Gulf and Atlantic coasts for the period from 1900 to 1984. Data from this source can be used to supplement HURDAT. It is not recommended that the NWS 38 determinations of the probability distributions of storm parameters be used for new flood studies. This is due both to the accumulation of additional data over the last two decades, and to limitations in the analysis methods which were based on storm families defined by landfalling, exiting, and alongshore tracks as referenced to a curvilinear shoreline.

A recent update to tropical storm data since the 1940s has been developed by D. Levinson of the National Climatic Data Center, and has been made available in HURDAT format, although it is not part of HURDAT. This data is among the latest available and might still be considered preliminary; it is being used at the time of this writing in studies along the northern Gulf of Mexico by both FEMA and the USACE. This and other data sources and data compilations are currently in development, largely in response to post-Katrina needs.

In order to confirm storm climatology criteria, the Mapping Partner is advised to confer with the FEMA Study Representative and technical representatives to identify data sources and appropriate methodologies for review and consideration.

D.2.4.4 Water Levels in Sheltered Waters

Water levels in sheltered waters may be influenced by a variety of factors that can alter coastal flood characteristics. Incoming storm surge and the resulting extreme SWELs along the shorelines of sheltered waters may achieve higher elevations than at adjacent open-coast locations owing to channelization and tidal amplification controlled by the orientation, geometry, and bathymetry of the basin; lower elevations may occur if restrictive tidal inlets impede the incoming tide. Factors such as these should be implicitly accounted for in any detailed 2-D storm-surge modeling, and so would not need special attention. However, small basins may also experience higher water levels from the contributions of other mechanisms such as direct precipitation and runoff, or from resonant basin oscillations called *seiche*. These are non-standard factors in a FEMA coastal study, but should be considered by the Mapping Partner if the initial scoping effort suggests that there is reason to believe that the local conditions are such that a special problem or sensitivity might exist.

For studies based not on a detailed 2-D model but on, for example, tide gage analysis, recorded tide elevations may require transposition from the tide gage to a nearby flood study site within the sheltered waters, to better represent the local stillwater elevation during the

1-percent-annual-chance flood event. Some general guidance for evaluating and applying tide gage data to ungaged locations is provided in this subsection, although the Mapping Partner must carefully assess the likely magnitude of error inherent in such approximations, and determine whether a more detailed study might be necessary.

D.2.4.4.1 Variability of Tide and Surge in Sheltered Waters

As a very long wave such as surge or tide propagates though a varying geometry, its amplitude changes in response to reflection, frictional damping, variations in depth causing shoaling, and variations in channel width causing convergence or divergence of the wave energy. In general, these changes are best investigated through application of 2-D long wave models. However, it may be possible to adopt simpler procedures that can provide sufficient accuracy for much less time and cost.

In some cases, tide data may have to be transposed from a gaged site to an ungaged site. If a sheltered water study site is located in the immediate vicinity of a tide gage, the Mapping Partner can use data from the gage without adjustments, but if the study site is distant from the tide gage, the tide data may need to be adjusted so as to reasonably represent the site. It is emphasized that "Considerable care must be exercised in transposing the adjusted observed [tide] data to a nearby site since large discrepancies may result" (USACE, 1986). Although transposition of historic tide data from a nearshore tide gage out to an open-coast location is much simpler and so preferable to its transposition farther inland, there remains a need for reasonable methods to estimate the variation of inland tidal elevations in ungaged regions of sheltered waters.

Some simple empirical evidence may permit an approximate evaluation of these variations, adequate for a FIS:

- Established tidal datums from multiple gages in the sheltered area reflect the natural variation of tide elevations; interpolation between gages gives a first-order estimate of spatial variation patterns

- The normal vegetation line may provide additional information between gages, insofar as it mirrors the general variation of the normal tidal elevation.

- Similarly, observed debris lines and high-water marks from historical storms may illustrate the variation of storm surge within the sheltered geometry, outside the surge generation zone.

Tides and storm surges propagating into sheltered water areas undergo changes controlled by frictional effects and basin geometry. The Mapping Partner must evaluate the differences between the physical settings of the nearest tide gage(s) and the study site, and the distance and hydraulic characteristics of the intervening waterways between these locations to establish a qualitative understanding of the potential differences in tidal elevations between the gaged and ungaged locations. If flood high-water marks are available in the vicinity of the ungaged sheltered water study site, these elevations shall be compared to recorded tide elevations to correlate surge components of the tidal stillwater between locations. In general, surge data are of more limited availability than tide data. It may sometimes be reasonable to assume similarity between surge and tide, and so infer surge variation from known tide variation. The validity of such inference is limited, however, by differences in amplitude and duration of high water from the two processes, and by the fact that tide is cyclic and so may not vary in the same manner as a single surge wave.

Both empirical equations and numerical models can be used to describe the variation of tides and surges propagating into sheltered water areas. The Mapping Partner shall select the most appropriate approach for the study, with consideration of the location of the study site within the sheltered water body, the complexity of the physical processes, and the cost of a particular approach. Appropriate numerical models can range from simple 1-D models to complex 2-D models. The Mapping Partner shall thoroughly evaluate the limitations and capabilities of appropriate models in view of the site-specific issues that need to be resolved to obtain reliable estimates of tidal flood elevations.

For simple tidal inlet settings, or as a first approximation before detailed numerical modeling, Mapping Partners may use analytical methods provided in the CEM (Chapter II-6-2(b)) to estimate bay tide amplitudes. Guidance for estimating the associated inlet parameters is also provided in the CEM. Examples provided in the CEM are limited to estimating the predicted astronomical tide amplitude in a small bay based on an adjacent open-coast tide range obtained from tide tables. These CEM methods may also be applied in a two-step process to transpose recorded tide gage data (SWELs) from one bay to another nearby ungaged sheltered water body as follows:

1. Apply the CEM methods and nomograms in reverse to estimate the adjacent open-coast annual maximum SWELs (astronomical tide elevation plus storm surge height) based on recorded SWELs from a primary tide gage in the sheltered water body closest to the flood study site. The physical setting of a primary tide gage may be such that recorded tide elevations are representative of open-coast tide elevations; however, this condition should not be assumed.
2. Using the estimated open-coast tide elevation, reapply the CEM methods and nomograms (in forward mode) to estimate the associated annual maximum SWELs in the ungaged sheltered water body where the study site is located. Use of the same open-coast stillwater elevation between the gaged and ungaged sheltered water areas is acceptable if it can be assumed that the annual extreme SWELs are generated from regional storm systems large enough in spatial extent to encompass the two locations.

When tidal elevations are to be established in an ungaged sheltered water body, it is recommended that a limited tidal monitoring program be undertaken to estimate tidal datums near the study site. NOAA (2003) provides guidance on methods and computational techniques for establishing tidal datums from a short series of record. The accuracy of the resulting datums may vary insignificantly between a one-month series of data and a 12-month series (NOAA, 2003); a short-term effort will usually be entirely adequate for use in a FEMA FIS of a small sheltered region.

The complex shorelines and bathymetry of sheltered waters may lead to significant changes in tide characteristics. The objective of short-term monitoring should be to provide observed data from which tidal datums may be estimated to check the accuracy of subsequent higher elevation estimates of extremal SWELs in ungaged sheltered water areas and, in turn, to increase the level of confidence in the resulting flood hazard elevations.

Irrespective of the approach taken, the Mapping Partner shall evaluate the physical setting of the tide gage(s) from which data are used. Observation of the gage setting may provide insight into the relative degree of sheltering or other characteristics of a given tide gage. Information on NOAA tide gages can be obtained at <http://www.co-ops.nos.noaa.gov/usmap.html>. Mapping Partners shall also determine if a tidal benchmark has been established near the flood study site (<http://tidesandcurrents.noaa.gov/bench.html>). Tidal benchmarks are elevation reference points near a tide gage to which tidal datums are referenced. Some tidal benchmarks are now tied to the NAVD88, or to the earlier NGVD29, providing an appropriate vertical elevation reference. Benchmark elevations may become invalid over time if changes occur in local tide conditions because of dredging, erosion, or other factors; the Mapping Partner shall review the publication date of the data together with information concerning any recent changes in the vicinity of the tide gage setting to ensure the data are appropriate.

If the physical setting and tidal processes of a coastal flood study site are particularly complex and the application of the simple methods described in the CEM are questionable, the Mapping Partner must confer with the FEMA Study Representative and technical representatives for further guidance on estimating tidal and surge elevations at ungaged sites.

D.2.4.4.2 Tidal Inlets

Tidal inlets control the movement of water between the open coast and adjacent sheltered waters. Inlets may be broadly classified as unimproved (natural) or improved (maintained). The physical opening of a tidal inlet, whether natural or maintained, has a direct and often significant effect on the propagation of tides, surge, and waves into sheltered waters and on subsequent coastal flood conditions. The Mapping Partner shall review the CEM Section II-6-2 on inlet hydrodynamics for comprehensive guidance on data, methods, and example problems related to the behavior of currents and waves at tidal inlets, for possible application in simplified studies within sheltered waters.

D.2.4.4.3 Seiche

Seiche is a low frequency oscillation occurring in enclosed or semi-enclosed basins, which may be generated by incident waves or atmospheric pressure fluctuations; seiching may also be called harbor oscillation, harbor resonance, surging, sloshing, and resonant oscillation. It is usually characterized by wave periods ranging from 30 seconds to 10 minutes, controlled by the characteristic dimensions and depth of the basin (CEM, 2003).

The amplitude of seiche is usually small; the primary concern is often with the associated currents that can cause large excursions and damage to moored vessels if resonance occurs. However, surface elevations and boundary flooding in an enclosed basin may become pronounced if the incoming wave excitation contains significant energy at the basin's natural seiche periods. The Mapping Partner shall investigate the likelihood of seiche under extreme water-level and wave conditions if the pre-project scoping effort indicates that a sensitive site has been affected by seiche during past storms. Bathymetry, basin dimensions, and incoming wave characteristics should be reviewed to determine the potential for seiching; the CEM

(Section II-5-6) provides background and guidance for estimating the natural periods of open and closed basins. Numerical models are most appropriate for evaluating the effects of long waves in enclosed basins and shall be considered for use in a sheltered water study if seiching is believed to have the potential to contribute significantly to boundary flooding during the 1-percent-annual-chance flood condition.

D.2.4.4.4 Documentation

The Mapping Partner shall document the characteristics of all gages located within or near the sheltered water study area. Methods adopted to infer the variation of tidal datums between gages shall be documented, as shall procedures used to transpose data from one site to another. If a brief field effort is undertaken to determine the variation of tidal datums within ungaged regions, the Mapping Partner shall fully document that effort, including: locations of observations; observation methods and instrumentation; dates and times of all observations; meteorological and oceanographic conditions during and preceding the period of observation; and other factors that may have had an influence on water levels, or may affect interpretation of the results. If surge variation is inferred from tide variation, the Mapping Partner shall document the basis for similarity assumptions, and the manner in which the inferences were made. Inlet analyses should be documented including all procedures, methodological assumptions, field surveys (dates, times, procedures, instrumentation, and findings), and all inlet data adopted from other sources.

D.2.4.5 1-Percent-Annual-Chance Stillwater Levels

The 1-percent-annual-chance flood on the Atlantic and Gulf coasts is not often the result of stillwater alone; other processes such as wave setup, wave heights, and wave runup ride atop the stillwater, which serves as a base. The exception might be well-sheltered areas, protected from waves and affected only by the high SWELs associated with tide or surge. Even in such areas, however, the total 1-percent flood level may include a physically independent contribution from rainfall runoff.

Consequently, there are two aspects of stillwater statistics for a Mapping Partner to consider: What is the 1-percent-annual-chance SWEL at a site? How does stillwater contribute to the total 1-percent level? Even if it is known that the BFE at the study site is determined by wave runup, for example, the former question may not be irrelevant, and the Mapping Partner may need to estimate the 1-percent SWEL separately from the higher BFE.

Two distinct stillwater components can be identified: astronomic tide and storm surge (wind and pressure setup). A third stillwater component is important in sheltered waters, but is not the result of coastal processes as are the others. This is the superelevation of tidal waters associated with rainfall runoff. The riverine 1-percent flood profile along a tidal river typically begins near MHW or MHHW at the mouth, and rises as one proceeds upstream. Although the riverine flood level along the lower reaches of the tidal river may be physically unrelated to coastal flood processes, the final flood mapping must represent the contributions of both mechanisms. Consequently, the rainfall runoff excess elevation may be considered a third type of coastal stillwater elevation.

The following subsections address methods by which the statistics of each stillwater type may be determined, and also give an overview of the ways in which the statistics of combined processes can be addressed.

D.2.4.5.1 Tide Statistics

The astronomic tide is a deterministic process. Consequently, tide statistics can be generated directly from the local tidal constituents. One simple way to do this is to sample the predicted tide at random times throughout the tidal epoch. Alternatively, predictions can be used to obtain highs and lows, from which corresponding statistics can be derived. It is noted that the maximum possible tide is given simply by the sum of the amplitudes of the 37 tidal constituents.

D.2.4.5.2 Surge Statistics

The development of surge statistics can be approached in two general ways. First, if sufficient data are available from tide gage records, then an extremal analysis of the residual after subtraction of the astronomic tide can be performed. As noted above, this requires determination of the annual peak residuals for the period of record, and a fit to a GEV or other appropriate distribution using the method of maximum likelihood (or an alternate acceptable method). The Mapping Partner should keep in mind that the 1-percent level determined in this way will include the contributions of all mean water components affecting the gage, including both static wave setup *to the degree it exists at the gage site*, and riverine rainfall runoff.

The second way in which 1-percent surge levels are determined is through numerical modeling of surge elevation using 1-D or 2-D models, as discussed above, combined with a statistical model relating the surge simulations to storm frequency and storm parameter distributions. Three ways of doing this have been used: the JPM, which has been used in many FEMA flood studies on the Atlantic and Gulf coasts in combination with the FEMA Storm-surge model; the more recent EST, which has been used in combination with the ADCIRC model for recent studies; and a Monte Carlo approach, which has been used for coastal setback determinations in the Florida Department of Environmental Protection, and which is particularly suited for use with the 1-D surge model, BATHYS, described previously. Because surge levels on the Atlantic Ocean and Gulf of Mexico are generally large, it is expected that JPM and EST studies with large 2-D surge models will most often be necessary. The 1-D BATHYS model with Monte Carlo simulation, or – more directly – with direct simulation of the wind record using, say, GROW data, may be adequate in some cases. Brief descriptions of the JPM, EST, and Monte Carlo methods are given in Subsection D.2.3.6.

D.2.4.5.3 Combined Effects: Surge Plus Tide

The simulation of storm surge is usually performed over water depths representing mean conditions, or some other fixed level. The 1-D Monte Carlo approach in which tide is incorporated as a time-dependent boundary condition is a notable exception.

Because tide is ubiquitous, the flood level associated with storm surge must be based on the combined surge-plus-tide levels. Four approaches of differing complexity are mentioned here.

First, if the surge and tide can be assumed to combine linearly (that is, neither is physically altered to an important degree by the presence of the other), then the simplest method is to simply add them in some manner. If a surge episode – such as a northeaster – is of relatively long duration compared with a tidal cycle, then high tide will be certain to occur at some time for which the surge is near its peak, and a simple sum of amplitudes may be sufficiently accurate.

However, if the surge duration is short – such as may be typical for hurricanes in northern latitudes – this approximation is inadequate. The next simplest assumption, still assuming linear superposition, is based on the fact that the PDF for a sinusoid is largest at its extrema – tide is generally near a local high water, or near a local low water, and spends more time near those values than in between. It may be reasonable, then, to assume that the peak surge occurs with equal probability near a high tide or near a low tide, taking mean high and mean low as representative values. Each of the corresponding elevation sums would be assigned 50 percent of the rate associated with the particular storm (as if each storm were to occur twice, once at high tide and once at low tide), and the frequency analysis would proceed with these divided rates.

A third, slightly more complex approach but still assuming physical independence, is based on the convolution method mentioned in Subsection D.2.3.3. In this method, the PDFs for both tide and surge without tide are used. Previous discussion has shown how both of these may be established. If the probability density of the tide level Z is denoted by $p_T(Z)$ and the probability density of the surge level is $p_S(Z)$, then the probability density of the sum of the two is given by:

$$p(Z) = \int_{-\infty}^{\infty} p_T(T) p_S(Z-T)\, dT = \int_{-\infty}^{\infty} p_T(Z-S) p_S(S)\, dS \tag{D.2.4-5}$$

where the indicated integrations are over all tide and surge levels.

In some cases, however, the essential assumption that the tide and surge can be linearly added is not satisfied. In shallow water areas extending a large distance inland, the enhanced depth associated with tide (or surge) affects the propagation and transformation of the surge (or tide). That is, there is a nonlinear hydrodynamic interaction between the two. In such a case, more complex methods are required because the nonlinear interaction can only be accounted for by hydrodynamic considerations, not by any amount of purely statistical effort. Two approaches to this issue have been adopted in study methods already identified. The FEMA storm surge methodology adopts a procedure in which a small number of storms are simulated around a set of tide assumptions with differing amplitudes and phases. These additional simulations are used to provide guidance for simple adjustments that are made to the large set of computations performed on MSL. The EST approach treats astronomic tide (amplitude and phase) as additional input vector components, which are incorporated into the hydrodynamic model as part of the boundary conditions. The 1-D Monte Carlo approach includes tide as part of the surge simulation and so does not require a separate step to combine the two.

Should the Mapping Partner be required to perform 2-D surge modeling, it will be necessary to consult the user's manuals or other documentation of the adopted models to obtain additional guidance on this topic.

D.2.4.5.4 Combined Effects: Surge Plus Riverine Runoff

The final instance of combined stillwater frequency to be described here, concerns the determination of the 1-percent SWEL in a tidal location subject to flooding by both coastal and riverine mechanisms. This is the case in the lower reaches of all tidal rivers.

The simplest assumption is that the extreme levels from coastal and riverine processes are independent, or at least widely separated in time. This assumption is generally acceptable because the storms that produce extreme rainfall and runoff may not be from the same set as the storms that produce the greatest storm surge. Furthermore, if a single storm produces both large surge and large runoff, the runoff may be significantly delayed by the time required for overland flow, causing the runoff elevation to peak after the storm surge. Clearly, there may be particular storms and locations for which these assumptions are not true, but even so they are not expected to be so common as to strongly influence the final statistics. If, for a steep terrain area of the east US coast, it is thought that peak runoff and peak surge may commonly coincide owing to local conditions, then the Mapping Partner must consider the likely correlation between the two, and discuss with the FEMA Study Representative whether a departure from the method given here should be used.

The simplified procedure is straightforward, beginning with development of curves or tables for rate of occurrence vs. flood level for each flood source (riverine and coastal). Rate of occurrence can be assumed equal to the reciprocal of the recurrence interval, so the 100-year flood has a rate of occurrence of 0.01 times per year. This is numerically equal to what is more loosely called the *flood elevation probability*. Then one proceeds as follows at each point of interest, P, within the mixed surge/runoff tidal reach.

1. Select a flood level Z within the elevation range of interest at point P.
2. Determine the rates of occurrence $R_{P,R}(Z)$ and $R_{P,S}(Z)$ of rainfall runoff and storm surge elevations exceeding Z at site P (number of events per year).
3. Find the total rate $R_{P,T}(Z) = R_{P,R}(Z) + R_{P,S}(Z)$ at which Z is exceeded at point P, irrespective of flood source.
4. Repeat steps (1) through (3) for the necessary range of flood elevations.
5. Plot the combined rates $R_{P,T}(Z)$ vs Z and find $Z_{P,100}$ by interpolation at $R_{P,T} \approx 0.01$.
6. Repeat steps (1) through (5) for a range of sites covering the length of the mixed tidal reach.
7. Construct the 100 year composite profile passing through the several combined 100-year elevation points, and blending smoothly into the pure-riverine and pure-surge 100-year profiles at the ends of the mixed reach.

The procedure is shown schematically in Figure D.2.4-3 in which the combined curve has been constructed by addition of the rates at elevations of 6, 8, 10, and 12 feet. The entire procedure can be implemented in a simple hand calculator program, with the input at point P being the 10-, 50-, 100-, and 500-year levels for both runoff and surge, as obtained from standard FIS report tables.

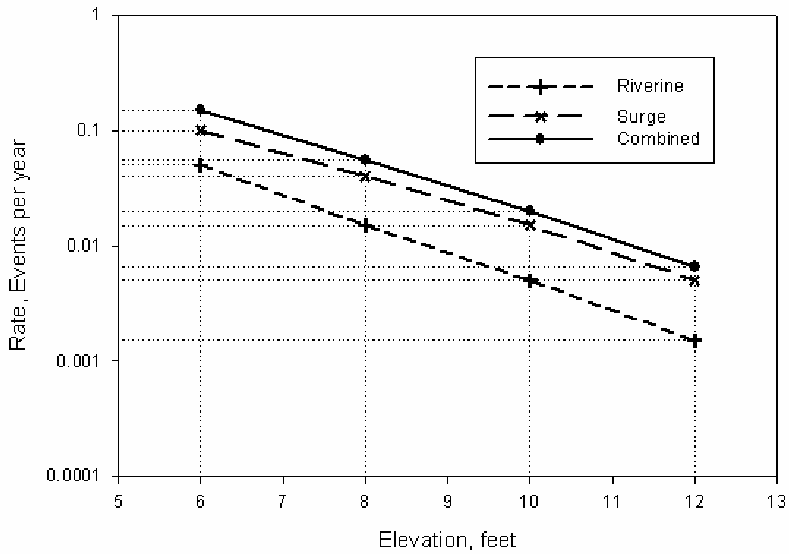

Figure D.2.4-3 Schematic Illustration of Riverine and Surge Rate Combination

D.2.4.6 Nonstationary Processes

Conceptually, a *stationary process* may be thought of as one that does not change in its essential characteristics over time; its descriptors are fixed or stationary. For example, a stationary random process would be one for which its mean, standard deviation, and other moments are unchanging over time. A nonstationary process is one for which these measures do change. Whether a fluctuating process is thought to be, or appears to be, nonstationary can depend upon the time window through which it is viewed. Processes that appear to display definite nonstationary trends when viewed at a short scale, may be seen to fluctuate around an unchanging mean when viewed from a more distant perspective. For example, the tide appears nonstationary when viewed over a period of one hour, but appears entirely stationary when viewed over an entire 19 year tidal epoch.

The appropriate time window for FEMA flood studies is established by the period of record covered by the available data on the one hand, and the probable lifetime of a particular study, on the other.

For practical FIS considerations, two sorts of nonstationarity seem significant. The first is the apparent change of sea level, which has been observed on all coasts. Because it is sea level relative to land that is most significant, an apparent change of sea level can be the result of either sea-level rise, or land subsidence.

The second type of nonstationarity that is important for coastal studies is the long-term change in tidal datums, which may occur as basins evolve through silting, dredging, migration and evolution of inlets, human construction including harbor improvements and breakwaters, and so forth. Both types are discussed below.

D.2.4.6.1 Relative Sea Level – Sea-level Rise

Sea level rise appears to be a real, long-term effect observed all along the U.S. coastline. The Philadelphia District of the USACE maintains a useful collection of sea-level rise links at their website <http://www.nap.usace.army.mil/cenap-en/slr_links.htm>. There is also a very large set of sea-level trend data for individual stations along the Atlantic and Gulf coasts, which can be obtained from the referenced NOAA site.

The significance of such data is two-fold. First, the Mapping Partner must be aware of these changes to properly interpret historical data upon which new studies might be partly based. This has been discussed, for example, in a prior subsection on tides. Second, the likely continuation of these trends into the future will have some impact, although usually small, on the interpretation of today's FIRMs at a future date. In particular, the Mapping Partner should consider the likely impact of sea-level rise on floodplain boundary delineations, and document any unusual changes that might be anticipated.

D.2.4.6.2 Relative Sea Level – Land Subsidence

Land subsidence produces the same sort of effect as sea-level change – a rise in the apparent sea level – but subsidence might be much the more significant factor in a local area. Many areas along the coasts of Texas, Louisiana, and Mississippi have subsided by several feet as a result of gas, oil, or water extraction over the past few decades.

Such large displacements make it imperative that historical data be interpreted with caution. The Mapping Partner must ensure that gage datums have been properly adjusted over time so that water-level records, benchmarks, observed high-water marks, and all similar data are properly interpreted, and properly related to current conditions represented in the hydrodynamic model grid.

The USGS is a primary repository of land subsidence data for the United States, and should be consulted to obtain local site information covering the entire period of study data that might be compromised by unrecognized subsidence. The USGS web pages may be searched for local subsidence information at <http://search.usgs.gov/>.

Other data sources may be more helpful in some cases. The Mapping Partner should consult with local city and county engineering departments, and the professional surveying community, which may be aware of isolated subsidence issues not reflected in the national programs.

D.2.4.6.3 Astronomic Tide Variation

Tide datums and tidal constituents may change over time owing to changes in the geometry of a tidal basin, so that tide may also constitute a nonstationary process. This makes it imperative that tide predictions for prior years (hindcasts) be made using tidal constituents appropriate to that time, and that tidal data be adjusted as necessary for shifts in tidal datums with respect to a fixed datum such as NAVD88 or NGVD29. The NOAA website can provide predictions for past times, but all such predictions are made using the current default set of constituents, and so may inaccurately portray past tide levels and datums. Archived copies of tidal constituents can be obtained from NOAA by special request. Flexibility in applications such as these makes it wise to use a tide prediction program such as NOAA's own program, NTP4.

D.2.5 Wave Determination

This subsection provides guidance for estimating wave conditions, from the region where the waves are generated by wind blowing across the water surface to the shoreline. The generation, transformation, and attenuation of waves are addressed.

D.2.5.1 Overview

One of the ultimate objectives of flood hazard studies is to determine wave dimensions on land areas flooded during the base flood. These overland wave dimensions are used in conjunction with stillwater flood levels to determine BFEs and flood insurance risk zones.

Estimation of wave dimensions on land requires knowledge of incident wave conditions at the shoreline during the base flood, as well as upland topography, development, and frictional characteristics. Incident wave characteristics at the shoreline will depend upon the wave characteristics that result from wave generation in the offshore and/or nearshore regions, shoaling effects, and, in some cases, wave attenuation cause by nearshore bottom interactions (e.g., wave dissipation due to bottom friction, bottom percolation, and/or movement of a cohesive [muddy] bottom).

The general study process is summarized in Figures D.2.5-1, D.2.5-2, and D.2.5-3.

Open-coast shorelines without wave attenuation as a result of nearshore bottom effects will result in depth-limited waves at the shoreline during the base flood, and the study procedure will follow the path shown on Figure D.2.5-1.

Sheltered water shorelines[1] without wave attenuation as a result of nearshore bottom effects may or may not result in wave heights smaller than depth-limited heights. The Mapping Partner will have to make this determination based on wave generation and fetch conditions during the base flood[2]. In the case of depth-limited waves, the study process will follow the path shown on Figure D.2.5-1; in the case of wave heights smaller than depth-limited heights, the study procedure will follow the path shown on Figure D.2.5-2.

Shorelines subject to waves that are attenuated as a result of bottom effects will experience wave heights less than depth-limited heights, and the study procedure will follow the path shown on Figure D.2.5-3. Note that this scenario could be used in both open coast and sheltered water situations.

[1] See Sections D.2.5.3 and D.4.2.2.1 for a discussion of sheltered waters.
[2] FEMA's model for overland wave propagation (WHAFIS) automatically assumes depth-limited waves if the fetch is 24 miles or greater.

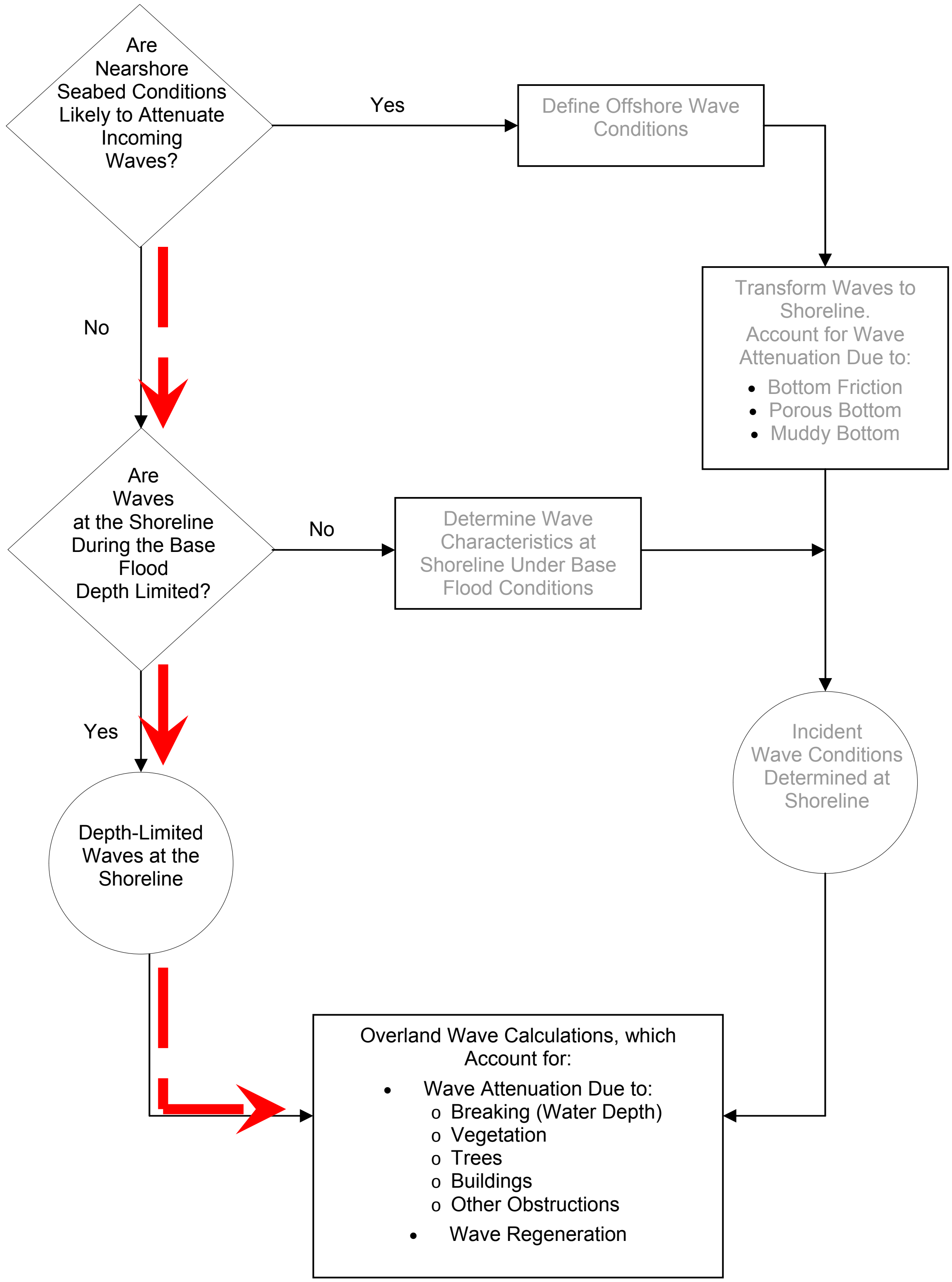

Figure D.2.5-1. Flow Chart for Determining Incident and Overland Wave Dimensions, Open Coast or Sheltered Water Shorelines without Wave Attenuation Due to Nearshore Bottom Effects; Depth-Limited Waves at Shoreline.

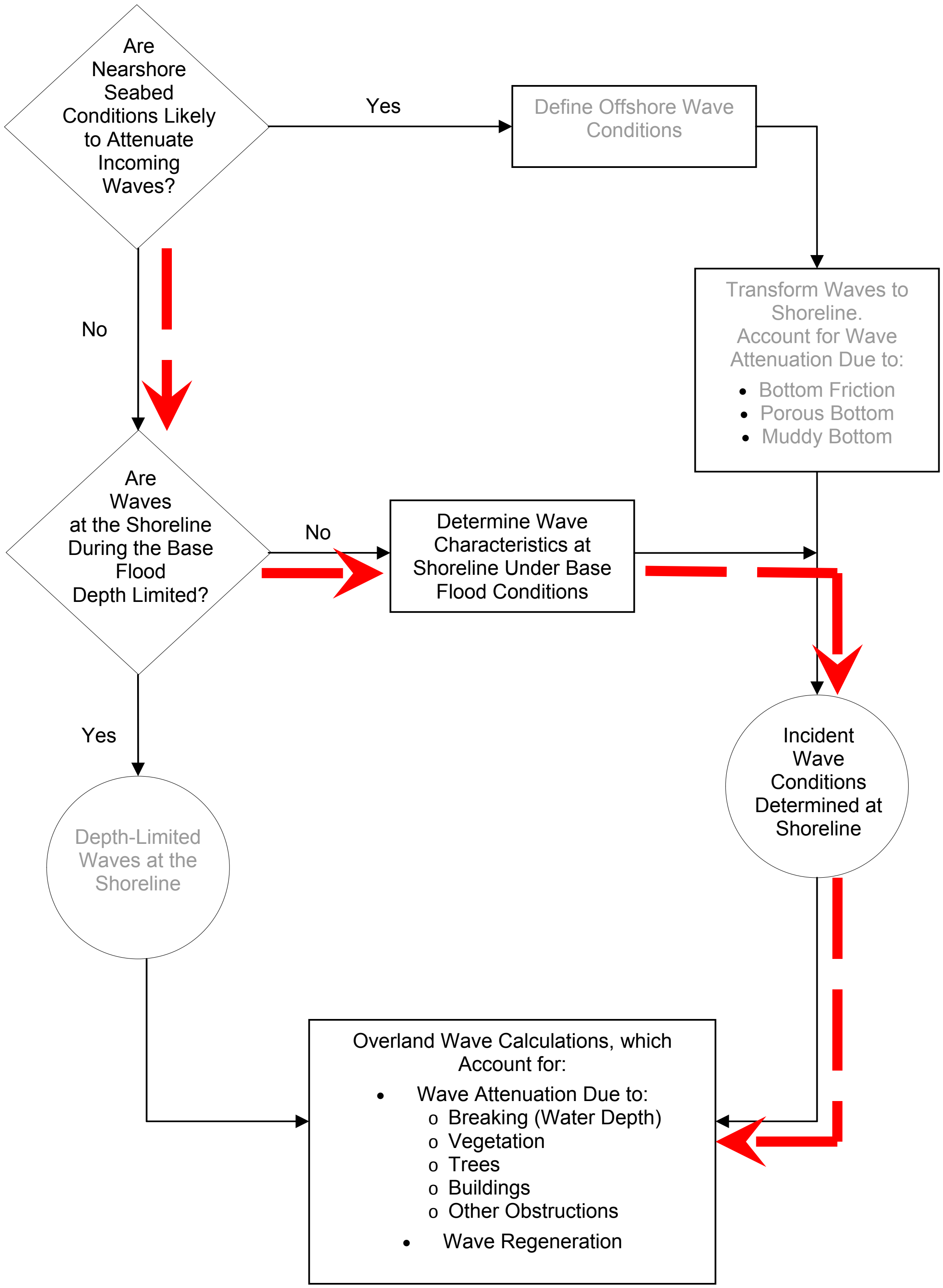

Figure D.2.5-2. Flow Chart for Determining Incident and Overland Wave Dimensions, Sheltered Water Shorelines <u>without Wave Attenuation</u> Due to Nearshore Bottom Effects; <u>Less than Depth-Limited Waves at Shoreline</u>.

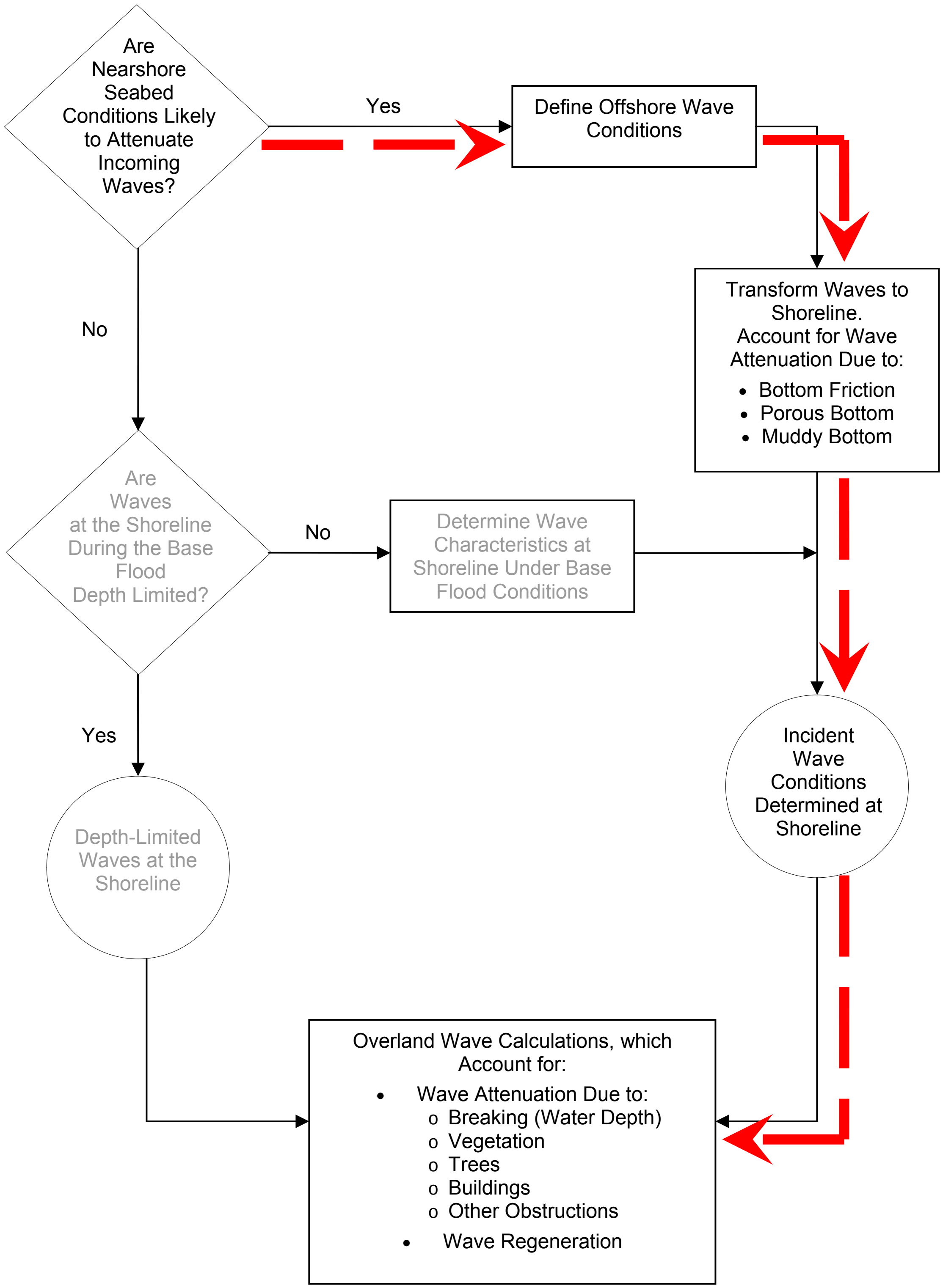

Figure D.2.5-3. Flow Chart for Determining Incident and Overland Wave Dimensions, Open Coast or Sheltered Water Shorelines with Wave Attenuation Due to Nearshore Bottom Effects.

Wave energy is dissipated when waves propagate over relatively broad, shallow areas. The dissipation can be caused by increased bottom friction, percolation in sandy seabeds, the movement of cohesive seabeds, and drag induced by vegetation (see Figure D.2.5-4 for a conceptual definition sketch). Dissipation mechanisms can result in smaller wave heights than predicted by typical shoaling and depth-induced breaking relationships. Available analysis methods rely on parameters that have a wide range of values and can be difficult to reliably quantify. The overall approach required to quantify dissipation may entail the use of empirical data, possibly collected by the Mapping Partner at the study site or available from a similar site. In most situations, the amount of dissipation will be small, and the effort required to analyze the dissipation processes can be great. In addition, the risk of overestimating wave dissipation with the available tools, resulting in an underestimation of flood risk, can be significant.

For the Atlantic and Gulf coasts, wave attenuation caused by bottom effects is likely to be a rare situation and will not be part of the typical flood study process. In instances where bottom effects are known to be a significant factor in the attenuation of waves (for example, portions coastal Louisiana with large expanses of muddy bottom in the nearshore), the Mapping Partner shall consult with the FEMA Study Representative prior to finalizing the study approach. If wave attenuation due to bottom interactions is to be included in the study, procedures outlined in Subsection D.4.5.3.2.1 should be used.

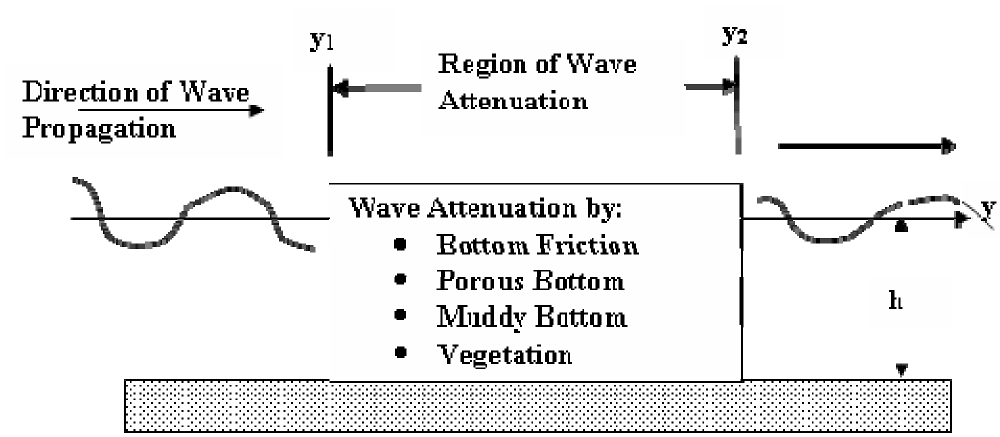

Figure D.2.5-4. Schematic of Wave Attenuation Processes Caused by Bottom Effects

D.2.5.2 Open Coasts

Many areas in which flood mapping is to be conducted are open coast settings. These open coasts may be characterized by a variety of morphologies ranging from a nearly straight coastline, as along many portions of the east coast of Florida, to convoluted coastlines, as near the entrances of large embayments such as the Chesapeake Bay. In order to carry out the wave setup and wave runup calculations required to conduct flood mapping, nearshore wave information must be developed. This requires first quantifying deepwater waves and then transforming these waves to the shoreline. Each of these steps is discussed below. A simplifying factor in the case of wave runup is that the nearshore wave height will usually be depth limited; thus, the determination of deepwater wave height will not be important to this determination. Wave setup depends more directly on the deepwater wave height; however, in areas where hurricanes govern, wave setup may account for only 20 to 25 percent of the total surge, thus perhaps relaxing the need for high accuracy in the determination of wave characteristics. In areas where extratropical storms are dominant in the determination of flood mapping, wave setup may represent a greater percentage of the total surge. The selection of a method for determining the deepwater wave height requires the Mapping Partner to evaluate the sensitivity of the total surge value of interest to the deepwater wave height and the capabilities of the various available methods to provide the requisite accuracy.

D.2.5.2.1 Wave Source

FEMA's mapping program is evolving towards the application of 2-D wave models, rather than the use of a single wave with defined characteristics. Additionally, FEMA appears to be moving toward the use of EST methodology rather than the traditional JPM in the mapping process. These approaches were discussed in detail in Subsection D.2.3. The most effective approach to determine nearshore wave conditions may depend on the type of methodology (EST or JPM) selected.

Three generic types of wave models are described below: (1) detailed 2-D models, (2) a simplified 2-D method that is a modification of a method originally developed by Bretschneider, and (3) a 1-D transect method. In addition, combinations of these methods may be possible. The model selected shall be discussed with and authorized by FEMA.

Additionally, hindcast wave data may already be available for a study area, and the Mapping Partner should investigate such data before undertaking detailed modeling for an FIS. One source of hindcast data is the USACE WIS project, described at http://frf.usace.army.mil/cgi-bin/wis/atl/atl_main.html.

D.2.5.2.1.1 Detailed Two-Dimensional Models

The 2-D wave models require a windfield and bathymetry/topography as input and have the advantage of determining wave conditions from deepwater to the breaking zone. Several models that are available for this purpose will be discussed later in this subsection. A disadvantage of 2-D models is that they add a considerable computational load to the overall process; however, computational power is increasing rapidly and several mapping efforts in which 2-D wave modeling is being applied are underway (as of July 2006). One question that the Mapping Partner must address in designing the computational process is the link between the storm-surge model and the wave model. One approach is to complete the storm-surge model run to determine the water elevations caused by wind and atmospheric pressure, and then run the wave model with these modified water levels. However, in shallow water and over land, the wave setup contributes to the water depth and thus affects the wind surge component. A second, more interactive and ambitious, approach is to run the surge model for a certain time, then run the wave model through wave setup, recompute the total water level, then continue with the wind surge model, and so forth.

Several publicly available wave models are available for 2-D modeling. These include WAM, SWAN (a shallow-water version of WAM), STWAVE, and REFDIF. In addition, commercial models are available, including the Danish Hydraulic Institute “Mike” series and Delft 3D. As noted, these models require the windfield and the bathymetry/topography to be input over the area of interest. Each of the available models has advantages and disadvantages. The Mapping Partner shall review the characteristics of the various available models and ensure that the selected model is appropriate for the particular conditions of the area to be mapped. In addition to reviewing the published characteristics of the models, the Mapping Partners should discuss their experiences with similar applications with other users. If the Mapping Partner has a successful experience with a particular model, this may help others select a model.

The use of a detailed 2-D wave model will usually require more than one grid system to be used, with the outer grid elements coarser than those of the grid system(s) closer to shore. These grid systems will usually, be “nested;” that is, the coarser grids will extend to shore, and their output will be used as input for the finer grids on their boundaries.

D.2.5.2.1.2 Simplified Two-Dimensional Models

In addition to detailed 2-D models, as described above, simplified 2-D parametric models are available for application. One such model, modified from a procedure presented in the Shore Protection Manual and based on the work of Bretschneider, is described in Subsection D.2.5.1 in the presentation of a procedure for calculating wave setup. This method is based on calculating the fields of deepwater wave height and period, using the hurricane parameters as input (central pressure deficit, radius to maximum winds, and forward translation speed). The modifications to the Bretschneider method, as presented in Subsection D.2.5.1, include recommendations for calculating the equivalent wave characteristics at shore with the hurricane at arbitrary distances from the shoreline.

If this method is of interest to the Mapping Partner, it should be verified, by one or more comparisons with 2-D models, that this simplified method provides sufficiently accurate results.

D.2.5.2.1.3 One-Dimensional Transect Method

The 1-D transect approach is the traditional FEMA methodology for shallow-water computations, but it may also be applied for deepwater conditions. The shallow-water applications consider a particular transect, and the waves and storm surge are determined along the transect for a specified windspeed field. One application of the transect method is to calculate waves to a nearshore location by a detailed 2-D model and then to apply the 1-D transect method for more landward locations. This method allows for ready application of detailed characteristics along the transect, such as bottom friction or vegetation characteristics. The transects are spaced commensurate with the longshore variability of the bathymetry/topography over which the waves are propagating. Following the transect calculations, the results may be interpolated in an alongshore direction to establish conditions between transects. The advantages of the transect method include the capability to calculate waves and wave setup in the same program. Thus, this involves a tradeoff between the detailed 2-D method, which requires iterations of the storm-surge modeling and the wave modeling, and the transect method, which conducts both wind surge and wave setup simultaneously. An additional advantage of the transect method is that the grid spacing along the transect can be sufficiently detailed to calculate wave setup, which may vary substantially over a fairly narrow cross-shore zone. Examples of this method and a further discussion will be presented in a later subsection.

D.2.5.2.2 Wave Transformation

Wave transformation describes the process by which waves are modified as they propagate from deepwater toward shore. Wave transformation processes include growth, refraction, diffraction, reflection, and dissipation. Of these processes, especially in natural areas, wave growth, refraction, and dissipation are generally of the greatest significance. Dissipation is generally of greatest significance in the shallower portions of the profile, although dissipation over long distances in deeper water may reduce the wave height considerably unless the wave system is in an active generation area.

For flooded areas, wave transformation includes the modifications to the waves as they propagate over dissipative bottoms and through vegetation and structures. Various approaches are available for calculating wave transformation. Some of these methods have been developed into computer programs, including FEMA's computer program WHAFIS, which is applied to a transect and will be discussed in greater detail in Subsection D.2.7.

An advantage of the 2-D models is that they account for several wave transformation processes. In particular, wave refraction is included in all of the models, and physics-based wave diffraction is included in some of the models. Publicly available 2-D models may include transformation processes with spatially variable bottom friction factors. The Mapping Partner shall review the characteristics of the various models and select a model that provides an appropriate "match" to the needs and resources of the particular mapping effort.

D.2.5.3 Sheltered Coast

Some features of the most appropriate methodology for sheltered coasts may be similar to those for open coasts. Again, windfields will be required and may be associated with tropical or extratropical storms. Portions of the Mississippi coastline, with barrier islands some 6 to 8 miles seaward of the mainland, represent a special case of a sheltered water body that is coupled with Gulf waters through inlets incised through the barrier islands. With high storm surges, as occurred in Hurricane Katrina, these barrier islands become inundated and the system is modified from a sheltered coast to an open-coast system. Lake Pontchartrain in Louisiana is a sheltered water body that is coupled to Lake Borgne through two passes.

As noted, the methods for determining wave heights for sheltered waters may be based on the same detailed models as those developed for open coasts, or on transect methods developed by FEMA, USACE, or others. The Mapping Partner shall investigate the range of possible models for their application to the particular geometry and determine the most appropriate method. As in the case of open coasts, if the determining storms are hurricanes, as they will be in the Gulf of Mexico and lower east coast, the windfield may be based on the parameters of each hurricane (JPM method), accounting for any wind reduction as the wind traverses over land before reaching the sheltered water, or directly for historical storms, if the EST method is applied. If the determining storms are extratropical, they are usually of larger scale than hurricanes, and the winds may be determined by examining the historical occurrences of storms and applying one of the previously discussed models that transforms winds to waves and storm surge. In some cases, tide gage data may be adequate to determine the surge levels of interest. However, depending on the water depth in which the tide gage is located, the wave setup included in the tide gage recordings may not be representative of wave setup at the shoreline.

D.2.5.4 Additional Considerations

D.2.5.4.1 Extratropical Storms

For some extratropical storms, such as those that dominate along the northeast coast, it may be appropriate to use a wave height with an established return period. The possible databases for this wave height include the WIS data developed by USACE or the Global Reanalysis of Ocean Weather (GROW) data that have been developed by Ocean Weather and are available commercially. The GROW data are based on the analysis of several decades of wind data and the more limited buoy data and can be extrapolated to the return period of interest. The Mapping Partner should compare results from these databases, and if they differ significantly, attempt to resolve the cause of the differences and select the most appropriate data source for further computations. In some areas, sufficient tide gage data may be able to serve as the basis for calculation of the BFE.

D.2.5.4.2 Wind Characteristics

Even in the case of windspeeds, which are constant when averaged over a long period, winds vary about the average. This raises the question of the appropriate windspeeds to use to calculate waves and storm surges. Windspeeds are usually reported as the maximum windspeed when averaged over a specific time interval. For example, the 1-minute windspeed would represent the fastest windspeed, averaged, in a 1-minute period. The 1-minute periods considered when determining the fastest 1-minute windspeed should be measured when the average windspeed is constant. Obviously, the speed representative of a particular windspeed decreases with the averaging interval. That is, the 1-minute windspeed will be greater than the 10-minute windspeed. For structural damage, it is usually the 3-second wind gust that is considered relevant. Statistically, this is on the order of 30 percent greater than the 1-minute windspeed. The appropriate windspeed for wind and storm surge computations is the 30-minute windspeed. Winds are present as a boundary layer over land and water, and the windspeed increases with elevation. The relevant height for calculating the windspeed for storm surge and waves is 33 feet (meteorologists use 10 meters, which is 32.8 feet). If the windspeed is available at an elevation, z, which differs from 33 feet, then the following relationship may be applied:

$$U(33) = U(z)\left(\frac{33}{z}\right)^{1/7} \tag{2.5-1}$$

in which *U* is the windspeed.

D.2.5.5 Documentation of Wave Attenuation

Areas where wave attenuation was examined and the results obtained shall be described. The characteristics of these areas that led to the consideration of wave attenuation and the values of the attenuation parameters used in the analysis shall be quantified. Results of interest include the potential effect of wave attenuation on the hazard zones and the decisions reached as to whether to further include wave attenuation in the analysis leading to hazard zone delineation. Any field measurements and/or observations shall be recorded, as well as documented or anecdotal information regarding previous overland damping during major storms, perhaps by runup events less than expected in the lee of attenuation features, as discussed in this subsubsection. Any notable difficulties encountered and the approaches to addressing them shall be clearly described.

D.2.6 Wave Setup

This subsection provides guidance for the determination of wave setup—an increase in the total stillwater elevation against a barrier caused by the attenuation of waves in shallow water.

D.2.6.1 Overview

In addition to wind, waves can also affect the mean nearshore water levels during hurricanes and severe storms. This occurs as a result of the transfer of momentum from waves to the water column (see Figure D.2.6-1). Wave setup increases as the water depth near a barrier decreases and wave dissipation increases.

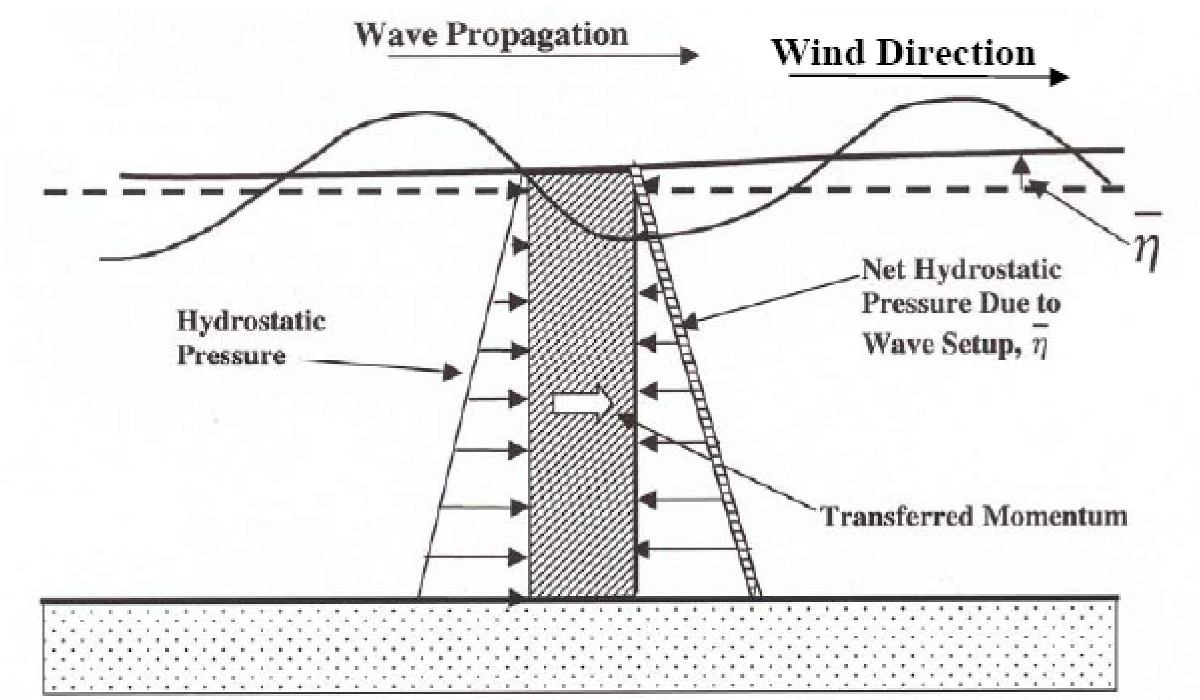

Figure D.2.6-1. Wave Setup Due to Transfer of Momentum.

Consider a train of waves approaching the shoreline. Outside of the breaker zone, a relatively small reduction in mean water level, termed a "setdown," will occur. This setdown is small, approximately 5 percent of the breaking wave height. However, as the waves break, they transfer momentum to the water column, causing a wave "setup" that can be on the order of 10 to 20 percent of the breaking wave height. This is a "static" wave setup, which remains approximately constant as long as the storm tide and incident wave conditions remain unchanged. Although theoretical equations exist for the case of idealized static wave setup, the actual static setup value depends on a number of factors, including wave nonlinearity, wave breaking characteristics, profile slope, and wave propagation through vegetation.

However, oscillations in the wave setup will also occur in nature, and this oscillation is known as "dynamic" wave setup (see Figure D.2.6-2). These oscillations will typically occur with periods of 10 to 20 times the mean wave period. The dynamic wave setup increases with narrow frequency spectra and narrow directional spectra, both uncharacteristic of hurricane and nor'easter conditions. Therefore, the dynamic setup component is considered to be small by comparison with the static component for the Atlantic and Gulf applications, and should not be included at present in the calculations for the Atlantic and Gulf storm surges.

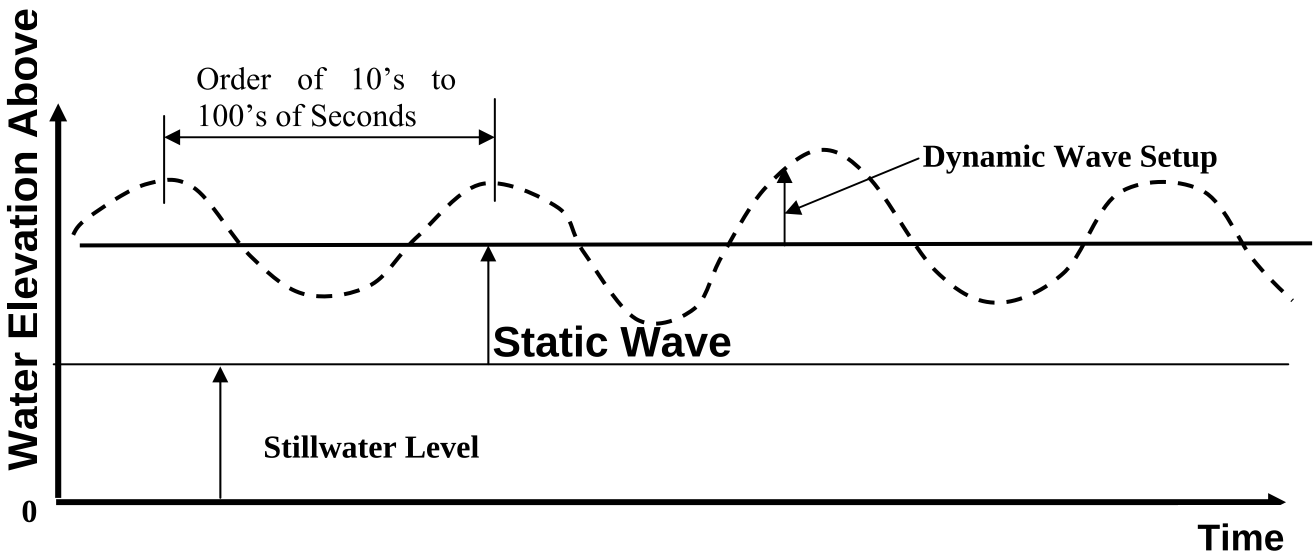

Figure D.2.6-2. Definitions of Static and Dynamic Wave Setup Components.

D.2.6.2 Wave Setup Implications for Flood Insurance Studies

Wave setup can be a significant contributor to the total water level landward of the +/- MSL shoreline and should be included in the determination of coastal BFEs. The manner in which it is included, however, is critical to the accuracy of the BFEs. There are two ways of estimating stillwater levels for use in an FIS. One involves separate calculations of storm surge and wave setup, and one computes storm surge and wave setup concurrently. Recall that the stillwater level comprised of the combination of these two components is the *mean water level* (MWL).

In the first case, wave setup must be added to the storm surge stillwater level for WHAFIS calculations (see Section 2.5), but *not* added to the storm surge stillwater level for wave runup calculations (wave runup models typically include wave setup effects in the computed wave runup heights) or for dune erosion removal/retreat (see Subsection 2.9.3.1).

In the second case, the surge and wave setup components may have to be decoupled before wave runup calculations and dune removal/retreat calculations can be made (to avoid double counting wave setup). This will require the Mapping Partner to make separate wave setup calculations, and to subtract the calculated wave setup from the combined stillwater elevation (MWL) before using RUNUP 2.0 (or most other wave runup procedures) or before estimating the frontal dune reservoir. WHAFIS calculations can proceed with the combined storm surge and wave setup stillwater level (MWL), but the wave setup value should not be input separately into WHAFIS, even if it is known.

Wave setup and its treatment in an FIS must be carefully documented by the Mapping Partner, and any questions over how to handle wave setup should be discussed with the FEMA Study Representative.

D.2.6.3 Guidelines for Estimating Static Wave Setup

There are several methods for establishing static wave setup. One method uses the results described in the USACE Shore Protection Manual (SPM), which present normalized wave setup as a function of bottom slope and the deepwater wave steepness (H_o/L_o), as shown in Figure D.2.6-3 (Note the symbol *S* for static wave setup in Figure D.2.6-3 will be replaced by $\overline{\eta}$ here). Other methods include those developed by Goda (2000) and the Direct Integration Method (DIM), an integration of the governing equations. DIM was developed in conjunction with the recent FEMA-sponsored development of the Pacific Coast *Guidelines* (FEMA 2004). The first two methods yield a computation of wave setup at the landward limit of flooding, while the latter (DIM) yields wave setup estimates at any point along a shore-normal transect.

A comparison analysis of these three methods was conducted by the Pacific *Guidelines* working group (TWG). TWG found that the DIM methodology yielded static wave setup values ranging from 60 to 100 percent larger than those from the SPM method. However, the DIM methodology values were less than 16 percent greater than those predicted by Goda. It was concluded by TWG that the DIM methodology provides a better estimate of wave setup than the SPM methodology.

The Mapping Partner should use the DIM methodology to determine static wave setup. A reduction of up to 16 percent (based on the comparison with the Goda methodology) may be applied to the DIM results if evidence[3] suggests a reduction is appropriate.

[3] Evidence that indicates a reduction is appropriate can include measured water level data during previous severe storms affecting the study area.

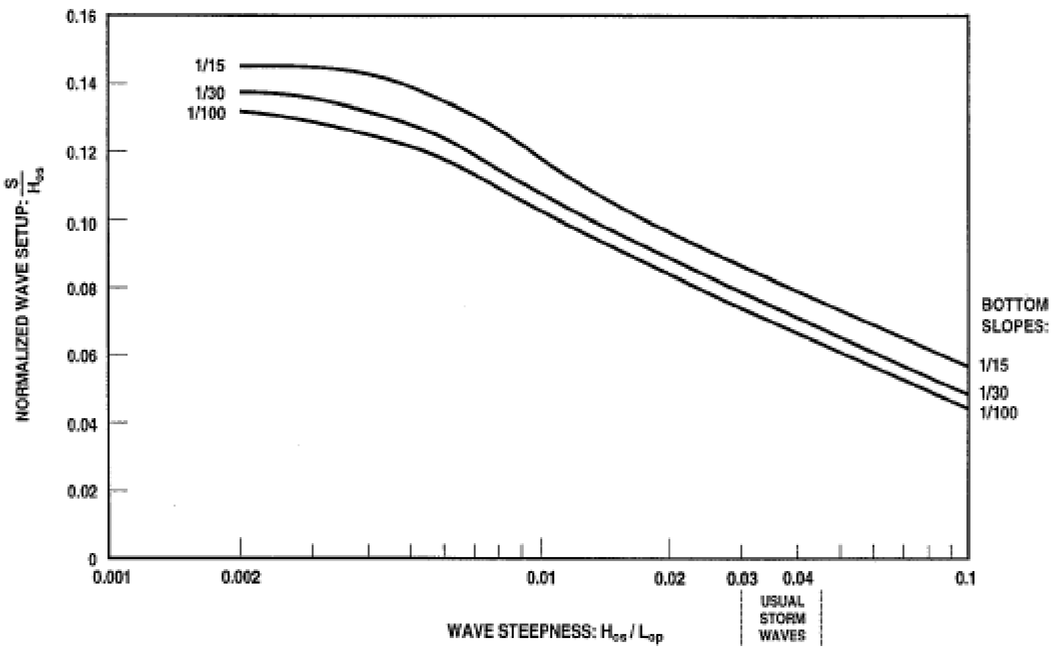

Figure D.2.6-3. Methodology for Calculating Wave Setup (from USACE SPM).

The DIM methodology can be written as follows for the static wave setup ($\overline{\eta}$) which allows direct calculation of the effect of profile slope (m) and deepwater wave steepness (H_o/L_o).

$$\overline{\eta} / H_o^{'} = 0.160 \frac{m^{0.2}}{(H_o^{'} / L_o)^{0.2}} \tag{D.2.6-1}$$

Note that the SPM and Goda methods provide the wave setup at the landward limit of flooding, thus, in some cases a method might be required to determine the wave setup value at the normal (+/- MSL) shoreline for later transect applications. It is recommended that the Mapping Partner proportion the maximum wave setup as determined by the SPM or Goda method to determine the approximate wave setup at the normal shoreline. Denoting the wave setup at the shoreline as $\overline{\eta_o}$ and the maximum setup as $\overline{\eta_{\max}}$, $\overline{\eta_o}$ can be approximated as

$$\overline{\eta_o} = \left[1 - \frac{3\kappa^2}{8} \frac{1}{\left(1 + \frac{3\kappa^2}{8}\right)} \right] \overline{\eta_{\max}} \tag{D.2.6-2a}$$

which simplifies to

$$\overline{\eta_0} = \left[\frac{8}{\left(8 + 3\kappa^2\right)} \right] \overline{\eta_{\max}} \tag{D.2.6-2b}$$

where κ is the ratio of breaking wave height to breaking water depth. For the case of significant wave height and non-vegetated slopes, typical values of κ range from 0.4 to 0.6.[4] These values result in

$$\overline{\eta_o} = 0.88 \text{ to } 0.94\ \overline{\eta_{\max}} \approx 0.9\ \overline{\eta_{\max}}$$

(D.2.6-2c)

Procedures for calculating wave setup on an open coast will be presented, followed by cases of setup on levees, which entail modifications to the open coast method. As seen in Equation (D.2.6-1), wave setup calculations require a reference wave height. In this case, the effective deepwater significant wave height is H'_o.

D.2.6.3.1 Wave Setup on an Open Coast

D.2.6.3.1.1 Determining a Reference Deepwater Significant Wave Height

Estimation of the static wave setup requires an estimate of the deepwater significant wave height, which can be calculated or determined from hindcast data (such as that provided by the USACE Coastal and Hydraulics Laboratory WIS or other sources). WIS modeling stations are located continuously along the Atlantic and Gulf coasts.

Because there are two primary statistical approaches for estimating storm surge elevations (JPM and EST), two approaches are recommended to determine a reference deepwater wave height. The JPM methodology requires the development of synthetic storms in accordance with the historical database. For hurricanes, this involves calculating storm surges and waves based on a large number of synthetic storms. For nor'easters, the database may be better suited to the EST method or the use of a wave hindcast method based on the windfields used to generate the storm surge.

D.2.6.3.1.2 JPM—Wave Setup Due to Hurricanes

The SPM provides recommendations for calculating the deepwater wave characteristics associated with a hurricane. These methods included two equations, one for the maximum significant wave height and one for the associated wave period. In addition, a graph was provided that represents the nondimensional distribution of significant deepwater wave heights in a hurricane. Each of these is discussed below.

The wave characteristics (significant height and associated period) are presented in the SPM in terms of the hurricane parameters in both English and metric systems. The equations below are presented for the English system. The parameters are:

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[4] The values of κ cited here assume wave setup is due to wave breaking only (i.e., no reduction in wave setup due to vegetation – see Sec. D.2.6.3.4.1) and waves are passing over a sloping surface without significant changes in slope. If the ground surface along the transect changes slope suddenly (e.g., a bluff or levee landward of a marsh) then the Mapping Partner may consider breaking the wave setup analysis into segments and calculating a different κ for each segment.

- Central pressure deficit: Δp in inches of mercury
- Forward translational speed of hurricane: V_F in knots
- Radius to maximum winds: R in nautical miles
- Maximum sustained windspeed at 33 feet above the sea surface: U_R in knots
- Coefficient depending on hurricane speed: α (dimensionless)
- Coriolis parameter: f (dimensionless)

where the Coriolis parameter, f, is given by

$$f = 0.524 \sin \phi \tag{D.2.6-3}$$

and ϕ is the latitude at the location of interest

The equations for maximum significant wave height and associated period are:

$$H'_{o,\max} = 16.5 e^{\frac{R \Delta p}{100}} \left[1 + \frac{0.208 \alpha V_F}{\sqrt{U_R}} \right] \tag{D.2.6-4}$$

and

$$T_s = 8.6 e^{\frac{R \Delta p}{200}} \left[1 + \frac{0.104 \alpha V_F}{\sqrt{U_R}} \right] \tag{D.2.6-5}$$

where

$$U_{\max} = 0.868\left(73\sqrt{\Delta p} - 0.575Rf\right) \tag{D.2.6-6}$$

The parameter U_R, is expressed in terms of U_{max} as:

$$U_R = 0.865U_{\max} + 0.5V_F \tag{D.2.6-7}$$

The value of the parameter α is recommended as unity (one) for slowly translating hurricanes, and this value is recommended for use here.

Figure D.2.6-4 presents the relationship for nondimensional significant wave height as a function of nondimensional distances relative to the hurricane center. The distances are made nondimensional by the hurricane radius to maximum winds (R).

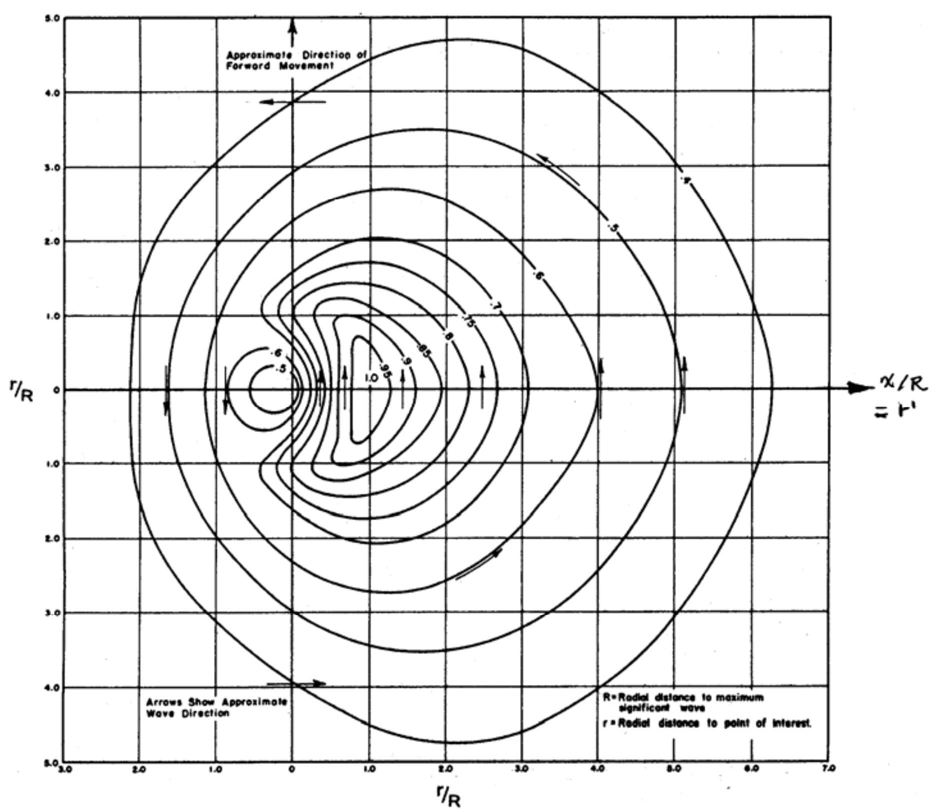


Figure D.2.6-4. SPM Relationship for Wave Heights Relative to Their Maximum in a Hurricane (USACE).

As shown in Figure D.2.6-4, the SPM model predicts waves that propagate in approximately the same direction as the local winds. For these purposes, wave height distributions are presented for two distances offshore, and it is recommended that the applied distribution be prorated by the actual distance of the hurricane center from the shoreline. The two distributions are presented in Figure D.2.6-5, along with the SPM distribution. The deviations from the SPM model are based on the recognition that waves diffract and disperse in advance of a hurricane. The two distributions are associated with the following positions: (1) distances of more than 4 radii from the shoreline, and (2) at the shoreline. Specifically, the recommended relevant deepwater wave heights at shore are:

Hurricane Center More Than 4 Radii (R) From the Shoreline

$$H_o / H_{o,\max} = 0.40 + 0.20\cos^2\left[\frac{\pi}{2}\left(\frac{r'-2}{12}\right)\right], \; -10 < r' < 14$$
$$H_o / H_{o,\max} = 0.40, \; r' < -10, \; r' > 14 \qquad \text{(D.2.6-8)}$$

Hurricane Center at the Shoreline

$$H_o / H_{o,\max} = 0.3, \quad r' < -3.0$$
$$H_o / H_{o,\max} = 0.3 + 0.233(r'+3), \quad -3.0 < r' < 0$$
$$H_o / H_{o,\max} = 1.0, \quad 0 < r' < 1.0$$
$$H_o / H_{o,\max} = 1.0 - 0.10(r'-1), \quad 1 < r' < 8$$
$$H_o / H_{o,\max} = 0.3, \quad r' > 6 \qquad \text{(D.2.6-9)}$$

and $r' = x / R$.

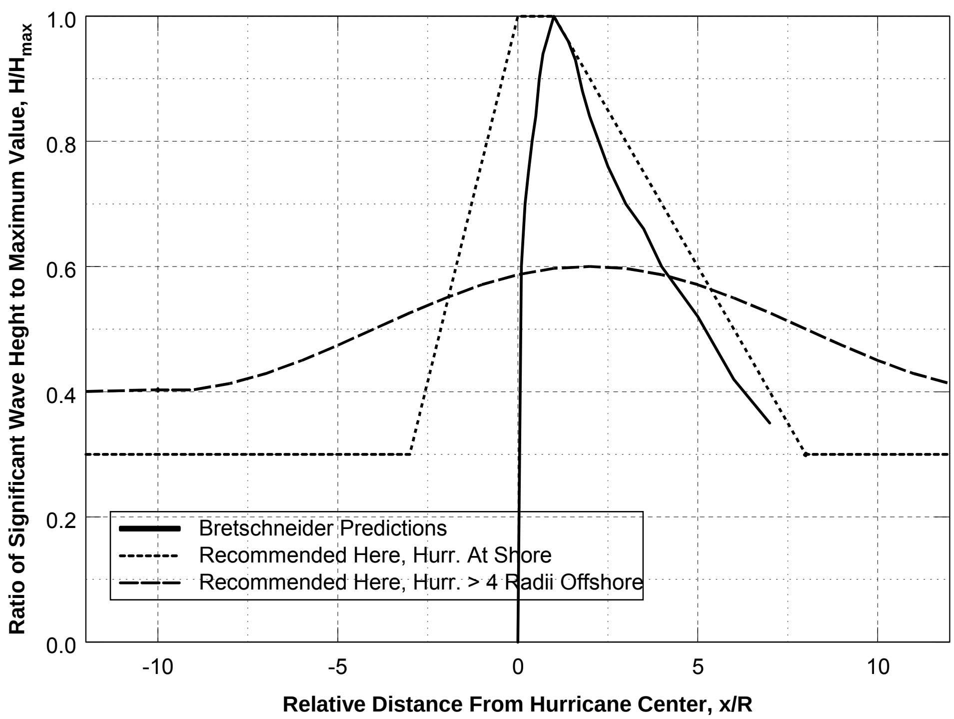


Figure D.2.6-5. Recommended Relative Wave Height Along a Line Perpendicular to Hurricane Translation Direction

With the maximum significant wave height and associated period known along a line perpendicular to the hurricane translation direction, the wave height at any location can be determined from the approximate graphical relationship in Figure D.2.6-5 or Equations (D.2.6-8) and (D.2.6-9), which present local significant deepwater wave height relative to the global maximum deepwater significant wave height. The recommended wave period at all locations is that given by Equation (D.2.6-5).

With the effective deepwater wave height and period, the effective profile slope (m) can be based on the average slope out to the breaking depth, which may be approximated by H'_o, and the static wave setup calculated by Equation (D.2.6-1). This completes the recommendations for applying the JPM to calculate wave setup for hurricanes on an open coast.

D.2.6.3.1.3 EST - Wave Setup Due to Nor'easters

As noted, the database for nor'easters may be better suited for applying the EST method. In this case, it is appropriate to determine a field of reference deepwater wave heights based on hindcasts using the windfield applied to calculate wind surge. The Mapping Partner may consider both 1-D and 2-D methodologies for calculating wave characteristics.

The method for determining a deepwater wave height in cases where the EST method is used to calculate wind surges differs only slightly from that of the JPM method. The difference is that historical storms, rather than synthetic storms, are used in the EST methodology. The general approach is to estimate the necessary parameters Δp, R, $V_{F,}$ *ect* for each of the historical storms and then to apply the procedures presented for the JPM method to calculate static wave setup.

The forward velocity (V_F) is determined from the path characteristics used in the simulation, so only the central pressure deficit (Δp) and the radius to maximum winds (R) need to be determined. The subsections below describe one approach to determine these variables. The Mapping Partner may evaluate other approaches.

D.2.6.3.1.3 Radius to Maximum Winds (*R*)

It is recommended that the radius to maximum winds (R) be determined from inspecting the historical windfield.

D.2.6.3.1.4 Central Pressure Deficit (Δp)

The central pressure deficit (Δp) can be related approximately to the maximum wind (U_{max}) in the windfield used in Equation (D.2.6-6), which is provided below in a different form:

$$\Delta p = 1.88x10^{-4}\left(\frac{U_{\max}}{0.868} + 0.575Rf\right)^2 \qquad \text{(D.2.6-10)}$$

With the above-referenced definitions and knowledge of the track of the hurricane, it is possible to apply the procedures described earlier for the JPM approach.

D.2.6.3.2 Wave Setup On a Coastal Structure

The following subsections address the case of wave setup on a coastal structure that could be overtopped. Figure D.2.6-6 presents the case of a nonovertopped levee.

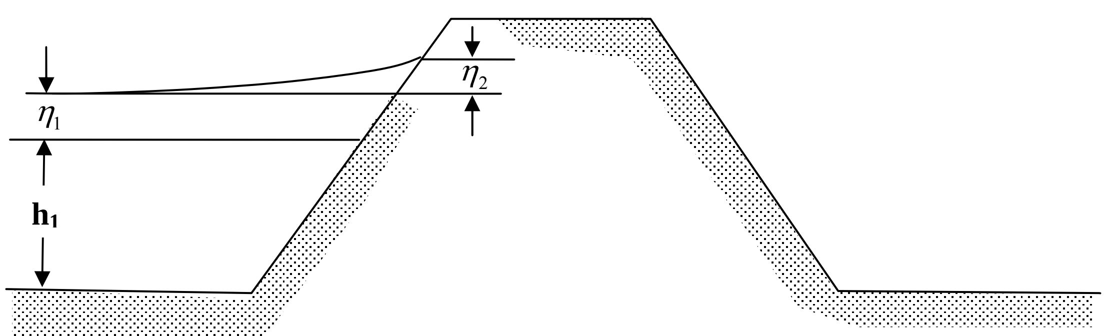


Figure D.2.6-6. Definition Sketch for Nonovertopped Levee

Because of the steep slopes associated with coastal structures such as levees, seawalls, and revetments, the wave setup is greater over this portion of the profile and must be treated separately. Referring to Figure 4, the setup must be considered in two components. The first setup component (η_1) is the water depth, h_1, determined at the toe of the levee, and the second setup component (η_2) is determined for the sloping structure. In order to quantify η_1, the breaking wave height and depth must be determined.

D.2.6.3.2.1 Determining the Breaking Wave Height and Water Depth

It can be shown that the nondimensional breaking wave height (H_o/L_o) is a function of the deepwater wave steepness (H'_o/L_o), as shown in Figure D.2.6-7.

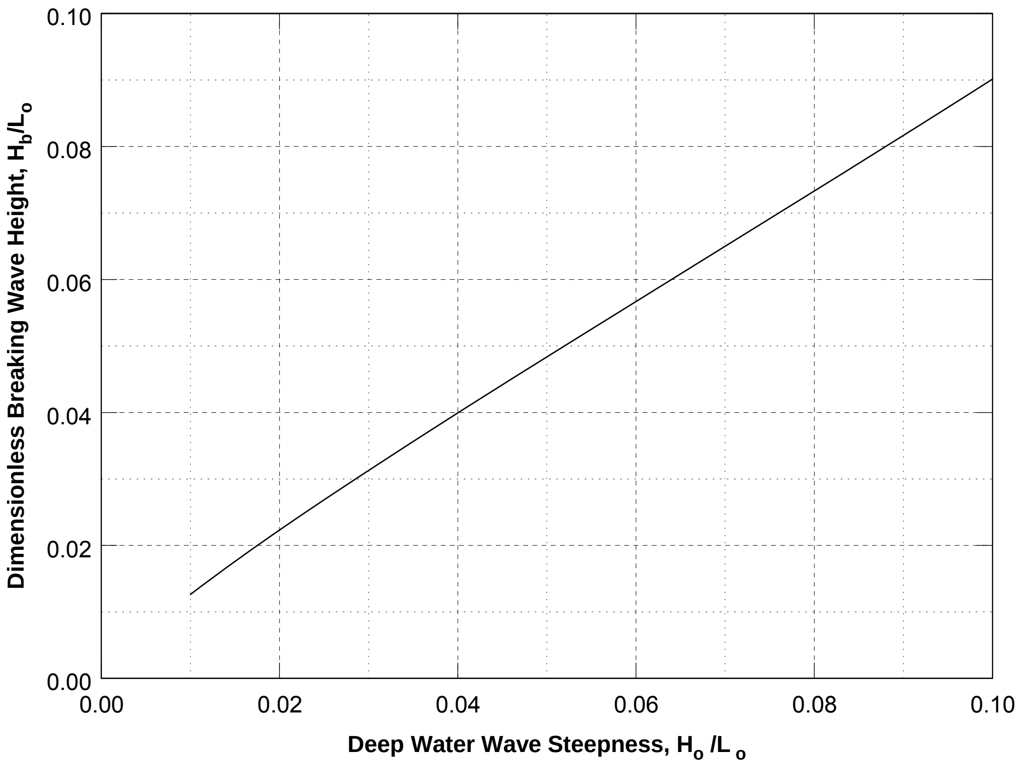


Figure D.2.6-7. Dimensionless Breaking Wave Height vs. Deepwater Wave Steepness

The nondimensional breaking wave height and depth associated with the maximum local waves are based on the deepwater wave steepness (H'_o/L_o), where L_o=5.127^2 in the English system of units being used here. The breaking wave height differs from the deepwater wave height by ±10 percent at most, over the range plotted in Figure D.2.6-7. Figure D.2.6-8 presents the dimensionless breaking water depth (h_o/ L_o), which will be useful later.

D.2.6.3.2.2 Nonovertopped Structure

The wave setup at depth $h_1$is determined by referring to Figure D.2.6-9, which presents the proportion of wave setup that would occur in any depth proportional to the breaking depth (the latter determined from Figure D.2.6-8). The value of η_2 is determined as

$$\eta_2 = 0.15(h_1 + \eta_1) \tag{D.2.6-11}$$

and the total wave setup is $\eta_T = \eta_1 + \eta_2$.

Later examples will illustrate the application of these methods.

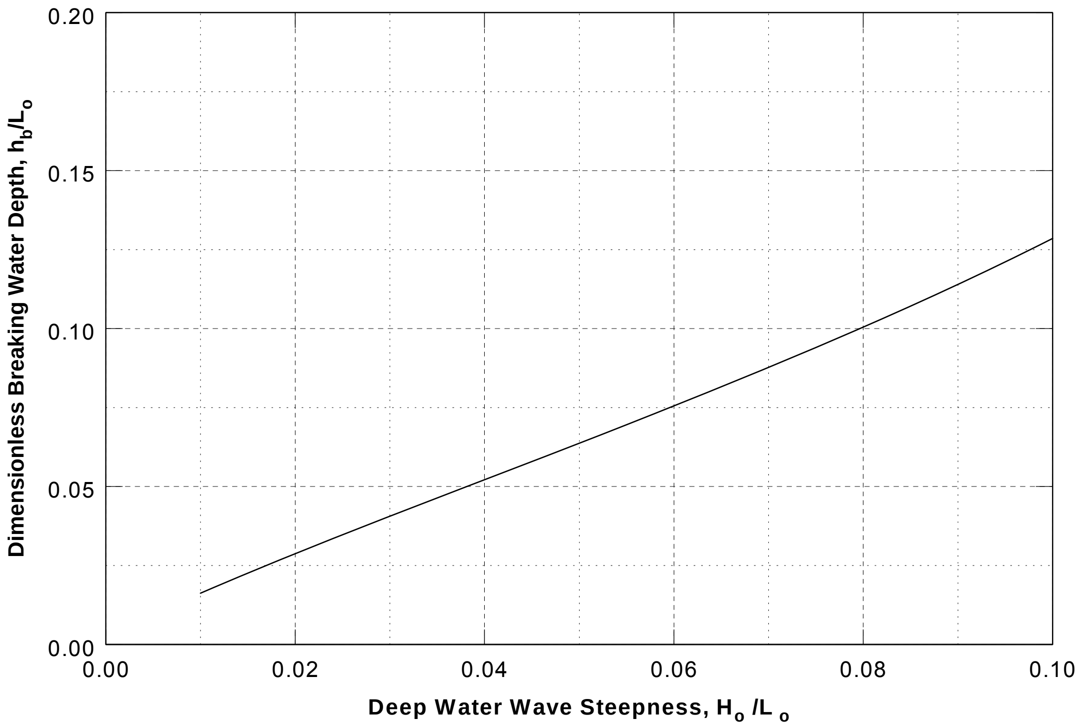


Figure D.2.6-8. Dimensionless Breaking Water Depth vs. Deepwater Wave Steepness.

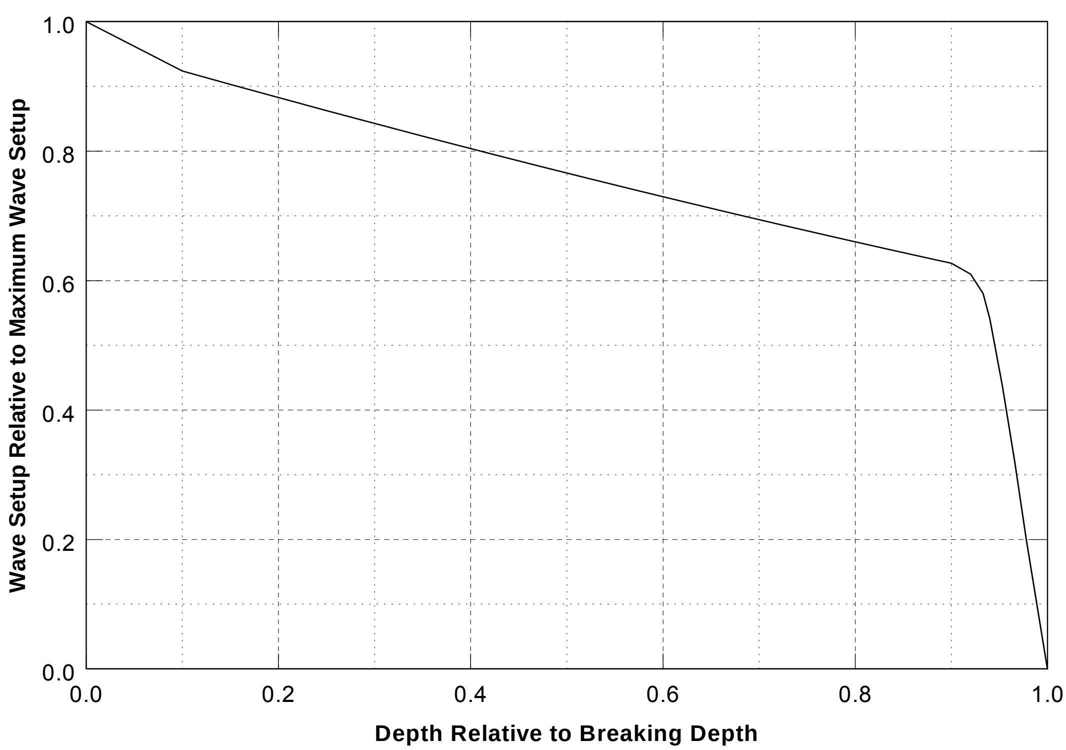


Figure D.2.6-9. Proportion of Maximum Wave Setup that Has Occurred vs. a Proportion of the Breaking Depth.

D.2.6.3.2.3 Overtopped Structure

For overtopped structures, the water depth (including the calculated storm surge) on top of the structure is denoted h_2. The recommended additional wave setup (η_2) for overtopped structures is:

$$\eta_2 = 0.15(h_1 + \eta_1)\left[1 - \left(\frac{h_2}{h_1}\right)^2\right] \qquad \text{(D.2.6-12)}$$

and, as before, $\eta_T = \eta_1 + \eta_2$.

D.2.6.3.3 Examples Illustrating Application of the Methodology

The following three examples illustrate the application of the methodology. The hurricane parameters are presented in Table D.2.6-1. For each of the examples, we will calculate the wave setup at three locations: $x/R = r'$ = -1.5, 1.0, 4.0. For all three examples, we consider a location where the latitude is 30^{o} and the effective profile slope is 0.01.

Table D.2.6-1 Hurricane Characteristics Considered in Examples

Example	Situation	Δp (in Hg)	R (n mi)	V_F (knots)	Hurricane Location Relative to Shoreline
1	Wave Setup on an Open Coast	2.5	40	12.0	At Shoreline
2	Wave Setup on a Nonovertopped Structure	3.0	20	14.0	40 n mi Seaward
3	Wave Setup on an Overtopped Structure	3.0	20	14.0	40 n mi Seaward

D.2.6.3.3.1 Example 1: Wave Setup On An Open Coast

For this case, the maximum significant deepwater wave height and period are determined from Equations (D.2.6-2) and (D.2.6-3) as: $H_{o,max}$= 56.0 ft and T_s = 15.9 sec.

For values of r' = -1.5, 1.0, and 4.0, the corresponding ratios of wave heights to the maximum are Equation (D.2.6-8): $H_o/H_{o,max}$ = 0.65, 1.0, and 0.70. Thus, the associated wave heights are 36.4 feet, 56.0 feet, and 39.2 feet, respectively. As noted, the appropriate period is determined from Equation 3 and the deepwater wave length, $L_o = 5.12T^2$ = 1,294 ft.

The wave setup values at the three shoreline locations of interest are determined from Equation (D.2.6-1) for the relevant deepwater wave steepnesses and a representative profile slope of 0.01, and are as presented in Table D.2.6-2.

Table D.2.6-2 Wave Characteristics and Setup at Three Locations for Example 1

Value of $r'(=x/R)$	H_o (ft)	$\bar{\eta}$ (ft)
-1.5	36.4	4.7
1.0	56.0	6.7
4.0	39.2	5.0

D.2.6.3.3.2 Example 2: Wave Setup at a Nonovertopped Structure

For this example, we consider that the water depth at the structure toe is 6 feet and that the structure is not overtopped. For hurricane conditions, the maximum wave height and its associated period are 38.3 feet and 13.2 seconds. The deepwater wave length is

L_o =5.12 T^2 = 892 feet. The breaking relative water depth is determined approximately from Figure 4 as h_o/L_o = 0.052. Thus the breaking depth is 46.4 feet. The ratio of h_1/h_b = 0.129, and Figure D.2.6-9 shows that approximately 91 percent of the maximum wave setup that would have occurred on an open coast has occurred at this water depth of 6 feet. Because the hurricane center is located at two times the radius to maximum winds from the shoreline, the wave height is determined as a prorated value of the two recommended relationships in Figure D.2.6-5 and Equations (D.2.6-8) and (D.2.6-9). The ratios of wave height to maximum wave height for the three longshore distances relative to the center of the hurricane are $H_o/H_{o,max}$ = 0.61, 0.80, and 0.65.

The total wave setup values if the structure were not present are shown in Column 3 of Table D.2.6-3. These values are reduced by a factor of 0.91 and tabulated in Column 4. Finally, the wave setup (η_2) as the waves propagate up on the structure is determined from Equation (D.2.6-11) and is presented in Column 5. The total wave setup at the structure (η_T), which is the sum of Columns 4 and 5, is shown in Column 6 in Table D.2.6-3.

Table D.2.6-3 Wave Characteristics and Setup at Three Locations for Example 2

Value of $r'(=x/R)$	H_o (ft)	$\bar{\eta}_{max}$ (ft)	$\bar{\eta}_1$ (ft)	$\bar{\eta}_2$ (ft)	$\bar{\eta}_T$ (ft)
-1.5	23.4	3.1	2.8	1.3	4.1
1.0	30.6	3.8	3.5	1.4	4.9
4.0	24.9	3.2	3.0	1.4	4.4

D.2.6.3.3.3 Example 3: Wave Setup at an Overtopped Structure

For this example, we consider that the water depth at the structure toe is 6 feet, as in Example 2; however, the structure is overtopped and has a crest elevation of 4 feet, relative to the adjacent ground. Because the hurricane conditions for Examples 2 and 3 are the same, the wave heights and periods are the same: 38.3 feet and 13.2 seconds. The setup on the structure is reduced in accordance with Equation (D.2.6-11), which reduces the additional setup values (η_2) as tabulated in Column 5 of Table D.2.6-4. In this case, the overtopping only reduces the total wave setup by approximately 3 percent. The total wave setup values are presented in Column 6 of Table 4.

Table D.2.6-4 Wave Characteristics and Setup at Three Locations for Example 3

Value of $r'(=x/R)$	H_o (ft)	$\overline{\eta}_{max}$ (ft)	$\overline{\eta}_1$ (ft)	$\overline{\eta}_2$ (ft)	$\overline{\eta}_T$ (ft)
-1.5	23.4	3.1	2.8	1.2	4.0
1.0	30.6	3.8	3.5	1.2	4.7
4.0	24.9	3.2	3.0	1.2	4.2

D.2.6.3.4 Wave Setup—Special Cases

D.2.6.3.4.1 Vegetation and Bottom Friction Effects

The methodology above represents approaches to calculating static wave setup on an open coast and on coastal levees (nonovertopped and overtopped). The methods do not account for wave setup effects caused by nonlinear waves or wave energy losses caused by bottom friction or waves propagating through vegetation. If the Mapping Partner deems these effects to be significant, Dean and Bender (2006) should be consulted. As an interim, simplified approach, results from Dean and Bender (2006) show that the incremental wave setup associated with wave energy dissipation through vegetated areas or over dissipative bottoms can be approximated as one-third of the wave setup that would occur if the energy dissipation were caused by wave breaking. Thus, depending on the height and density of vegetation, or the nature of the dissipative bottom, the Mapping Partner may reduce the otherwise calculated wave setup by up to two-thirds.

As a preliminary rule of thumb for the vegetation case, if extensive, dense stands of vegetation extend near or above the base flood wave crest elevation, the two-thirds reduction might be appropriate; if extensive, dense stands of vegetation extend to the approximate base flood mean water elevation, a one-third reduction might be appropriate; if extensive, dense vegetation does not extend above the mid-depth of mean water level, no reduction for vegetation should be used.

D.2.6.3.4.2 Wave Setup across Barriers Islands and Large Bays

There may be instances where wave setup calculations along a specific transect are complicated by the topography along the transect and possibly by 2-dimensional effects. For example:

- Case 1: storm surge and waves propagate over a low-lying or eroded barrier island, across a small bay, and onto the mainland
- Case 2: storm surge and waves propagate over a barrier island, and across a large bay or sound that separates the offshore barrier from the mainland

If, in the first case, storm surge inundates the entire barrier island or a large portion of the island, waves will pass over the island, possibly regenerate across the bay and propagate onto the mainland. Wave setup in this case will rise as the overtopped barrier is approached, then will remain roughly constant across the bay, and will increase again as the waves break on the mainland. The wave setup on the mainland may be higher than it would have been on a non-overtopped portion of the barrier, due to wave regeneration across the bay.

If, in the first case, only a small portion of the barrier is overtopped by surge and waves, wave setup calculations along a transect through the overtopped section may overstate the wave setup on the mainland. The wave setup that passes across the overtopped section may be drained laterally into regions of the bay where no wave setup crosses the island. Two-dimensional effects should be considered in this case.

The second case (large bay) may be similar to the partially overtopped barrier case, where two-dimensional effects come into play. The volume of water that is required to "fill" the potential wave setup across the large bay can be approximated as the average bay width times the bay length times the average wave setup height. This volume must be supplied by flow across the barrier or by other means (e.g., rainfall across the bay and freshwater discharge into the bay) or the wave setup height will not be realized across the entire bay. The Mapping Partner should evaluate the various factors that may limit wave setup in this case, including the fraction of the barrier that is overtopped, the bay dimensions, the duration of the storm surge hydrograph above the barrier elevation, rainfall and freshwater discharge, etc. If sufficient water is not available to "fill" the potential wave setup, the Mapping Partner should examine 2-dimensional effects across the bay and estimate wave setup along the mainland shoreline accordingly. Final wave setup calculations on the mainland will then be made.

D.2.6.3.4.3 Decay of Wave Setup across Flooded Lands

Some previous Flood Insurance Studies have been completed using the assumption that wave setup will decay in the inland direction at some prescribed rate (e.g., one foot of wave setup decay per 1,000 ft of inland flooding, or all wave setup will decay across the barrier island width, etc.). These rules of thumb should not be used. Absent the types of 2-dimensional effects described in the previous section, wave setup at the inland limit of flooding will be equal to or greater than the wave setup at the +/- MSL shoreline

D.2.7 Overland Wave Propagation

This subsection provides guidance for estimating wave heights and wave crest elevations on flooded land areas. FEMA's WHAFIS model is described.

D.2.7.1 Overview

The fundamental analysis of overland wave effects for an FIS is provided by the WHAFIS 3.0 program, a DOS-based program that uses representative transects to compute wave crest elevations in a given study area. Transects must be specified by the Mapping Partner, who must also identify topographic, vegetative, and cultural features along each transect landward of the shoreline. WHAFIS uses this and other input information to calculate wave heights, wave crest elevations, flood insurance risk zone designations, and flood zone boundaries along the transects (FEMA, 1988). The Mapping Partner can specify an incident wave height, or WHAFIS can compute an incident wave height at the seaward end of each transect. Please note that the WHAFIS-calculated incident wave height is based on the fetch provided by the Mapping Partner and does not take into account refraction, diffraction, or bottom dissipation effects. The Mapping Partner should perform separate wave transformation calculations if these effects will cause the incident wave height to depart markedly from the value generated by WHAFIS. The Mapping Partner should consult FEMA's approved wave model list at http://www.fema.gov/plan/prevent/fhm/en_coast.shtm if additional wave studies are required.

The original basis for the WHAFIS model was the 1977 NAS report *Methodology for Calculating Wave Action Effects Associated with Storm Surges*. The NAS methodology accounted for varying fetch lengths, barriers to wave transmission, and the regeneration of waves over flooded land areas. Because the incorporation of the NAS methodology into the initial version of WHAFIS, periodic upgrades have been made to WHAFIS to incorporate improved or additional wave considerations. Figure D.2.7-1 illustrates the basic factors that WHAFIS considers in its overland wave height and wave crest elevation calculations.

The current WHAFIS model is fully documented (*Technical Documentation for WHAFIS Program Version 3.0*, FEMA, September 1988). Briefly, the wave action conservation equation governs wave regeneration caused by wind and wave dissipation by marsh plants in the model. This equation is supplemented by the conservation of waves equation, which expresses the spatial variation of the wave period at the peak of the wave spectrum. The wave energy (equivalently, wave height) and wave period respond to changes in wind conditions, water depths, and obstructions as a wave propagates. These equations are solved as a function of distance along the wave analysis transect.

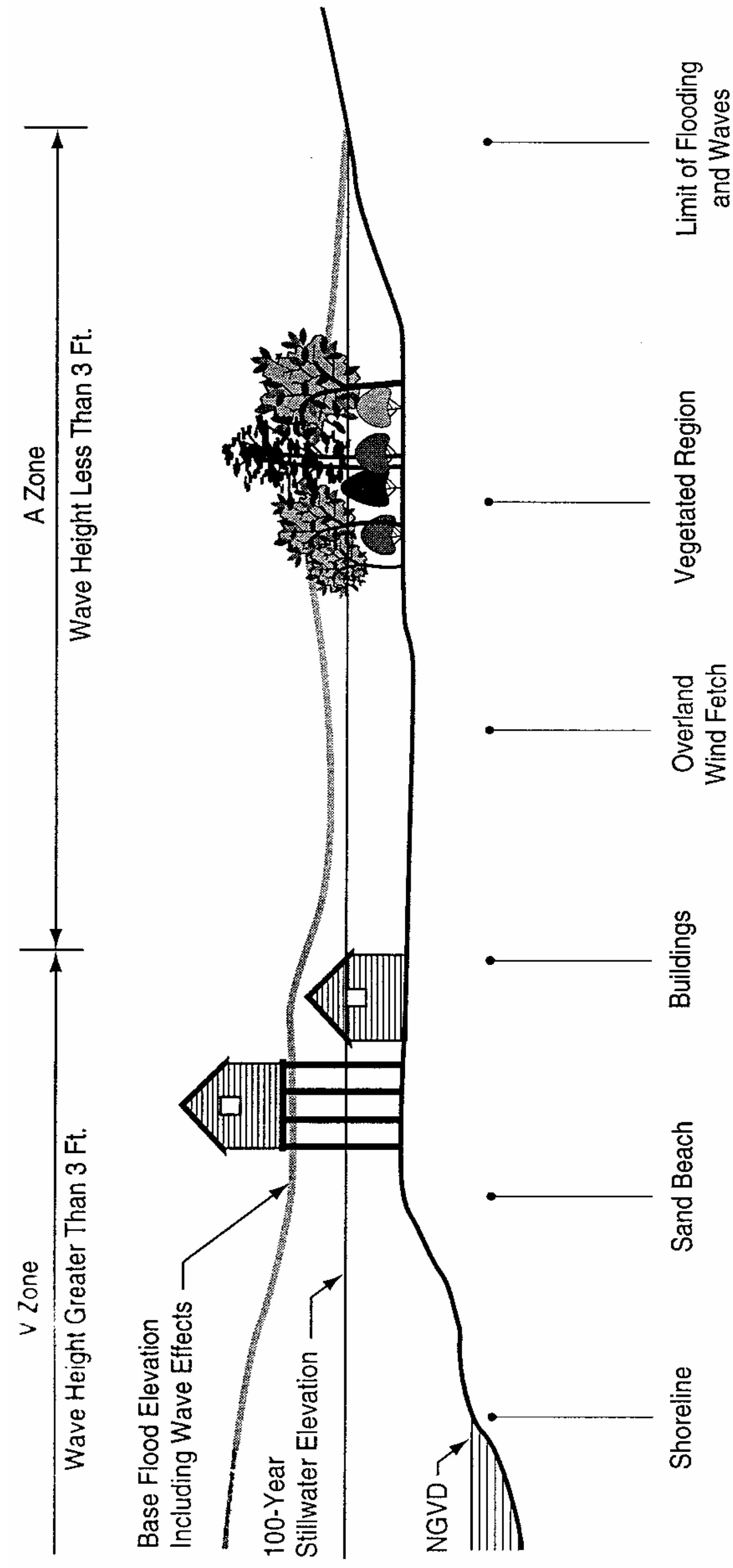


Figure D.2.7-1. Wave Height and Wave Crest Variations Along a WHAFIS Transect

A fundamental element in this wave treatment remains unchanged from the NAS methodology: the controlling wave height[5] (approximately, the average height of the highest 1 percent of waves during storm conditions) is limited to 78 percent of the local stillwater level depth. Also, the model assumes that 70 percent of the controlling wave height lies above the SWEL, resulting in the wave crest elevation being 0.55 times the local stillwater depth above the SWEL, or 1.55 times the local stillwater depth above the ground elevation (see Figure D.2.7-2).

The WHAFIS program is available as a stand-alone program, or as a part of FEMA's Coastal Hazard Analysis Modeling Program (CHAMP). CHAMP is a Windows-interfaced Visual Basic program that allows the user to enter data, perform coastal engineering analyses, view and tabulate results, and chart summary information for each representative transect along a coastline, within a user-friendly graphical interface. With CHAMP, the user can import digital elevation data; perform storm-induced erosion treatments, wave height analyses, and wave runup analyses; plot summary graphics of the results; and create summary tables and reports in a single environment.

- WHAFIS 3.0 is available at http://www.fema.gov/plan/prevent/fhm/dl_wfis3.shtm.
- CHAMP 1.2 is available at http://www.fema.gov/plan/prevent/fhm/dl_champ.shtm.

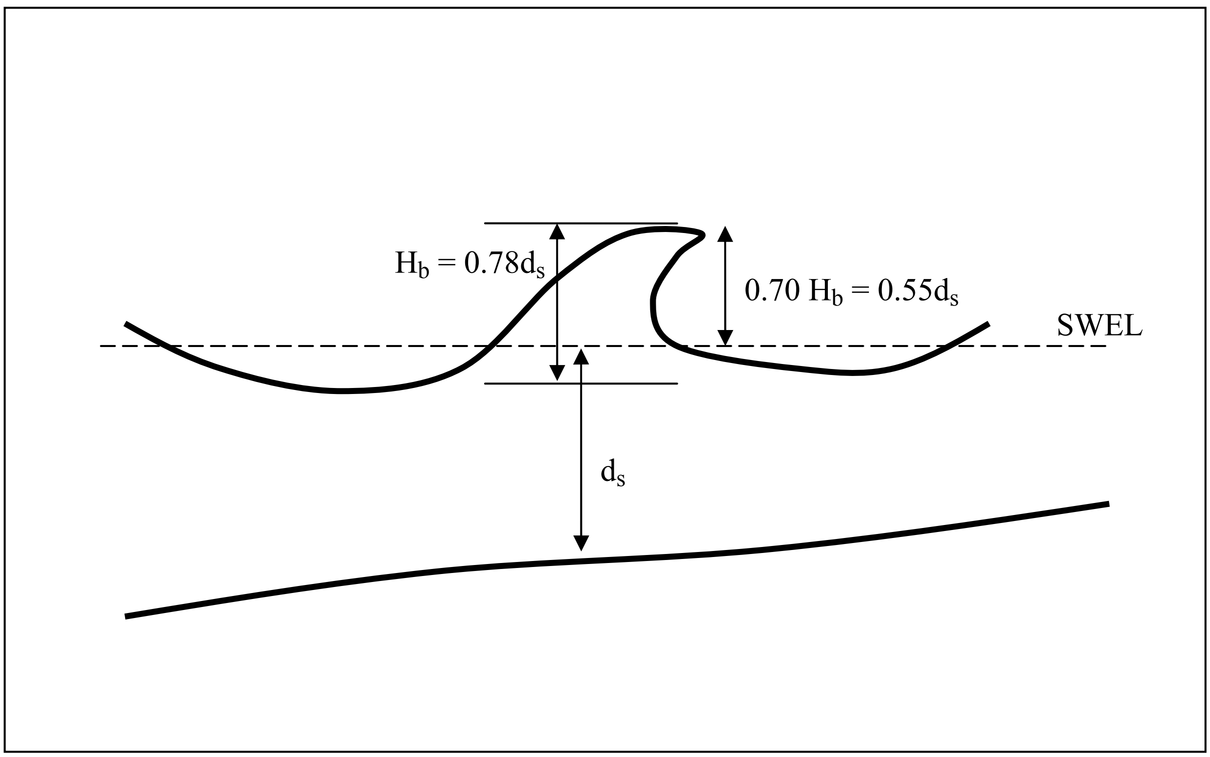


Figure D.2.7-2 WHAFIS relationships between local stillwater depth, d_s, maximum breaking wave height, H_b, and wave crest elevation.

[5] For NFIP purposes, the controlling wave height is taken to be 1.6 times the significant wave height.

D.2.7.2 WHAFIS Transect Considerations

The WHAFIS model considers the study area by representative transects. For accurate WHAFIS results, transects must be representative of major topographic, vegetative, and cultural features. Highly variable upland areas will require more closely spaced transects than areas where features are uniform. Closer spacing of transects may be also desirable along uniform upland areas, to reduce potential problems associated with the interpolation of flood insurance risk zones and BFEs between transects. However, Mapping Partners should be advised that spacing transects too closely may result in irregular gutters and an increased workload, without a significant increase in map quality. There are no set rules for transect spacing, but transects will usually be spaced from a few hundred feet apart (where upland characteristics are highly variable) to a few thousand feet apart (where uplands are uniform and development is sparse).

Transects should be located along any shoreline across which damaging waves may propagate during the base flood. This certainly includes all open-coast shorelines and other shorelines along large sheltered bodies of water subject to storm surge flooding (bays, sounds, and estuaries). However, damaging waves are not likely to accompany storm surge flooding along portions of small tributaries leading into large coastal bodies of water, particularly where those tributaries are narrow and winding and fetches are short. WHAFIS transects will not be required in these instances.

Transects should be oriented in the direction that waves propagate across the 0.0-ft NGVD29 or NAVD88 shoreline (from water to land) during the base flood. In most instances, this results in transects approximately perpendicular to the shoreline. However, in cases where the shoreline curves or has a highly variable shape (near tidal inlets or bay mouths, or on islands, or at the ends of peninsulas and spits), waves may approach at angles that deviate significantly from the perpendicular, and some transects may be required that are not shore-perpendicular. Another consequence of curved or irregular shorelines can be crossing transects. In general, specification of crossing transects should be minimized, but some crossings may be necessary to preserve the range of possible wave approach directions in the study area.

Some situations may arise where barrier islands are flooded during a severe storm, and transects can be drawn from the island's open-coast shoreline across a bay or sound and onto the mainland. If there is a large and/or unusually shaped embayment behind the island, it may be necessary to place additional transects just along the mainland shore. These transects may not be parallel to the transects originating at the island's open coast, and they may cross the longer, open-coast transects. The Mapping Partner may consider using multiple sets of transects (one set limited to the island, and one crossing the mainland shoreline) before the final transect selection is made.

The Mapping Partner shall also consider multiple flooding sources when specifying transects. For example, different transects may be required along different sides of a barrier island, if both the open coast and the back side of the island are subject to waves during a severe storm (high winds and waves may approach the island from different directions). This situation may require multiple specifications for water level and wave height, and multiple overland wave height analyses, with the flood map based on the more severe water level and wave conditions on land. Ultimately, transect specification requires a balance between representing coastal flood and severe wave conditions in developed upland areas (or other upland areas of interest) and study resources. In some cases, multiple analyses may be required and conducted; in other cases, a single analysis based on the dominant flood source and associated wave conditions may be performed.

D.2.7.3 WHAFIS Input Considerations

Another important consideration is the specification of input water level and wave conditions for each transect. On open coast Atlantic and Gulf shorelines, the typical procedure is to specify the base (1-percent-annual-chance) SWEL (including wave setup) and the controlling wave height (or the initial period of dominant waves) at the transect start. WHAFIS then computes an appropriate depth-limited wave height at the transect start. The only check necessary is to confirm that incident waves are likely to exceed that height and that a wave condition limited by water depth occurs.

On sheltered shorelines, the procedure is not as simple. The peak water level and peak wave conditions may not occur at the same time. For example, winds blowing across the longest fetch, which generate the highest wave heights at a particular shoreline, may also act to empty a water body or set down the water level. In such cases, the Mapping Partner may have to conduct several analyses, using different combinations of water levels and associated wave conditions, to determine the most severe upland flood conditions to be mapped. The Mapping Partner should keep in mind that on some sheltered shorelines the peak wave height may be smaller than the depth-limited height at the shoreline.

The Mapping Partner should also be aware that mapping flood hazards on an island or an upland area with multiple shorelines and flood sources may actually involve the mapping of a statistical flood surface, not a hydraulic surface representing a single flood event. This scenario is most likely where a barrier island is separated from the mainland by a bay or sound large enough to generate large waves against the back side of the island, and where flooding and waves can strike the island from two directions. A complete analysis of this scenario require the specification of transects, water levels and wave conditions at both shorelines, and multiple WHAFIS analyses. At any point on the island, the highest water surface and wave heights from the analyses would control the flood mapping.

Past practice in such cases has sometimes involved running a single set of transects across the island, starting at the side with the highest SWEL and most severe waves. The user then identified an area of transition between the different SWELs, with the higher SWEL extending inland to the highest point of the ground profile, after erosion considerations have been addressed. WHAFIS performed a linear interpolation within a transect segment where SWELs differed at the end stations. The interpolated elevations were compared to the ground elevations and adjusted, if necessary, to be above the ground elevations. Using this method, the Mapping Partner may have to input the SWEL a second time to identify areas of constant elevation and elevation transition.

Mapping Partners should note that the increasing use of modern hydrodynamic, storm surge, and wave models to provide input water level and wave conditions may complicate the specification of WHAFIS incident conditions at the shoreline and base flood SWELs along transects. Mapping partners shall consult with the FEMA Study Representative before using outputs from these models to specify WHAFIS inputs.

Once water level and wave conditions are determined and ground elevations along transects are input, natural and cultural features along the transects shall be specified.

- Vegetation: WHAFIS has two separate routines for vegetation: one for rigid vegetation that can be represented by an equivalent "stand" of equally spaced circular cylinders (NAS, 1977), and one for marsh vegetation that is flexible and oscillates with wave action (FEMA, 1984). For either type, the Mapping Partner shall exercise considerable care in selecting representative parameters and in ruling out the possibility that the vegetation will be intentionally removed or that effects would be markedly reduced during a storm through erosion, uprooting, or breakage. Details on coding vegetation are contained in Subsection D.2.7.3.2.

- Coastal Structures: The location, height, and extent of elongated manmade structures (seawalls, revetments, dikes, and levees, for instance) should be identified and shown as part of the ground profile, after each structure's stability under forces of the base flood is confirmed as discussed in Subsection D.2.10.

- Buildings: Buildings shall be specified on the transect as rows perpendicular to the transect. Because buildings are not always situated in perfect rows, the Mapping Partner shall exercise judgment to determine which buildings can be represented by a single row. The required input value for each row of buildings is the ratio of open space to total space. This is simply the sum of distances between buildings in a row, divided by the total length of that row. The Mapping Partner shall examine the first several rows of buildings along the shoreline to determine whether they will be obstructions during the base flood – only large, fully-engineered buildings with solid, nonbreakaway shearwalls, deep beams, or other horizontal structural elements extending below the BFE should be considered as obstructions. It is useful to contact local officials to obtain construction information and the lowest floor elevations of structures before coding buildings as obstructions. If buildings are elevated above the base flood wave crest on pilings, columns, or other open foundations, waves will propagate under the structures with minimal reduction in height. The mapping partner should code these buildings using the BU card (see Subsection D.2.7.3.1) and indicate 100-percent open space. This procedure acknowledges the presence of the pile-elevated buildings and allows others to see that the buildings were considered in the analysis, but recognizes that the presence of the open-foundation buildings will not lead to wave height reductions or flood insurance risk zone changes.

- Post-Storm Situations: Mapping Partners may encounter situations where many or all of the buildings and development in a study area have been destroyed during a storm. Mapping Partners must decide whether to run WHAFIS using existing (close to bare earth) conditions or with the assumption that most of the buildings and development will be replaced in a short period of time. Unless directed otherwise by the FEMA Study Representative, Mapping Partners shall code WHAFIS transects to the conditions that exist at the time of the study, and not in anticipation of future buildings and development in the study area. The Mapping Partner has no assurance of the exact nature or location of future buildings and development, so including them in WHAFIS is not appropriate.

WHAFIS allows the user to account for wave regeneration over flooded areas, using either the overwater fetch (OF) or inland fetch (IF) transect codes. WHAFIS uses an 80-mph sustained windspeed for OF calculations during the base flood, and a 60-mph sustained windspeed for IF calculations.

D.2.7.3.1 Input Coding for WHAFIS

After all the necessary input data have been identified on the transect, the Mapping Partner performing the study shall divide the transect into contiguous segments, each representing a continuous open fetch or a single obstruction. Fetches are flooded areas with no obstruction, while obstructions include dunes, manmade barriers, buildings, and vegetation. The Mapping Partner shall subdivide the fetches at points where the ground elevation changes abruptly and in the transition area of changing SWELs. The Mapping Partner shall subdivide obstructions into smaller segments at the transect's seaward edge to model the wave dissipation more accurately. Rigid vegetation shall have two to three seaward segments, extending 10 to 50 feet, and the first two or three rows of buildings shall have a segment for each row. Marsh vegetation will be subdivided within WHAFIS, so segmented input from the Mapping Partner is not necessary.

The Mapping Partner shall enter the necessary data using 11 line types, including the title line. The 10 remaining lines, each describing a certain type of fetch or obstruction, are listed as follows:

- The Initial Elevation (IE) line describes the initial overwater fetch and the initial SWELs.
- The IF and OF lines define the endpoint stationing and the elevation of inland and overwater fetches, respectively.
- Obstructions are categorized either as buildings (BU line), rigid vegetation (VE line), marsh vegetation (VH and MG lines), dunes or other natural or manmade elongated barriers (DU line), or areas where the ground elevation is greater than the base SWEL (AS line).
- The End of Transect (ET) line requires no data but indicates the end of the input data.

Each line has an alphanumeric field describing the type of input for that line, followed by 10 numeric fields describing the parameters.

To ensure proper modeling, the Mapping Partner shall enter all segments of each transect either as fetches or obstructions, with one input line used for each fetch or obstruction segment. The first two columns of each line identify the type of fetch or obstruction. The remaining 78 columns consist of one field of six columns followed by nine fields of eight columns. The Mapping Partner shall right-justify the numbers in any data field only if no decimal point is used. Decimal points are permitted but not required. The endpoint of one fetch or obstruction is the beginning of the next. The first two numeric fields of each line are used to read in the stationing (measured in feet from the beginning of transect) and elevation (in feet) of the endpoint. The last two fields used on each line are for entering new SWELs. An interpolation is performed within a transect segment starting at the closest station with an input SWEL. This interpolation uses the new SWEL input at the endpoint of the segment, and the SWEL input at a previous segment. If these fields are blank or zero, the SWELs remain unchanged.

The input data requirements are summarized below for each line type. The Title line must be the first line, followed by the IE line, followed by any combination of the various fetch and obstruction lines. The ET line must be the last card entered for the transect. A blank line must follow to signify the end of the run. If multiple transects are being run, the Title line for the next transect will follow the blank line. All units are in feet unless otherwise specified.

TITLE Line (Title)
This line is required and must be the first input line.

Data Field	Columns	Contents of Data Fields
0	1-2	Blank
1-10	3-80	Title information centered about column 40

IE Line (Initial Elevations)

This line is required and must be the second line. It is used to begin a transect at the shoreline and to compute the wave height arising through the overwater fetch.

Data Field	Columns	Contents of Data Fields
0	1-2	IE
1	3-8	Stationing of endpoint of initial overwater fetch, in feet (zero at beginning of transect)
2	9-16	Ground elevation at endpoint in feet (usually zero at beginning of transect)
3	17-24	Overwater fetch length (miles), if wave condition is to be calculated. Values of 24 miles or greater yield identical results.
4	25-32	10-percent-annual-chance SWEL in feet
5	33-40	1-percent-annual-chance SWEL in feet
6	41-48	Initial wave height in feet; a blank or zero causes a default to a calculated wave height
7	49-56	Initial wave period (seconds); a blank or zero causes a default to a calculated wave period. The period is usually the most convenient wave specification for open coasts.
8-10	57-80	Not used

AS Line (Above Surge)

This line is used to identify the endpoint of an area with a ground elevation greater than the 1-percent-annual-chance SWEL (such as a high dune or other land mass). It is used when the ground surface temporarily rises above the 1-percent-annual-chance SWEL. The line immediately preceding the AS line must enter the stationing and elevation of the point at which the ground elevation first equals the 1-percent-annual-chance SWEL. The SWEL on the inland side may differ from the SWEL on the seaward side. The ground elevation entered on the AS line must equal the SWEL that applies to the inland side of the land mass. Computer calculations will be terminated if a ground elevation greater than the 1-percent-annual-chance SWEL is encountered.

Data Field	**Columns**	**Contents of Data Fields**
0	1-2	AS
1	3-8	Stationing at endpoint, in feet, of area above 1-percent-annual-chance SWEL
2	9-16	Ground elevation in feet at endpoint
3	17-24	A blank or zero indicates no change to the 10-percent-annual-chance SWEL; otherwise new 10-percent-annual-chance SWEL
4	25-32	A blank or zero indicates no change to the 1-percent-annual-chance SWEL; otherwise new 1-percent-annual-chance SWEL
5-10	33-80	Not used

BU Line (Buildings)

This line enters information needed to compute wave dissipation at each group of buildings.

Data Field	**Columns**	**Contents of Data Fields**
0	1-2	BU
1	3-8	Stationing of endpoint, in feet, of group of buildings
2	9-16	Ground elevation at endpoint, in feet
3	17-24	Ratio of open space between buildings to total transverse width of developed area
4	25-32	Number of rows of buildings
5	33-40	A blank or zero indicates no change to 10-percent-annual-chance SWEL; otherwise new 10-percent-annual-chance SWEL
6	41-48	A blank or zero indicates no change to 1-percent-annual-chance SWEL; otherwise new 1-percent-annual-chance SWEL
7-10	49-80	Not used

DU Line (Dune)

This line enters information necessary to compute wave dissipation over flooded sand dunes and other natural or manmade elongated barriers (such as levees and seawalls).

Data Field	Columns	Contents of Data Fields
0	1-2	DU
1	3-8	Stationing at top of dune or barrier, in feet
2	9-16	Elevation at top of dune or barrier, in feet
3	17-24	A blank or zero indicates a dune or other natural barrier; any other number indicates a seawall or other manmade barrier
4	25-32	A blank or zero indicates no change to 10-percent-annual-chance SWEL; otherwise new 10-percent-annual-chance SWEL
5	33-40	A blank or zero indicates no change to 1-percent-annual-chance SWEL; otherwise new 1-percent-annual-chance SWEL
6-10	41-80	Not used

IF Line (Inland Fetch)

This line enters the parameters necessary to compute wave regeneration through somewhat sheltered fetches and over shallow inland water bodies. The IF regeneration is computed using a sustained windspeed of 60 mph.

Data Field	Columns	Contents of Data Fields
0	1-2	IF
1	3-8	Stationing at endpoint of fetch, in feet
2	9-16	Ground elevation at endpoint, in feet
3	17-24	A blank or zero indicates no change to 10-percent-annual-chance SWEL; otherwise new 10-percent-annual-chance SWEL
4	25-32	A blank or zero indicates no change to 1-percent-annual-chance SWEL; otherwise new 1-percent-annual-chance SWEL
5-10	33-80	Not used

OF Line (Overwater Fetch)

This line enters the parameters necessary to compute wave regeneration over large bodies of water (such as large lakes or bays) using a sustained windspeed of 80 mph. If an inland body of water is sheltered and has a depth of 10 feet or less, the IF line calling for reduced windspeed should be used.

Data Field	Columns	Contents of Data Fields
0	1-2	OF
1	3-8	Stationing at endpoint of fetch, in feet
2	9-16	Ground elevation at endpoint, in feet
3	17-24	A blank or zero indicates no change to the 10-percent-annual-chance SWEL; otherwise new 10-percent-annual-chance SWEL
4	25-32	A blank or zero indicates no change to 1-percent-annual-chance SWEL; otherwise new 1-percent-annual-chance SWEL
5-10	33-80	Not used

VE Line (Vegetation)

This line enters parameters necessary to compute wave dissipation due to rigid vegetation stands. See Subsection 2.7.3.2 for additional information on coding with the VE card.

Data Field	Columns	Contents of Data Fields
0	1-2	VE
1	3-8	Stationing at endpoint of vegetation, in feet
2	9-16	Ground elevation at endpoint, in feet
3	17-24	Mean effective diameter of equivalent circular cylinder, in feet
4	25-32	Average actual height of vegetation, in feet
5	33-40	Average horizontal spacing between plants, in feet
6	41-48	Drag coefficient; a blank or zero, causes a default to 1.0
7	49-56	A blank or zero indicates no change to 10-percent-annual-chance SWEL; otherwise new 10-percent-annual-chance SWEL
8	57-64	A blank or zero indicates no change to 1-percent-annual-chance SWEL; otherwise new 1-percent-annual-chance SWEL
9-10	65-80	Not used

VH Line (Vegetation Header for Marsh Grass)

Marsh grass is often part of a plant community that may consist of several types. The VH line is used to enter data that apply to all plant types modeled in the transect segment. To enter data for each plant type, MG lines for each plant type must follow the VH line. See Subsection 2.7.3.2 for additional information on coding with the VH card.

Data Field	Columns	Contents of Data Fields
0	1-2	VH
1	3-8	Stationing at endpoint of marsh vegetation segment, in feet
2	9-16	Ground elevation at endpoint, in feet
3	17-24	Reg_p, number of the primary seacoast region for default plant parameters. See Figure D.2.7-3.
4	25-32	Wt_p, weighting factor for the primary seacoast region
5	33-40	Reg_s, number of secondary seacoast region. See Figure D.2.7-3
6	41-48	N_{p1}, number of plant types; range is 1 to 10, inclusive. One MG line is required for each plant type.
7	49-56	A blank or zero indicates no change to the 10-percent-annual-chance SWEL; otherwise new 10-percent-annual-chance SWEL
8	57-64	A blank or zero indicates no change to the 1-percent-annual-chance SWEL; otherwise new 1-percent-annual-chance SWEL
9	65-72	Not used
10	73-80	This field is for overriding the default method of averaging flood hazard factors in A Zones; if 1 in column 80, averaging process begins or ends at end of vegetation segment; otherwise, default averaging method is used

MG Line (Marsh Grass)

This line is used to enter data for a particular plant type. The first MG line must be preceded by a VH line. For the common seacoast marsh grasses listed in Table D.2.7-2, some potentially useful default values are supplied in Table D.2.7-4, and the program can provide additional default values (FEMA, October 1984). If a plant type not listed in the table is used, then appropriate data must be developed for Fields 2 through 9. See Subsection 2.7.3.2 for additional information on coding with the MG card.

Data Field	Columns	Contents of Data Fields
0	1-2	MG
1	5-8	Marsh plant type abbreviation (see Table D.2.7-2)
2	9-16	C_D, effective drag Coefficient; default value is 0.1
3	17-24	F_{cov}, decimal fraction of vegetated area to be covered by this plant type; a blank or zero causes a default to be calculated so that each plant type is represented equally
4	25-32	h, mean unflexed height of stem (feet); for marsh plants, the inflorescence is not included
5	33-40	N, number of plants per square foot
6	41-48	D_1, base stem diameter (inches)
7	49-56	D_2, midstem diameter (inches)
8	57-64	D_3, top stem diameter (inches)
9	65-72	CA_b, ratio of the total frontal area of cylindrical part of leaves to frontal area of main stem
10	73-80	Not used

ET Line (End of Transect)

This line is required and must be the last card, because it identifies the end of input for the transect.

Data Field	Columns	Contents of Data Fields
0	1-2	ET
3-10	3-80	Not used

D.2.7.3.2 Treatment of Vegetation by WHAFIS

For the areas of rigid vegetation located on the transect, the required input values are the drag coefficient, CD; mean wetted height, h; mean effective diameter, D; and mean horizontal spacing, b. The value of CD should vary between 0.35 and 1.0, with 1.0 being used in most cases of wide vegetated areas. When the vegetation is in a single stand, the Mapping Partner shall use a value of 0.35. The Mapping Partner shall obtain representative values for h, D, and b from field surveys.

For marsh vegetation, a more complicated specification is required for completeness. The eight parameters used to describe the dissipational properties of a specific type are explained in Table D.2.7-1. However, WHAFIS incorporates considerable basic information on the eight common types of seacoast marsh plants listed in Table D.2.7.2 (FEMA, 1984). That information can be used by specifying either the Table D.2.7.2 abbreviation or a geographical region, as indicated in Figure D.2.7-3. Figure D.2.7-3 shows the coastal wetland regions of the Atlantic and Gulf coasts, along with the identifying numbers used in WHAFIS. If the site is near a regional border, the likely plant parameters can be interpolated using an input weighting factor. Although the South Texas region has insignificant amounts of marsh grass, it is included for use in spatial interpolation. Figures D.2.7-4 and D.2.7-5 provide information on the typical salt tolerance and vertical distribution of plants across the profile.

Climate affects the geographic range of each marsh plant type, so that some plant types are not found in all regions. Table D.2.7-3 lists the dominant plant type in each region, where the term "dominant" refers to the plant types that cover the largest amount of area in the marshes. Table D.2.7-4 shows the significant plant types in each region, where the term "significant" refers to the plant types that occur in large enough patches (at least 10,000 square feet) to significantly affect waves. For marsh plants, simply the coastal wetland region, plant type, and area or percentage of coverage may be specified. Given this information, WHAFIS will supply default values for the other marsh plant parameters appropriate to the site (FEMA, 1984).

Following the identification of the marsh plant types present, the area and fraction of coverage, F_{cov}, for each plant type must be calculated. The total area of marsh vegetation coverage is determined for each transect. The different types of vegetation within this area usually occur in patches. F_{cov} is defined for each plant type as the ratio of the patch area for that type to the total marsh area. Using the above data, a fairly good determination can be made of the plant types present, but an attempt should be made to confirm these plant types. Local, county, or State officials may provide some assistance, and a site visit can be very useful.

Table D.2.7-1. Marsh Plant Parameters

Parameter	Explanation
C_D	Effective drag coefficient. Includes effects of plant flexure and modification of the flow velocity distribution. Default value is 0.1, usually appropriate for marsh plants without strong evidence to the contrary.
F_{cov}	Fraction of coverage. A default value is calculated by the program so that each plant type in the transect is represented equally, and the sum of the coverage for the plant types is equal to 1.0.
h	Unflexed stem height (feet). The stem height does not include the flowering head of the plant, the inflorescence.
N	Number density. Expressed as plants per square foot. The relationship to the average spacing between plants, b, can be expressed as $N = 1/b^2$.
D_1	Base stem diameter (inches). Default value may be determined from stem height and regression equations built into the program.
D_2	Midstem diameter (inches). Default value may be determined from plant type and base stem diameter.
D_3	Top stem diameter (inches), at the base of the inflorescence. Default value may be determined from plant type and base stem diameter.
CA_b	Ratio of the total frontal area of the cylindrical portion of the leaves to the frontal area of the stem below the inflorescence. Default value may be determined from the plant type.

Table D.2.7-2. Abbreviations of Marsh Plant Types used in WHAFIS

Species or Subspecies	**Abbreviation**
Cladium jamaicense (saw grass)	CLAD
Distichlis spicata (salt grass)	DIST
Juncus gerardi (black grass)	JUNM
Juncus roemerianus (black needlerush)	JUNR
Spartina alterniflora (medium saltmeadow cordgrass)	SALM
Spartina alterniflora (tall saltmeadow cordgrass)	SALT
Spartina cynosuroides (big cordgrass)	SCYN
Spartina patens (saltmeadow grass)	SPAT

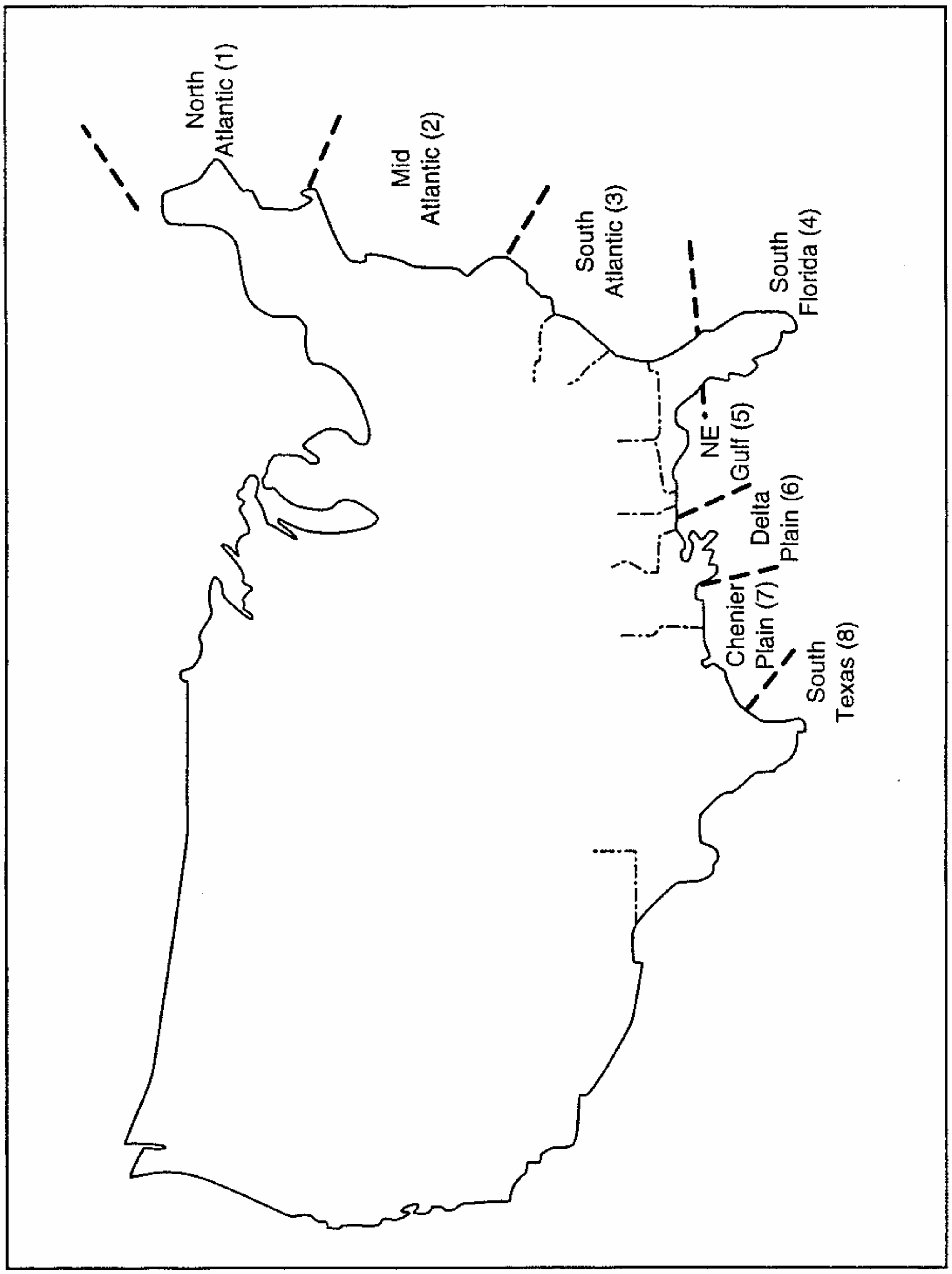


Figure D.2.7-3. Coastal wetland regions of Atlantic and Gulf coasts having enough marsh grass to significantly affect wave heights.

Table D.2.7-3. Dominant Marsh Plant Types by Region and Habitat

Region Number	Region Name	Habitat	Dominant Species
1	North Atlantic	salt[1] brackish[2]	**S. alterniflora* (medium, tall) *Spartina patens*
2	Mid-Atlantic	salt brackish	*S. alterniflora* (medium, tall) **Juncus roemerianus/S. patens*
3	South Atlantic	salt brackish	**S. alterniflora* (medium, tall) *J. roemerianus*
4	South Florida	salt brackish	*S. alterniflora* (medium, tall) **C. jamaicense*
5	Northeastern Gulf	salt brackish	--- **J. roemerianus*
6	Delta Plain	salt brackish	**S. Alterniflora* (medium, tall) *S. patens*
7	Chenier Plain	salt brackish	*S. alterniflora* (medium, tall) **S. patens*
8	South Texas	salt brackish	--- ---

Salt concentration is greater than 20 parts per thousand (ppt)

[2]Salt concentration is between 5 and 20 ppt

*When more than one dominant plant type occurs within the region, the indicated type covers the largest geographic area (acreage)

--- Insignificant amounts of marsh plants within the given habitat in the region

Table D.2.7-4. Significant Marsh Plant Types in Each Seacoast Region and WHAFIS Default Regional Plant Parameter Data

REGION NO.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
REGION NAME:	**NORTH ATLANTIC**	**MID-ATLANTIC**	**SOUTH ATLANTIC**	**SOUTH FLORIDA**	**NORTHEASTERN GULF**	**DELTA PLAIN**	**CHENIER PLAIN**	**SOUTH TEXAS**
CLAD	---	---	---	7.50(+) 0.0656 6	6.00(2) 0.0260 6	---	---	---
DIST	---	0.78(1) 0.0039 211	1.00(1) 0.038 243	1.00(+) 0.0038 248	--- ---	1.08(4) 0.0035 102	1.08(+) 0.0035 102	---
JUNM	1.23(1) 0.0042 300	1.23(+) 0.0042 300	---	---	---	---	---	---
JUNR	---	2.95(+) 0.0095 147	2.95(+) 0.0095 147	---	2.95(3) 0.0095 147	3.00(4) 0.0106 83	2.95(+) 0.0095 147	---
SALM	1.39(1) 0.0184 45	1.06(1) 0.0103 36	1.63(1) 0.0141 12	1.63(+) 0.0141 12	---	1.67(4) 0.0141 21	2.62(5) 0.0211 16	---
SALT	1.86(1) 0.0175 37	2.21(1) 0.0169 18	3.20(1) 0.0183 10	3.20(+) 0.0183 10	---	3.20(4) 0.0183 10	3.20(+) 0.0183 10	---
SCYN	---	---	8.29(+) 0.0492 6	---	---	4.00(4) 0.0267 7	---	---
SPAT	1.03(1) 0.0025 409	0.85(1) 0.0019 327	1.65(1) 0.0019 236	---	2.58(2) 0.0026 236	1.88(4) 0.0016 333	1.88(+) 0.0019 333	---

Data arranged in vertical triplets:
h, stem height below inflorescence, in feet
D, base diameter, in feet
N, number density, in inverse square feet

Parenthetical references indicate data source:
1 = Hardisky and Reimold, 1977
2 = Monte, August 1983
3 = Kruczynski, Subrahmanyam, Drake, 1978
4 = Hopkinson, Gosselink, Parrondo, 1980, Diameters extrapolated
5 = Turner and Gosselink, 1975, Diameters extrapolated
+ = Extrapolated Data
--- = Insignificant amounts of this plant type in the region

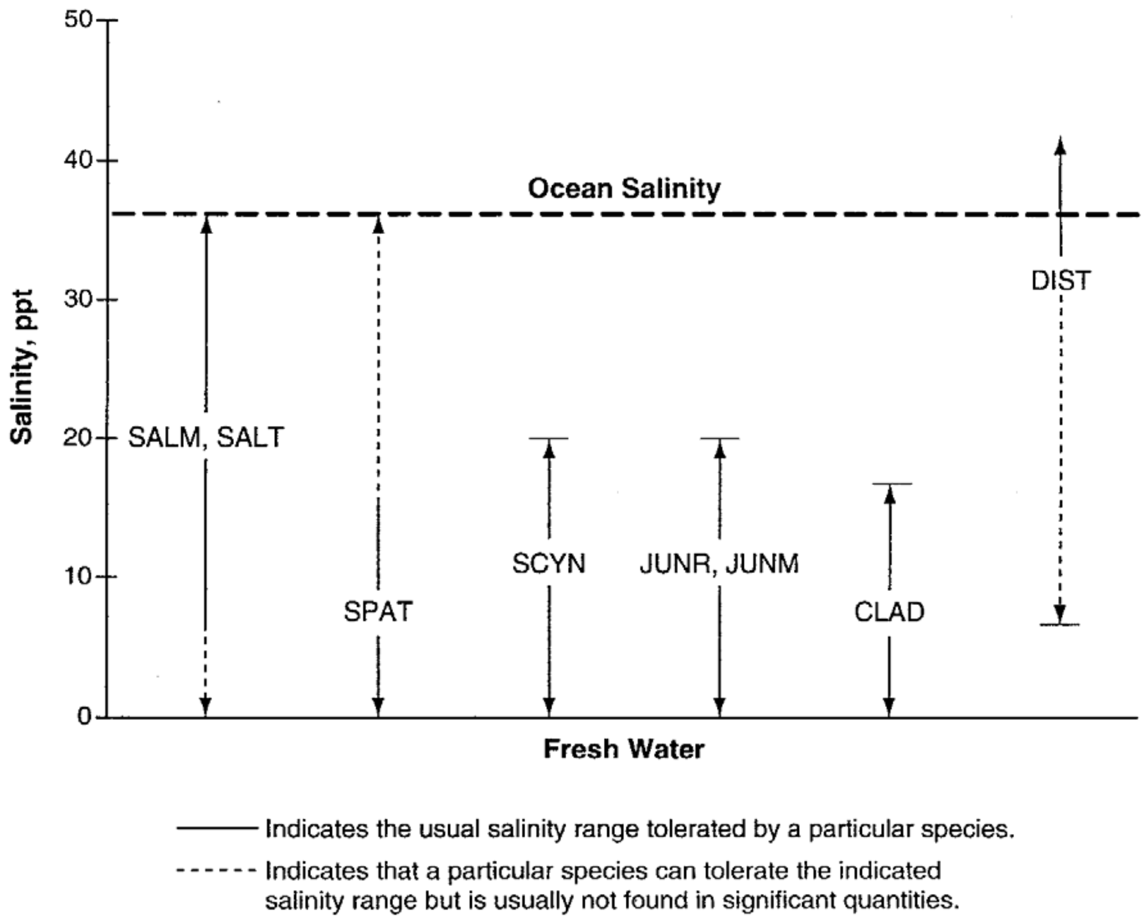


Figure D.2.7-4. Salinity Tolerance of Marsh Plants, from Knutson and Woodhouse, 1983

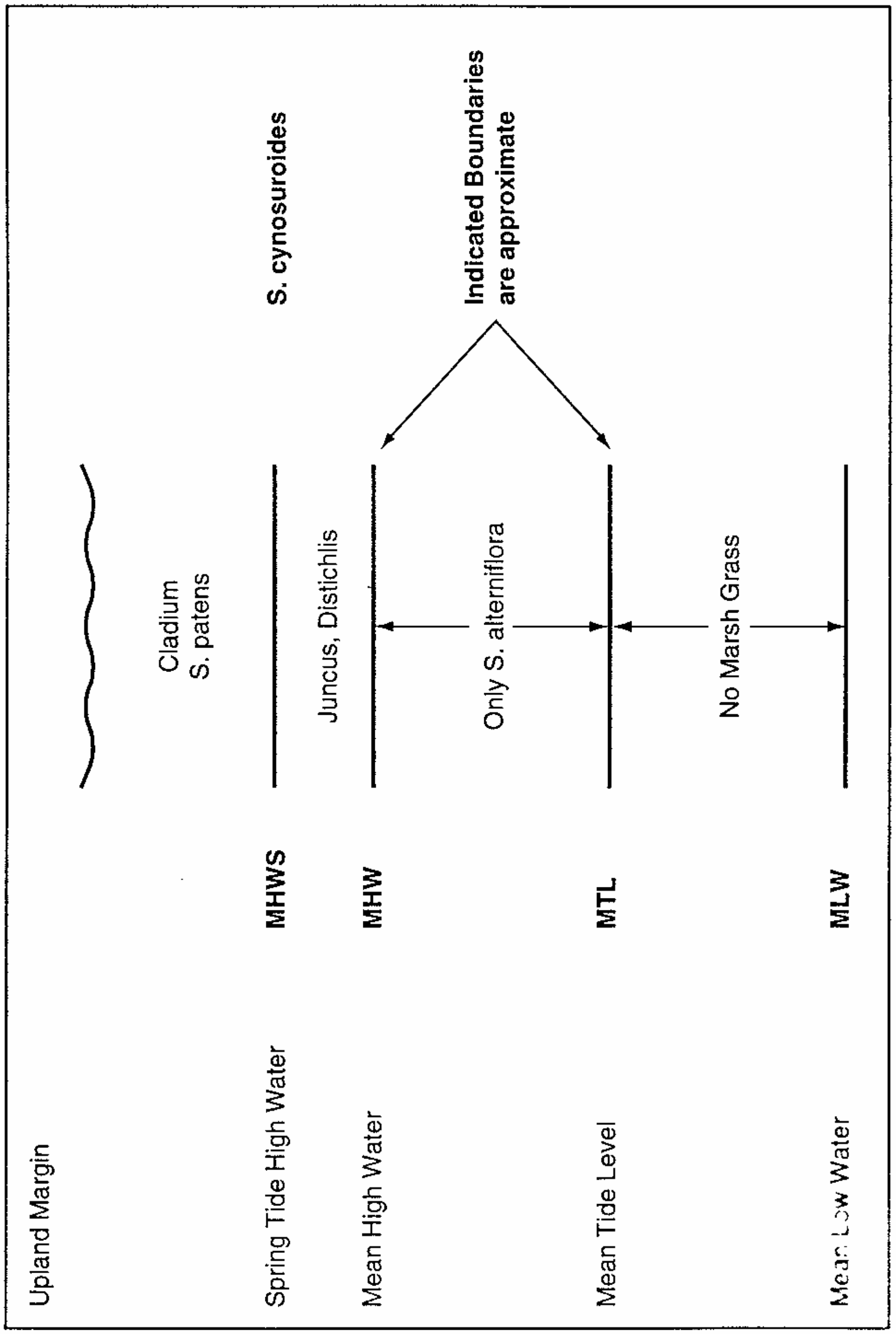


Figure D.2.7-5. Tidal Control on Salt Marsh Plant Viability

D.2.7.4 WHAFIS Output Description

The output of the program provides all the data necessary for plotting the BFEs and flood insurance risk zones along the transect. The output is in six parts, as discussed below.

Part 1 - Input

This is a printout showing all input data lines and the parameters assigned to each line, both manually and by default. This is followed by a more detailed printout with column headings for each input data line. When VH and MG Lines are used, a separate insert will be printed directly beneath the MG Line to show any default values supplied by the computer.

Part 2 - Controlling Wave Heights, Spectral Peak Wave Period, and Wave Crest Elevations

This is a list of the calculated controlling wave heights, spectral wave peak periods, and wave crest elevations at the endpoint of each fetch and obstruction of the input, and at calculation points generated between the input stations.

Part 3 - Location of Areas Above 1-Percent-Annual-Chance Surge

This is a list of the locations where the ground elevation is greater than the 1-percent-annual-chance stillwater (surge) elevation. Only areas identified by AS lines are listed.

Part 4 - Location of Surge Elevations

This is a list of the 10- and 1-percent-annual-chance stillwater (surge) elevations and the stationing of the points where each set of SWELs first becomes fully effective.

Part 5 - Location of V Zones

This is a list of the locations of the V/A Zone boundaries and the locations of the V-zone areas relative to these boundaries. The stationing is given for each V/A Zone boundary. The locations of the V-zone areas in relation to these boundaries are given as windward or leeward of the boundary.

Part 6 - Numbered A Zones and V Zones

This is a list of the zone data needed to delineate the flood hazard boundaries on the FIRM. The location of a flood zone boundary and the wave crest elevation at that boundary are on the left. Between the boundary listings are the zone designations and FHFs. Under FEMA's Map Initiatives Procedure guidelines, all numbered V and A Zones should be changed to VE and AE Zones, respectively (elevations will not change), and the FHFs can be ignored (FEMA, 1991). When the same zone and elevation are repeated in a list, they should be treated as a single zone.

D.2.7.5 WHAFIS Error Messages

The error messages that may appear when running the model are described below.

- "AS card ground elevation less than SWEL, should use other type card, job dumped." Only use the AS (above surge) line when the ground elevation is above the SWEL. Otherwise use IF, OF, BU, DU, VE, or VH.

- "Ground elevation greater than surge elevation encountered, job dumped." If ground elevation is above surge elevation, the AS card should be used.

- "Average depth less than or equal to zero, job dumped." The water depth must be greater than zero, or a wave height cannot be computed. Check the SWEL and the ground elevation if the point of job dump is not the last point along the transect profile.

- "The above card contains illegal data in the first 2 columns." Check input data for incorrect values, or input in the wrong columns. Aside from the title line, the first two columns in each line should contain the card identifiers.

- "Transmitted wave height at last fetch or obstruction = ______ which exceeds 0.5." Code the transect profile up to the inland limit where ground elevation intersects the SWEL so that wave height should decrease to zero. If the scope of work ends at the corporate limits before the ground elevation meets the SWEL, this message can be ignored.

- "Array dimensions exceeded. Job dumped." The size of the array is limited, and the number of input parameters has exceeded the array. Check the number of input parameters at the location where the job dumped.

- "Invalid data in field 1 of IF card, a" etc. Check input data to make sure that data are in the correct columns.

- "Wave period less than or equal to zero in subroutine fetch. Abort run." Either a fetch length or a wave period must be input for the program to run properly. Check input data.

- "Invalid data in field 3 or field 5 of VH card." Check input data.

- "Invalid data in field 4 of VH card." Check input data.

- "Invalid data in field 3 of MG card." Check input data. The fraction of vegetated area covered by the stated plant type should be a decimal number between 0.0 and 1.0.

- "Missing MG card or incorrect data in field 6 of VH card." The MG card must always follow the VH card. Field 6 of the VH card pertains to the number of plant types, and one MG card is required for each plant type.

- "Invalid input data." Check input data for invalid characters, such as an O instead of a zero. Check to be sure that all data are in their correct columns.

- "Fcov was found to be negative for plant type = _______." Check input data to be sure that the decimal fraction of the vegetated area covered by the plant type is not negative.

- "Ncov is .LE. zero in Sub.Lookup when it should be .GT. zero. Abort run." Check input for number of plants covering the area.

- "The first card is not an IE card, this transect is aborted. Continued to next transect." The first card after the title line must always be an IE card. Check input data.

- "**** The surge elevation at this station (stationing ____), which is ____ card, is less than the ground elevation. The interpolation process is continued. *** Please double check the surge and ground elevations in the vicinity of this station" The surge elevation should not be below the ground elevation. If the interpolated surge elevation is below the ground elevation, insert additional cards to specify surge and ground elevations and use an AS card if necessary.

- "Interpolation line cuts off more than two portions of high ground ridge. This transect is aborted, re-assign 1-percent-annual-chance elevations at high ground stations." When the interpolated value falls below the ground elevation, insert additional cards to better model the area and set the SWEL equal to the ground elevation where appropriate. Insert AS cards as necessary.

- "**** Unreasonable high ground elevation at station ____ which is ____ card. This transect is aborted, continued to next transect. **** Double check the surge and ground elevations in the vicinity of this station. If the ground elevations are correct, either assign a higher surge elevation or use AS cards." Add additional input data as necessary to better define the ground elevation and surge elevation in this area.

D.2.7.6 WHAFIS Documentation for the FIS

The Mapping Partner shall document all assumptions used to define input waves for WHAFIS analyses, including a brief description of offshore wave conditions, and a description of wave transformation, attenuation or dissipation between the wave source area and the shoreline. In sheltered waters, this shall include a summary of fetch determination, winds (speeds, directions, and duration), and bathymetry used in hindcasts. The documentation shall include the approximations or assumptions used in the analysis. When observational data, such as wave buoy data, are available, the wave height, period, and spectral parameters should be compared to the predicted waves.

The Mapping Partner shall document the WHAFIS analysis assumptions, methods, input data, and results. This shall include documentation of any field observations or measurements, as well as available historical or anecdotal information regarding overland wave propagation during flooding events.

See Subsection D.2.12 for additional documentation considerations.

D.2.8 Wave Runup and Overtopping

This subsection provides guidance for calculating wave runup and overtopping on barriers. Special cases where runup occurs on steep slopes and where runup exceeds barrier or bluff crests are discussed. Guidance for mapping flood hazards based on runup and overtopping values is given.

D.2.8.1 Wave Runup

D.2.8.1.1 Overview

Wave runup is the uprush of water from wave action on a shore barrier intercepting stillwater level. The extent of runup can vary greatly from wave to wave in storm conditions, so that a wide distribution of wave runup elevations provides the precise description for a specific situation. The water wedge generally thins and slows during its excursion up the barrier, as residual forward momentum in wave motion near the shore is fully dissipated or reflected. The notable characteristic of this process for the present purposes is the wave runup elevation, the vertical height above the stillwater level[6], ultimately attained by the extremity of the uprushing water. Wave runup at a shore barrier can provide flood hazards above and beyond those from stillwater inundation and incident wave geometry, as illustrated in Figure D.2.8-1.

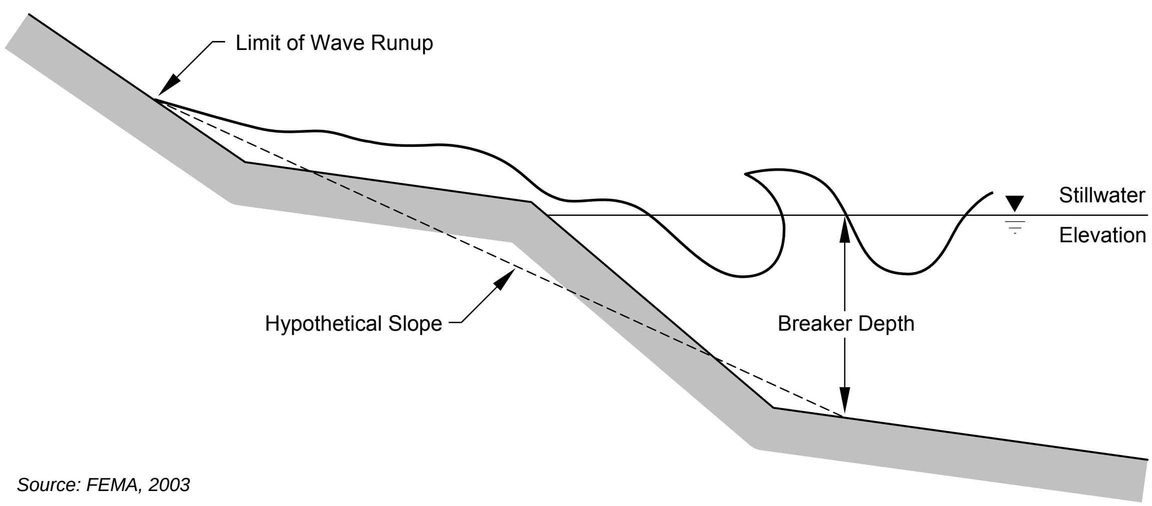


Figure D.2.8-1. Wave Runup Sketch.

[6] The Mapping Partner must be aware of the relationship between stillwater level, wave setup, and wave runup. Outputs from some runup and overtopping calculation procedures include wave setup effects; thus, an accurate specification of input water level for the procedures is necessary to avoid double-counting wave setup. The Mapping Partner must also know whether water-level outputs from modern hydrodynamic and storm-surge models -- which will be used as inputs to transect-based wave height, wave runup, and wave overtopping analyses -- include or exclude wave setup. If wave setup is not included in the model water-level output, the runup procedures described here can be applied directly. If wave setup is included in the model water-level output, the Mapping Partner must estimate and subtract out wave setup from the stillwater level before using the runup procedures described here.

Current policy for the NFIP is to define the wave runup elevation as the value exceeded by 2 percent of the runup events[7]. The 2-percent value was chosen during the development of the Pacific Coast *Guidelines and Specifications for Flood Hazard Mapping Partners* (see Appendix D, Subsection D.4). This runup elevation is a short-term statistic associated with a group of waves or a particular storm. It is a standard definition of runup, commonly denoted as $R_{2\%}$. This 2 percent is different from the 1-percent-annual-chance condition that is associated with long-term extreme value statistics. The 1-percent condition has a 1-percent annual probability of occurrence, which corresponds approximately to the 100-year condition, while the runup statistic corresponds to a 2-percent exceedance occurance in several hours of waves. To avoid confusion, the 2-percent runup is referred to as the "total runup" or just the "runup" and is denoted as $R_{2\%}$. Unless otherwise indicated, the runup referred to in all subsections of D.2 is the 2-percent runup.

Incident wave runup on natural beaches or barriers is usually expressed in a form originally due to Hunt (1959) in terms of the so-called Iribarren number, ξ, as follows:

$$\xi = \frac{m}{\sqrt{H/L}} \tag{D.2.8-1}$$

in which m is a representative profile slope and is defined, depending on the application, as the beach slope or the slope of a barrier that could be either a dune or a constructed element such as a breakwater or revetment. H and L are wave height and length, respectively. The wave characteristics in the Iribarren number can be expressed in terms of breaking or deepwater characteristics. For these purposes, two wave characteristics in the Iribarren number are used, including that based on the significant deepwater wave height (H_o) and peak or other wave period (T) of the deepwater spectrum, and that based on the significant wave height at the toe of a barrier. The first definition for a sandy beach is as follows:

$$\xi_o = \frac{m}{\sqrt{H_o/L_o}} \tag{D.2.8-2}$$

where L_o is the deepwater wave length:

$$L_o = \frac{g}{2\pi} T^2 \tag{D.2.8-3}$$

and g is the gravitational constant. The beach profile slope is the average slope out to the breaking depth associated with the significant wave height.

[7] Walton (1992) concluded that both theory and laboratory experiments show that the 2-percent runup height above the stillwater level is approximately 2.2 times the mean runup height. Past NFIP policy was to define the runup elevation based on the mean runup.

The 2-percent incident wave runup on natural beaches (R_{inc}) is expressed in terms of the Iribarren number as:

$$R_{inc} = 0.6 \frac{m}{\sqrt{H_o / L_o}} H_o \tag{D.2.8-4}$$

For the case of runup on a barrier, the Iribarren number is formulated using the significant wave height at the toe of the barrier (see Subsection D.2.8.1.5).

The following subsections discuss runup on beaches and barriers in more detail, using RUNUP 2.0, ACES, and other methods. Special runup cases are also discussed.

D.2.8.1.2 FEMA Wave Runup Model Description (RUNUP 2.0)

NOTE: The result obtained from FEMA's RUNUP 2.0 model is the mean runup value. Since current NFIP policy is to use the 2-percent runup, if RUNUP 2.0 is used in an FIS, interim guidance calls for the mean runup height, obtained with the RUNUP 2.0 model, to be multiplied by 2.2 to obtain the 2-percent runup height. This value is then added to the 1-percent-annual-chance stillwater level *without wave setup* to obtain the total wave runup elevation for an FIS.

The current version of the FEMA Wave Runup Model is RUNUP 2.0. This model requires the following inputs: the stillwater flood level (without wave setup), the shore profile and roughness, and incident deepwater wave conditions. The program computes, by iteration, a mean wave runup elevation fully consistent with the guidance available (Stoa, 1978). This determination includes an analysis separating the profile into an approach segment next to the steeper shore barrier, and interpolation between runup guidance for simple configurations bracketing the specified situation.

Additional description of the workings of RUNUP 2.0 can assist informed preparation of input and interpretation of output. The incorporated guidance gives runup elevation, as a function of wave condition and barrier slope, for eight basic shore configurations distinguished by water depth at the barrier toe, along with the approach geometry. Where those basic geometries do not appropriately match the specified profile, reliance is placed on the composite slope method (Saville, 1958); this assumes that the input shore profile (composite slope) is equivalent to a hypothetical uniform slope, as shown in Figure D.2.8-1. The runup elevations are derived from laboratory measurements in uniform wave action, rather than the irregular storm waves usually accompanying a flood event. Runup guidance for uniform waves, however, also pertains to the mean runup elevation from irregular wave action with identical mean wave height and mean wave period. Figure D.2.8-2 presents an overview of the basic computation procedure in RUNUP 2.0.

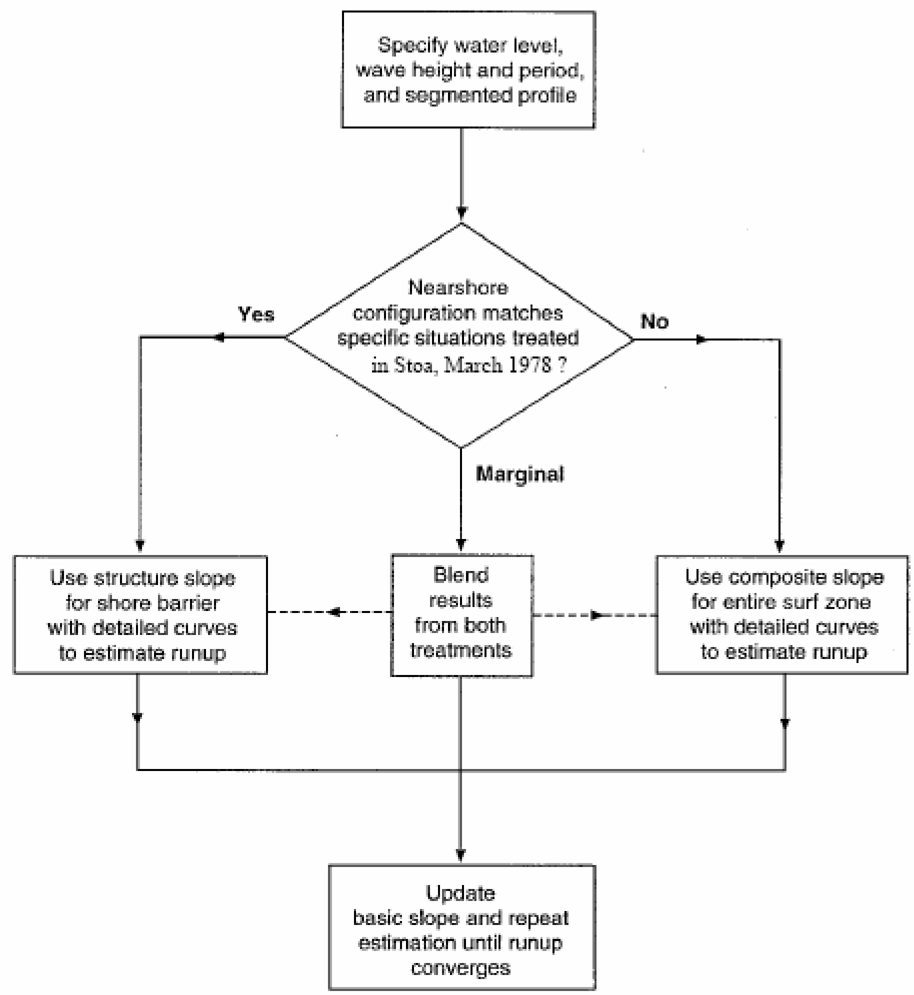


Figure D.2.8-2 Overview of Computation Procedure Implemented in FEMA Wave Runup Model (RUNUP 2.0)

D.2.8.1.2.1 RUNUP 2.0 Input Preparation

The input to the Wave Runup Model is done by transects along the study area shorelines, as was done for overland wave propagation calculations. Because the runup results are very sensitive to shore slope or steepness, it is important to have at least one transect for each distinct type of shore geometry. Often, areas with similar shore slopes are located throughout a community, and the results of one transect can be applied to all similar areas. This is especially typical of New England communities with rocky bluffs. When the Wave Runup Model is being applied to dune remnants where eroded slopes are fairly uniform, transect location is governed by the upland land-cover characteristics, which are major considerations in the WHAFIS model.

The ground profile for the transect is plotted from the topography and bathymetry referenced to a common vertical datum. The profile must extend from an elevation below the breaker depth to an elevation above the limit of runup, or to the maximum ground elevation. An adequate vertical extent for the transect description will usually be 1.5 times the wave height above and below the SWEL. If the landward profile does not extend above the computed runup, it will be assumed that the last positive slope segment continues indefinitely. This is very common with low barriers. The Mapping Partner shall select the last slope carefully, so it is representative. To complete the description, each slope segment of the profile will need a roughness coefficient. Common values are presented in Table D.2.8-1. The roughness coefficient must be between zero (maximum roughness) and one (hydraulically smooth), and values for slope segments above the SWEL control the estimated runup. The roughness coefficient (*r*) is used as a multiplier for runup magnitude (*R*), defined on a smooth barrier to estimate wave runup with a rough barrier.

Table D.2.8-1. Values for Roughness Coefficient in Wave Runup Computations

Roughness Coefficient	Description of Barrier Surface
1.00	Sand; smooth rock, concrete, asphalt, wood, fiberglass
0.95	Tightly set paving blocks with little relief
0.90	Turf, closely set stones, slabs, blocks
0.85	Paving blocks with sizable permeability or relief
0.80	Steps; one stone layer over impermeable base; stones set in cement
0.70	Coarse gravel; gabions filled with stone
0.65	Rounded stones, or stones over impermeable base
0.60	Randomly placed stones, two thick on permeable base; common riprap installations
0.50	Cast-concrete armor units: cubes, dolos, quadripods, tetrapods, tribars, etc.

Transects are approximated by the minimum adequate number of linear segments, up to a limit of 20. Segments may be horizontal, or higher at the landward end; portions with the opposite inclination should be represented as horizontal when developing the transect approximation. The use of many linear segments to represent a transect may be a wasted effort, because the Wave Runup Model may combine adjacent segments in defining the appropriate approach and barrier extents. With the runup computation procedure, the Mapping Partner shall apply engineering judgment in transect representation to obtain the most valid estimate of wave runup elevation.

The input transect must reflect wave-induced modifications expected during the base flood event, including erosion on sandy shores with dunes. The Mapping Partner shall represent only coastal structures expected to remain intact throughout the base flood event on a specific transect. Besides the transect specification, other required input data for the Wave Runup Model are the base flood SWEL and the incident mean wave condition in deepwater. The specified SWEL should exclude any contributions from wind-wave effects. If available elevations include wave setup, the Mapping Partner shall remove that component before using this model so that the calculated runup elevations do not indicate a doubled wave setup. Basic empirical guidance relates runup at a barrier to the water level in the absence of wave action and thus includes the wave setup component.

The mean wave condition to be specified for valid results with the Wave Runup Model may be derived from other common wave descriptions by simple relationships. Wave heights in deepwater generally conform to a Rayleigh probability distribution, so that the mean wave height equals 0.626 times either the significant height based on the highest one-third of waves, or the zero-moment height derived from the wave energy spectrum. No exact correspondence between period measures exists; but the mean wave period can usually be approximated as 0.85 times the significant wave period or the period of peak energy in the wave spectrum.

Table D.2.8-2 lists a series of wave height and period combinations, of which one should be fairly suitable for runup computations at fully exposed coastal sites (depending on the local storm climate). These mean wave conditions have wave steepness values typical of U.S. hurricanes or within 30 percent of a fully arisen sea for extratropical storms. Commonly, the Mapping Partner may have some difficulty in specifying a precise wave condition as accompanying the base flood. In that case, the Mapping Partner shall also consider wave heights and periods 5 percent higher and 5 percent lower than those selected (or whatever percentages suit the level of uncertainty) and shall run the model with all nine combinations of those values. The average of computed runup values then provides a suitable estimate for mean runup elevation[8]. A wide range in computed runups signals the need for a more detailed analysis of expected wave conditions or for reconsideration of the transect representation.

Table D.2.8-2 Appropriate Wave Conditions for Runup Computations Pertaining to 1-Percent-Annual-Chance Event in Coastal Flood Map Projects

Mean Wave Period (Seconds)	Mean Deepwater Wave Height (Feet)
HURRICANES	
8	12
9	15.5
10	19
11	23
12	27.5
EXTRATROPICAL STORMS	
11	18
12	21.5
13	25
14	29
15	33.5

[8] The resulting averaged mean runup can then be multiplied by 2.2 to obtain the 2-percent wave runup value for an FIS.

D.2.8.1.2.2 RUNUP 2.0 Operation

The input to the FEMA Wave Runup Model consists of several separate lines, specifying an individual transect and the hydrodynamic conditions of interest within particular columns. All input information is echoed in an output file, which also includes computed results on wave breaking and wave runup.

The input format is outlined in Table D.2.8-3. The first two lines of the input give the Name and Job Description, which must be included for each transect. The next line of input is the Last Slope, which contains the cotangent of the shore profile continuing from the most landward point provided. This is followed by the profile points, which define the nearshore profile in consecutive order from the most seaward point. Each line gives the elevation and station of a profile point and the roughness coefficient for the segment between that point and the following point. The roughness coefficient on the last profile line is for the continuation defined in the Last Slope line. The number of profile points cannot exceed 20. The final input is the series of hydrodynamic conditions of interest. Each line here contains the SWEL, a mean wave height in deepwater, and a mean wave period.

Table D.2.8-3 Description of Five Types of Input Lines for Wave Runup Model

Name Line

This line is required and must be the first input line.

Columns	Contents
1-2	Blank
3-28	Client's Name
29-60	Blank
61-70	Engineer's Name
71-80	Job Number

Job Description Line

Columns	Contents
1-2	Blank
3-76	Project description or run identification
77-80	Run Number

Last Slope Line

This line is required and defines the slope immediately landward of the profile actually specified in detail.

Columns	Contents
1-4	Slope (horizontal over vertical or cotangent) of profile continuation
5-80	Blank

Profile Lines

These lines must appear in consecutive order from the most seaward point landward. Each line has the elevation and station of a profile point and the roughness coefficient for the section between that point and the following point. The roughness coefficient on the last profile line is for the continuation defined in the Last Slope Line. The Mapping Partner shall ensure that at least one profile point with a ground elevation greater than the SWEL is specified. The number of Profile Lines cannot exceed 20.

Columns	Contents
1	Last point flag. The most landward point on the profile is indicated by a 1. If not the last point, leave blank.
2	Blank
3-7	Elevation in feet
8	Blank
9-14	Horizontal distance. It is common to assign the shoreline (elevation 0.0) as point 0, with seaward distances being negative and landward distances positive.
15	Blank
16-20	Roughness coefficient in decimal form between 0.00 (most rough) and 1.00 (smooth)
21-80	Blank

Water Level and Wave Parameter Lines

These lines specify the hydrodynamic conditions for runup calculations on each profile; namely, the base flood SWEL and mean wave height and period for deepwater. Typically, the SWEL remains constant for a given profile, while the selected wave conditions closely bracket that expected to accompany the base flood. A maximum of 50 of these lines can be input for each profile.

Columns	Contents
1	Last line, new transect flag. A 1 indicates the last line for a given transect 1 and notifies that another transect is following. If not the last line, or if the last line of the last transect, leave blank.
2-6	SWEL in feet.
7	Blank
8-12	Incident mean wave height described in deepwater, $\overline{H}_o'$, in feet, greater than 1 foot
13	Blank
14-18	Mean wave period, $\overline{T}_s$, in seconds
19-80	Blank

The output as shown in Table D.2.8-4 has two parts. The first page is a printout of the transect listed as a numbered set of profile points, the cotangents (slopes) of the segments, and the roughness coefficient for each segment. The second page is the output table of computed results for each set of conditions, including runup elevation and breaker depth values, each with respect to the specified SWEL, along with an identification of the segment numbers giving the seaward limit to wave breaking and the landward limit to mean wave runup.

```
CLIENT- FEMA                    ** WAVE RUNUP-VERSION 2.0  ** ENGINEERED BY DAB      JOB 1
PROJECT-Runup calculations on Eroded Transect                                      RUN 1    PAGE  1
*******************************************************************************************************************

                  CROSS SECTION PROFILE

                LENGTH   ELEV.    SLOPE     ROUGHNESS

         1      -2670.0   -34.0
                                   97.50       1.00
         2      -1500.0   -22.0
                                   76.25       1.00
         3       -585.0   -10.0
                                   72.50       1.00
         4       -150.0    -4.0
                                   36.43       1.00
         5        -99.0    -2.6
                                   42.22       1.00
         6         53.0     1.0
                                   24.64       1.00
         7        223.0     7.9
                                   39.60       1.00
         8        322.0    10.4
                                     .99       1.00
         9        335.0    23.5
                                   10.00       1.00
        10        350.0    25.0

                    LAST SLOPE   1.00     LAST ROUGHNESS   1.00
```

Table D.2.8-4. Output Example for the FEMA Wave Runup Model

D.2.8.1.2.3 RUNUP 2.0 Output Messages

Several output messages alert the user to specific problems encountered in running the program. All but the last three indicate that the program has stopped without completing the runup calculations.

- "NEGATIVE RUN PARAMETER, PROGRAM STOPS" An input value of wave height or wave period is read as negative or zero. Check that the input has been entered in the correct columns.

- "MORE THAN 20 POINTS IN PROFILE, PROGRAM STOPS" The program accepts a maximum input of 20 points defining the nearshore profile. This encourages a profile approximation that is not overly detailed, because each transect is to represent an extensive area.

- "**** Ho/Lo LESS THAN 0.002 ****" or "**** Ho/Lo GREATER THAN 0.07 ****" These limits on wave steepness pertain to the extent of incorporated guidance on breaker location. They should be adequate to include appropriate mean wave conditions for extreme events and also conform to the usual limits in detailed guidance on wave runup elevations.

- "DATA EXCEEDED TABLE" An entry into subroutine LOOK of the program is not within the parameter bounds of the data table from which a value is sought.

- "SOLUTION DOES NOT CONVERGE" After 10 iterations, the current and previous estimates of runup elevation continue to differ by more than 0.15 foot, and both values are provided in the output table. The calculation is usually oscillating between these two runup estimates when this occurs.

- "COMPOSITE SLOPE USED BUT WAVE MAY REFLECT, NOT BREAK" The output runup elevation relies to some extent on a composite-slope treatment, but the overall slope is steep enough that the specified wave may reflect from the nearshore barrier. Thus, the application of a calculated breaker depth in determining overall slope and runup elevation is questionable.

- "WARNING; COMPOSITE SLOPE USED, BUT INPUT PROFILE DOES NOT EXTEND TO BREAKER DEPTH" If the input profile does not extend seaward of the breaker depth, an incorrect breaker depth may be computed, and the associated runup elevation will also be incorrect. The input profile should include bathymetry to 30 or 40 feet in depth.

D.2.8.1.3 Wave Runup using ACES

FEMA also permits use of the Automated Coastal Engineering System (ACES, USACE, 1992) for runup and overtopping calculations against vertical and sloping structures. (Note that ACES v. 1.07 is on the FEMA list of accepted models for coastal wave effects, which can be found at http://www.fema.gov/plan/prevent/fhm/en_coast.shtm). It should also be noted that ACES uses more up-to-date methods than those contained in the *Shore Protection Manual* (USACE, 1984) or those used in RUNUP 2.0.

ACES v. 1.07 has three wave runup programs: *Irregular Wave Runup on Beaches, Irregular Wave Runup on Riprap*, and *Wave Runup and Overtopping on Impermeable Structures*. Wave setup contributions are included in each of the runup calculations.

The *Irregular Wave Runup on Beaches* module calculates several values of runup (R_{max}, $R_{2\%}$, $R_{10\%}$, $R_{33\%}$, and $\overline{R}$) based on laboratory experiments of runup on smooth, impermeable slopes. The calculations are made given the deepwater significant wave height, peak wave period, and foreshore slope (which yield the surf similarity parameter, $\xi = \tan\theta / (H_o/L_o)^{1/2}$), and using the general relationship

$$\frac{R_{x\%}}{H_0} = a\xi^{\,b} \qquad \text{(D.2.8-5)}$$

where a and b are constants that depend on the statistic (x%) desired, from Mase (1989).

The *Irregular Wave Runup on Riprap* calculation is part of the *Rubble-mound Revetment Design* module. This method calculates the expected maximum runup elevation and provides a conservative estimate of the maximum runup elevation, based on thesmall-scale laboratory tests of Ahrens and Heimbaugh (1988). The calculations are made given the deepwater significant wave height, peak wave period, and foreshore slope (which yield the surf similarity parameter), and using the general relationship

$$\frac{R_{max}}{H_0} = a\,\xi / (1 + b\,\xi) \tag{D.2.8-6}$$

where *a* and *b* are constants given by Aherns and Heimbaugh (1989).

The *Wave Runup and Overtopping on Impermeable Structures* module calculates the runup elevation associated with incident uniform waves at the structure toe (described by $H_i = H_s$) acting on smooth or rough structures. Other inputs are the peak wave period, nearshore slope, structure slope, and roughness coefficients. The pertinent relationships are:

$$\frac{R}{H_i} = c\,\xi / (1 + d\,\xi) \tag{D.2.8-7}$$

for rough slopes,

and

$$\frac{R}{H_i} = C \tag{D.2.8-8}$$

for smooth slopes,

where *c* and *d* are the armor unit coefficients given by Ahrens and McCartney (1975), and coefficient C varies with the surf similarity parameter ξ , based on the work of Ahrens and Titus (1985).

The ACES runup modules represent improved guidance over that contained in the SPM (USACE, 1984). ACES guidance may be preferable to RUNUP 2.0 in some instances. The *Irregular Wave Runup on Beaches* calculation is maintained in the CEM. The *Irregular Wave Runup on Riprap* calculation is reported to be beneficial because it works well for both shallow water and deepwater at the toe of the revetment.

D.2.8.1.4 Runup on Vertical Structures

Basic empirical guidance incorporated within the RUNUP 2.0 computer model generally does not extend to vertical or nearly vertical flood barriers. For such configurations, RUNUP 2.0 will usually provide a runup elevation, but the result may be misleading because reliance on the composite-slope method can yield an underestimate of actual wave runup with the abrupt barrier. Where a vertical wall exists on a transect, the Mapping Partner shall develop a runup estimate using the specific guidance in Figure D.2.8-3, taken from the *Shore Protection Manual* (USACE, 1984). As within RUNUP 2.0, these empirical results for uniform waves should be used by specifying the mean wave height and mean wave period for entry and taking the indicated runup as a mean value in storm wave action.

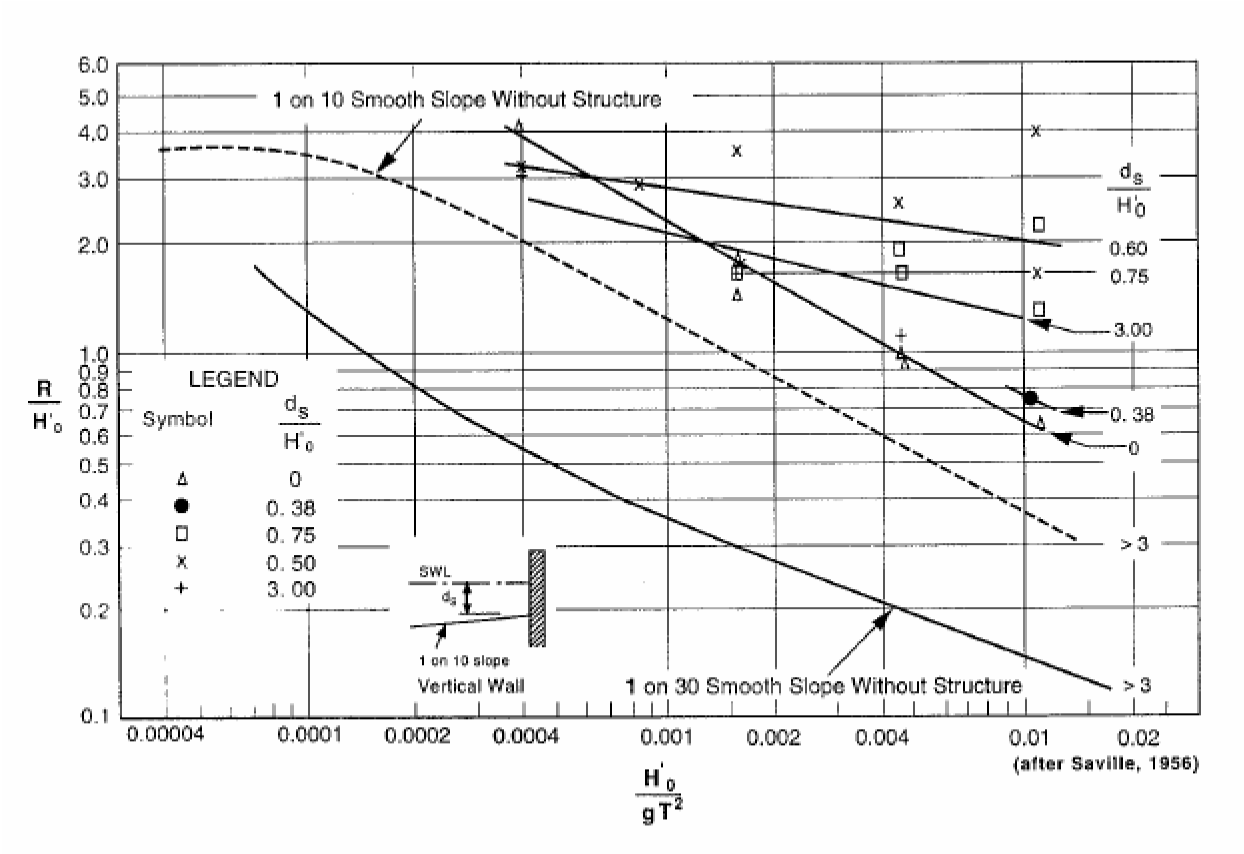


Figure D.2.8-3 Wave Runup Guidance from Vertical Wall, From Shore Protection Manual (USACE, 1984)

D.2.8.1.5 Methodology for Calculating Wave Runup on Barriers

In this subsection, "barriers" include steep dune features and coastal armoring structures, such as revetments. Runup elevations on barriers depend not only on the height and steepness of the incident wave (and its interaction with the preceding wave), but also on the geometry (and construction) of the structure. Runup on structures can also be affected by antecedent conditions resulting from the previous waves and structure composition. Because of these complexities, runup on structures is best calculated using equations developed with tests on similar structures with similar wave characteristics, with coefficients developed from laboratory or field experiments.

The recommended approach to calculating wave runup on structures is based on the Iribarren number (ξ) and reduction factors developed by Battjes (1974), van der Meer (1988), de Waal and van der Meer (1992), and described in the CEM (USACE, 2003). This approach, referred to as the *Technical Advisory Committee for Water Retaining Structures* (TAW) method, is clearly articulated in van der Meer (2002) and includes reduction factors for surface roughness, the influence of a berm, structure porosity, and oblique wave incidence. The TAW method is useful, as it covers a wide range of wave conditions for calculating wave runup on both smooth and rough slopes. In addition to being well documented, the TAW method agrees well with both small- and large-scale experiments.

It is important to note that other runup methods and equations for structures of similar form may provide more accurate results for a particular structure. The Mapping Partner shall carefully evaluate the applicability of any runup method to verify its appropriateness. Figure D.2.8-4 shows a general cross section of a coastal structure, a conceptual diagram of wave runup on a structure, and definitions of parameters.

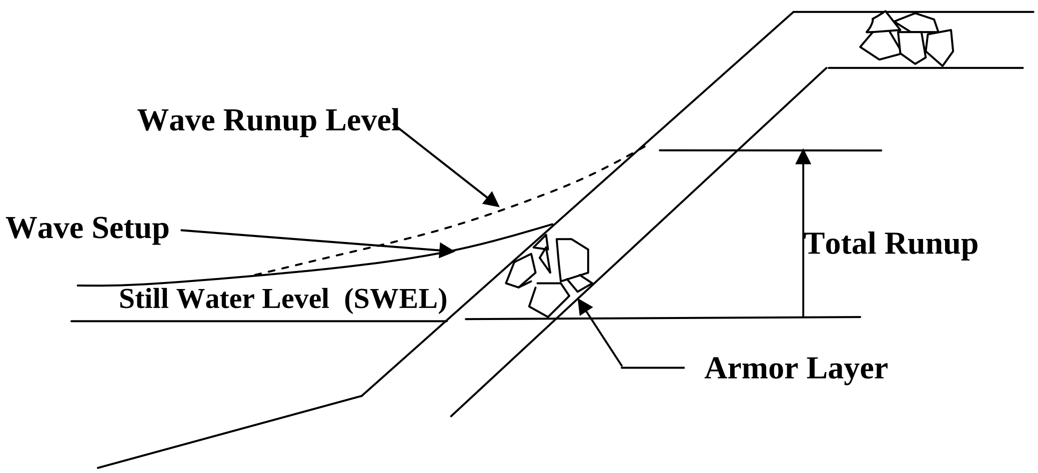


Figure D.2.8-4. Runup on Coastal Structures, Definition Sketch

Most of the wave runup research and literature shows a clear relationship between the vertical runup elevation and the Iribarren number. Figure D.2.8-5 shows the relative runup (R/H_{mo}) plotted against the Iribarren number for two different methods: van der Meer (2002) and Hedges and Mase (2004). In Figure D.2.8-5, both runup equations are derived from laboratory experimental data and are plotted within their respective domains of applicability for the Iribarren number. Each equation shows a consistent linear relationship between the relative runup and ξ_{om} for values of ξ_{om} below approximately 2. For values of ξ_{om} above approximately 2, only the van der Meer method is applicable. Moreover, due to its long period of availability and wide international acceptance, the van der Meer relationship (also referred to as the TAW runup methodology) is recommended here. The Mapping Partner shall characterize the wave conditions in terms of ξ_{om} and be aware of the runup predictions provided by the various methods available in the general literature.

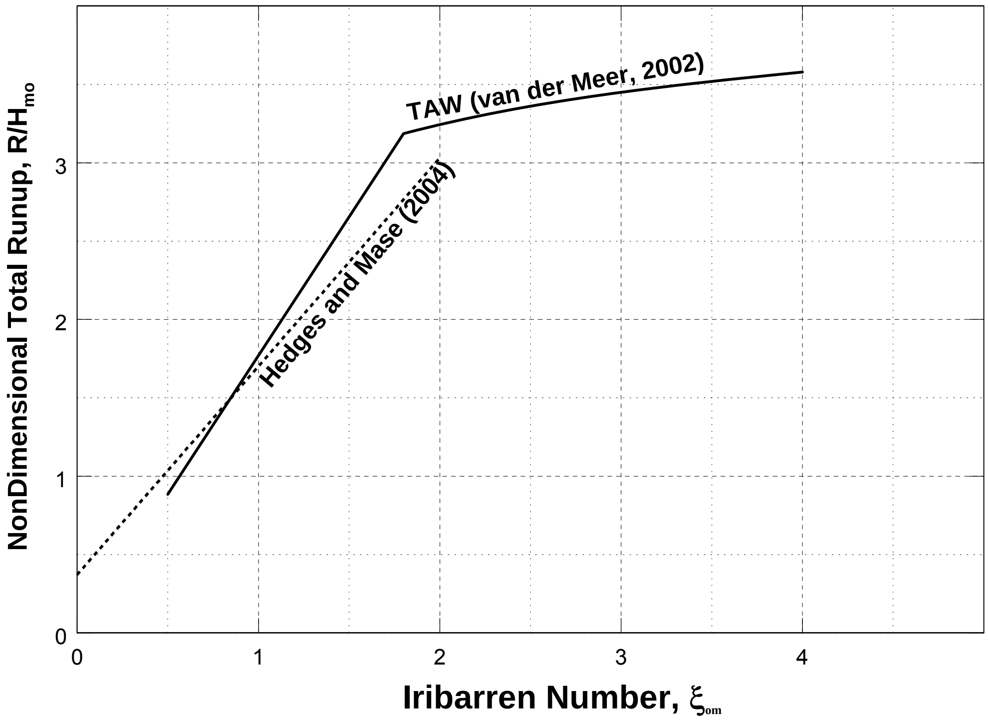


Figure D.2.8-5. Nondimensional Total Runup vs. Iribarren Number

The general form of the wave runup equation recommended for use (modified from van der Meer, 2002) is:

$$R = H_{mo} \begin{Bmatrix} 1.77\gamma_r\gamma_b\gamma_\beta\gamma_P\xi_{om} & 0.5 \le \gamma_b\xi_{om} < 1.8 \\ \gamma_r\gamma_b\gamma_\beta\gamma_P\left(4.3 - \dfrac{1.6}{\sqrt{\xi_{om}}}\right) & 1.8 \le \gamma_b\xi_{om} \end{Bmatrix} \qquad \text{(D.2.8-9)}$$

where:

- R is the 2-percent runup = $2\sigma_2$
- H_{mo}= spectral significant wave height at the structure toe
- γ_r= reduction factor for influence of surface roughness
- γ_b= reduction factor for influence of berm
- γ_β = reduction factor for influence of angled wave attack
- γ_P = reduction factor for influence of structure permeability

Equations for quantifying the γ parameters are presented in Table D.2.8-5. The reference water level at the toe of the barrier for runup calculations is DWL2%. Additionally, because some wave setup influence is present in the laboratory tests that led to Equation D.2.8-9, the following adjustments are made to the calculation procedure for cases of runup on barriers.

Table D.2.8-5. Summary of γ Runup Reduction Factors

Runup Reduction Factor	Characteristic/Condition	Value of γ for Runup	
Roughness Reduction Factor, γ_r	Smooth Concrete, Asphalt, and Smooth Block Revetment	$\gamma_r = 1.0$	(D.2.8-10)
	1 Layer of Rock With Diameter, D. H_s / D = 1 to 3.	$\gamma_r = 0.55$ to 0.60	
	2 or More Layers of Rock. H_s / D = 1.5 to 6.	$\gamma_r = 0.5$ to 0.55	
	Quadratic Blocks	$\gamma_r = 0.70$ to 0.95. See Table V-5-3 in CEM for greater detail	
Berm Section in Breakwater, γ_b, B = Berm Width, $\left(\frac{\pi d_h}{x}\right)$ in radians	Berm Present in Structure Cross section. See Figure D.4.5-8 for Definitions of B, L_{berm} and Other Parameters	$\gamma_b = 1 - \frac{B}{2L_{berm}}\left[1+\cos\left(\frac{\pi d_h}{x}\right)\right], 0.6 < \gamma_B < 1.0$ $x = \begin{cases} R \text{ if } \frac{-R}{H_{mo}} \le \frac{d_h}{H_{mo}} \le 0 \\ 2H_{mo} \text{ if } 0 \le \frac{d_h}{H_{mo}} \le 2 \end{cases}$ (D.2.8-11) Minimum and maximum values of γ_b = 0.6 and 1.0, respectively	
Wave Direction Factor, γ_β, β is in degrees and = 0° for normally incident waves	Long-Crested Waves	$\gamma_\beta = \begin{cases} 1.0, 0 < \lvert\beta\rvert < 10^o \\ \cos(\lvert\beta\rvert - 10^o), 10^o < \lvert\beta\rvert < 63^o \\ 0.63, \lvert\beta\rvert > 63^o \end{cases}$ (D.2.8-12)	
	Short-Crested Waves	$1 - 0.0022\lvert\beta\rvert, \lvert\beta\rvert \le 80^o$ $1 - 0.0022\lvert 80\rvert, \lvert\beta\rvert \ge 80^o$ (D.2.8-13)	
Porosity Factor, γ_P	Permeable Structure Core	$\gamma_P = 1.0$, $\xi_{om} < 3.3$; $\gamma_P = \frac{2.0}{1.17(\xi_{om})^{0.46}}$, $\xi_{om} >$ 3.3 and porosity = 0.5. for smaller porosities, proportion γ_P according to porosity . See Figure D.2.8-7 for definition of porosity (D.2.8-14)	

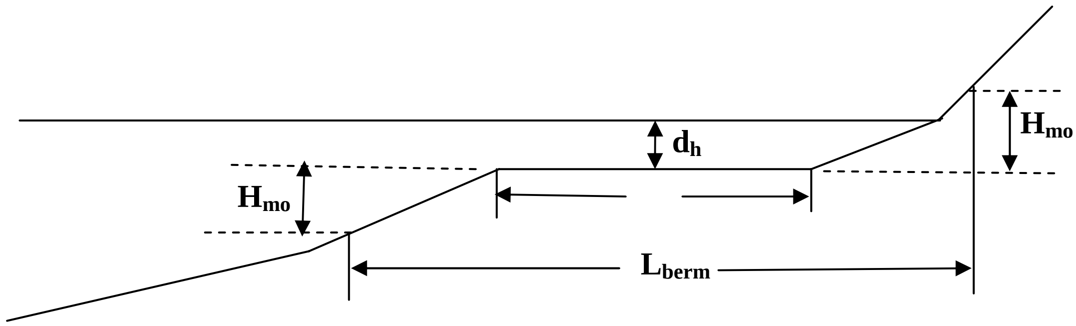


Figure D.2.8-6. Berm Parameters for Wave Runup Calculations

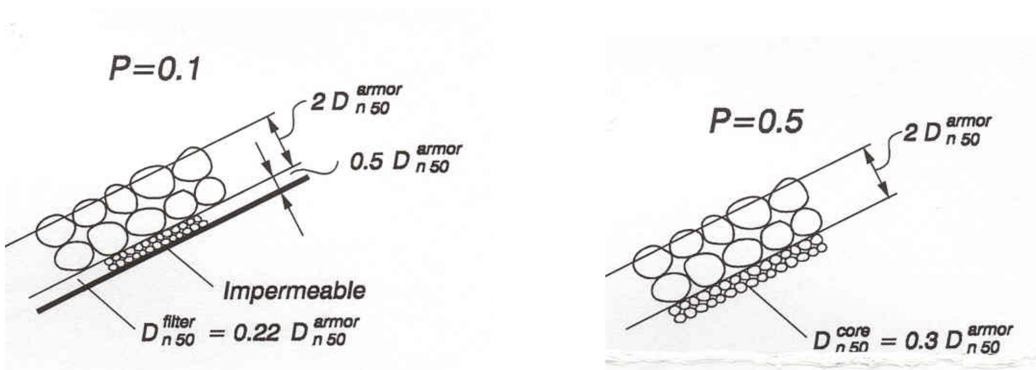


Figure D.2.8-7. Structure Porosity Definition

For a smooth, impermeable structure of uniform slope with normally incident waves, each of the γ runup reduction factors is 1.0.

In calculating the Iribarren number to apply in Equation D.2.8-9, the Mapping Partner shall use Equation D.2.8-2 and replace H_o with H_{mo} and replace T with $T_{m-1.0}$ (the spectral wave period). H_{mo} and $T_{m-1.0}$ are calculated as:

$$H_{mo} = 4.0\sqrt{m_o} \tag{D.2.8-15}$$

$$T_{m-1.0} = \frac{T_p}{1.1} \tag{D.2.8-16}$$

where H_{mo} is the spectral significant wave height at the toe of the structure and T_p is the peak wave period. In deepwater, H_{mo} is approximately the same as H_s, but in shallow water, H_{mo} is 10-to15-percent smaller than the H_s obtained by zero up crossings (van der Meer, 2002). In many cases, waves are depth limited at the toe of the structure, and H_b can be substituted for H_{mo}, with

H_b calculated using a breaker index of 0.78 unless the Mapping Partner can justify a different value. The breaker index can be calculated based on the bottom slope and wave steepness by several methods, as discussed in the CEM (USACE, 2003). In terms of the Iribarren number, the TAW method is valid in the range of $0.5 < \xi_{om} < 8\text{-}10$, and in terms of structure slope, the TAW method is valid between values of 1:8 to 1:1. The Iribarren number as described above is denoted ξ_{om}, as indicated in Equation D.2.8-9.

Runup on structures is very dependent on the characteristics of the nearshore and structure geometries. Hence, better runup estimates may be possible with other runup equations for particular conditions. The Mapping Partner may use other runup methods, based on an assessment that the selected equations are derived from data that better represent the actual profile geometry or wave conditions. See the CEM (USACE, 2003) for a list of presently available methods and their ranges of applicability.

D.2.8.1.6 Runup from Smaller Waves

In some cases, neither of the previously described methods for computing runup on beaches or barriers is applicable. These special cases include steep slopes in the nearshore, with large Iribarren numbers or conditions otherwise outside the range of data used to develop the total runup for natural beach methods. Also, use of the TAW method is questionable where the toe of a structure, or a naturally steep profile such as a rocky bluff, is high relative to the water levels, limiting the local wave height and calculated runups to small values. In these cases, it is necessary to calculate runup with equations in the form of Equation D.2.8-9, to avoid double inclusion of the setup, and to carry out the calculations at several locations across the surf zone using the average slope in the Iribarren number. With this approach, it is possible that calculations with the largest waves in a given sea condition may not produce the highest runup, but that the highest runup will be the result of waves breaking at an intermediate location within the breaking zone.

The recommended procedure is to consider a range of (smaller) wave heights inside the surf zone in runup calculations. The concept of a range of calculated runup values is depicted schematically in Figure D.2.8-8, where an example transect and setup water-surface profile are shown. Figure D.2.8-8 also shows the corresponding range of depth-limited breaking wave heights calculated on the basis of a breaker index and plotted by breaker location on the shore transect. The Iribarren number was also calculated and plotted by breaker location in Figure D.2.8-8. The calculation of ξ at each location uses the deshoaled deepwater wave height corresponding to the breaker height, the deepwater wave length and the average slope calculated from the breaker point to the approximate runup limit. Note that this average slope, also called composite slope as defined in the CEM (USACE, 2003) and SPM (USACE, 1984), increases with smaller waves because the breaker location approaches the steeper part of the transect near the shoreline. This increases the numerator in the ξ equation. Also, the wave height decreases with shallower depths, reducing the wave steepness in the denominator of the ξ equation. Hence, as plotted in Figure D.2.8-8, ξ increases as smaller waves closer to shore are examined, increasing the relative runup (R/H). However, because the wave height decreases, the runup value (R) reaches a maximum and then decreases.

The following specific steps are used to determine the highest wave runup caused by a range of wave heights in the surf zone:

1. Calculate the runup using the methods described earlier for runup on a barrier. This requires iteration for this location to determine the average slope based on the differences between the runup elevation and the profile elevation at the location and the associated cross-shore locations. Iterate until the runup converges for this location.
2. Repeat the runup calculations at different cross-shore locations until a maximum runup is determined.

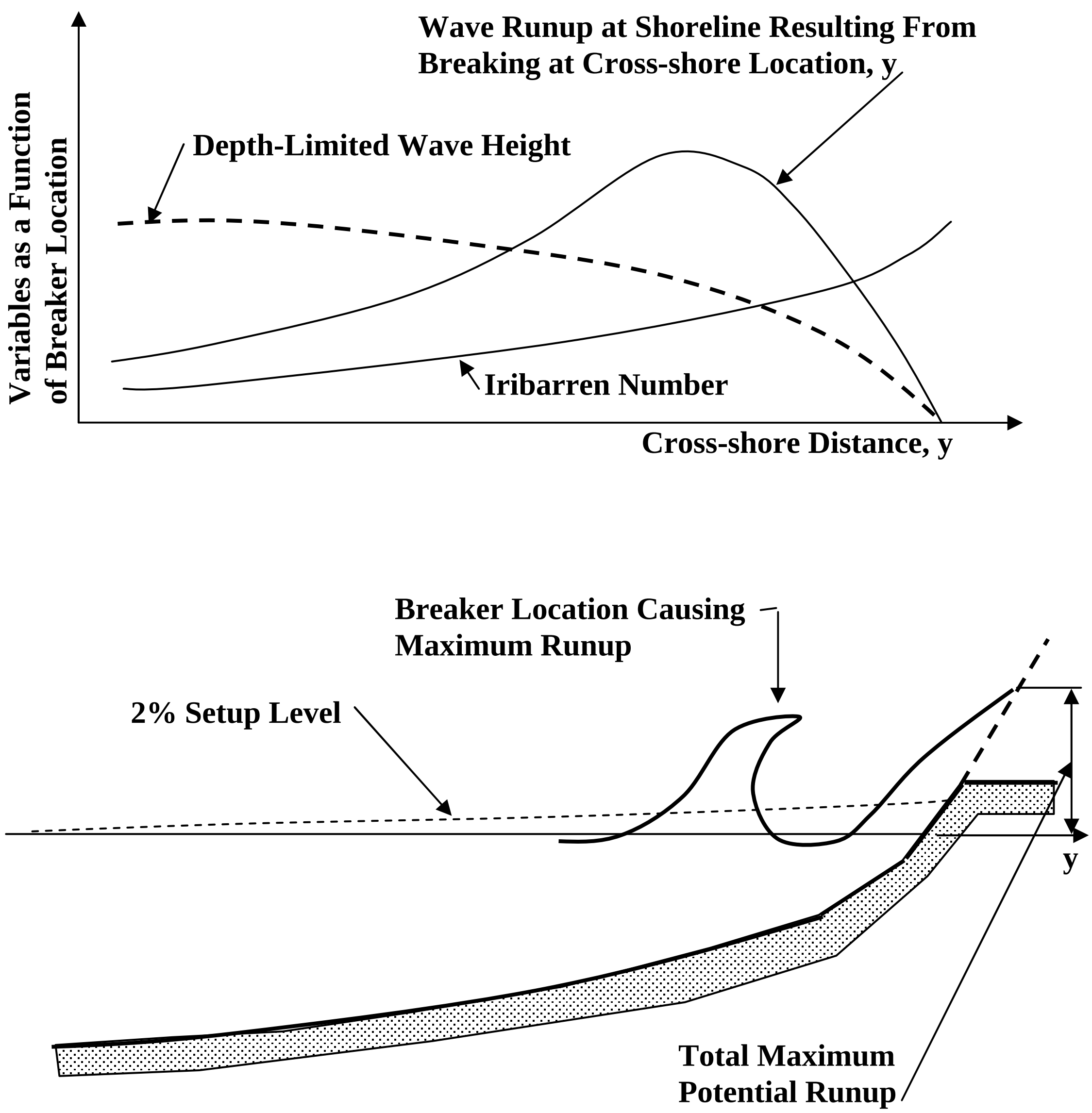


Figure D.2.8-8. Example Plot Showing the Variation of Surf Zone Parameters

D.2.8.1.7 Wave Runup in Special Situations

To interpret and apply the calculated results properly, the Mapping Partner shall examine the output of RUNUP 2.0 carefully for each situation. One important consideration is that a mean runup elevation below the crest of a given barrier does not necessarily imply that the barrier will not occasionally be overtopped by floodwaters (see Subsection D.2.8.2). Other cases may yield results of more immediate concern, in that RUNUP 2.0 may calculate a runup elevation exceeding the maximum barrier elevation; this outcome can occur because the program assumes the last positive slope to continue indefinitely. For bluffs or eroded dunes with negative landward slopes, a general rule has been used that limits the wave runup elevation to 3 feet above the maximum ground elevation, even when the potential runup along the imaginary slope extension exceeds 3 feet. When the runup overtops a barrier, such as a partially eroded bluff or a structure, the floodwater percolates into the bed and/or runs along the back slope until it reaches another flooding source or a ponding area. The runoff areas are usually designated as Zone AO, with a depth of 1, 2, or 3 feet. Ponding areas are designated as Zone AH (depth of 3 feet or less), with BFEs shown. Procedures for the treatment of sizable runoff and ponding are discussed in Section D.2.8.2.4. .

A fairly typical situation on the Atlantic and Gulf coasts is that wave runup exceeds the barrier top and flows to another flooding source, such as a bay, river, or backwater. It may not be necessary in this situation to compute overtopping rates and ponding elevations; only the flood hazard from the runoff must be determined. Simplified procedures have been used to determine an approximate depth of flooding in the runoff area (Williams, 1983). These procedures are illustrated in Figure D.2.8-9 and discussed below.

When the potential runup is at least 3.0 feet above the barrier crest, a VE Zone is delineated landward of the barrier, as shown in Figure D.2.8-9. The BFE for that VE Zone is capped at 3 feet above the crest of the barrier. When the runup depth in excess of the barrier crest is 0.1 to 1.5 feet, the VE Zone BFE is the runup elevation (rounded to the nearest whole foot), and an AO Zone with a depth of 1 foot should be mapped landward until another flooding source is encountered (Zone AE) or the floodplain limit is reached (Zone X). Similarly, for a runup depth of 1.5 to 2.9 feet above the barrier crest, the VE Zone BFE is the runup elevation (rounded to the nearest whole foot). In this case, however, an AO Zone with depth of 2 feet should be mapped, then transitioned landward into an AO Zone with a depth of 1 foot and then into subsequent flood insurance risk zones, if any.

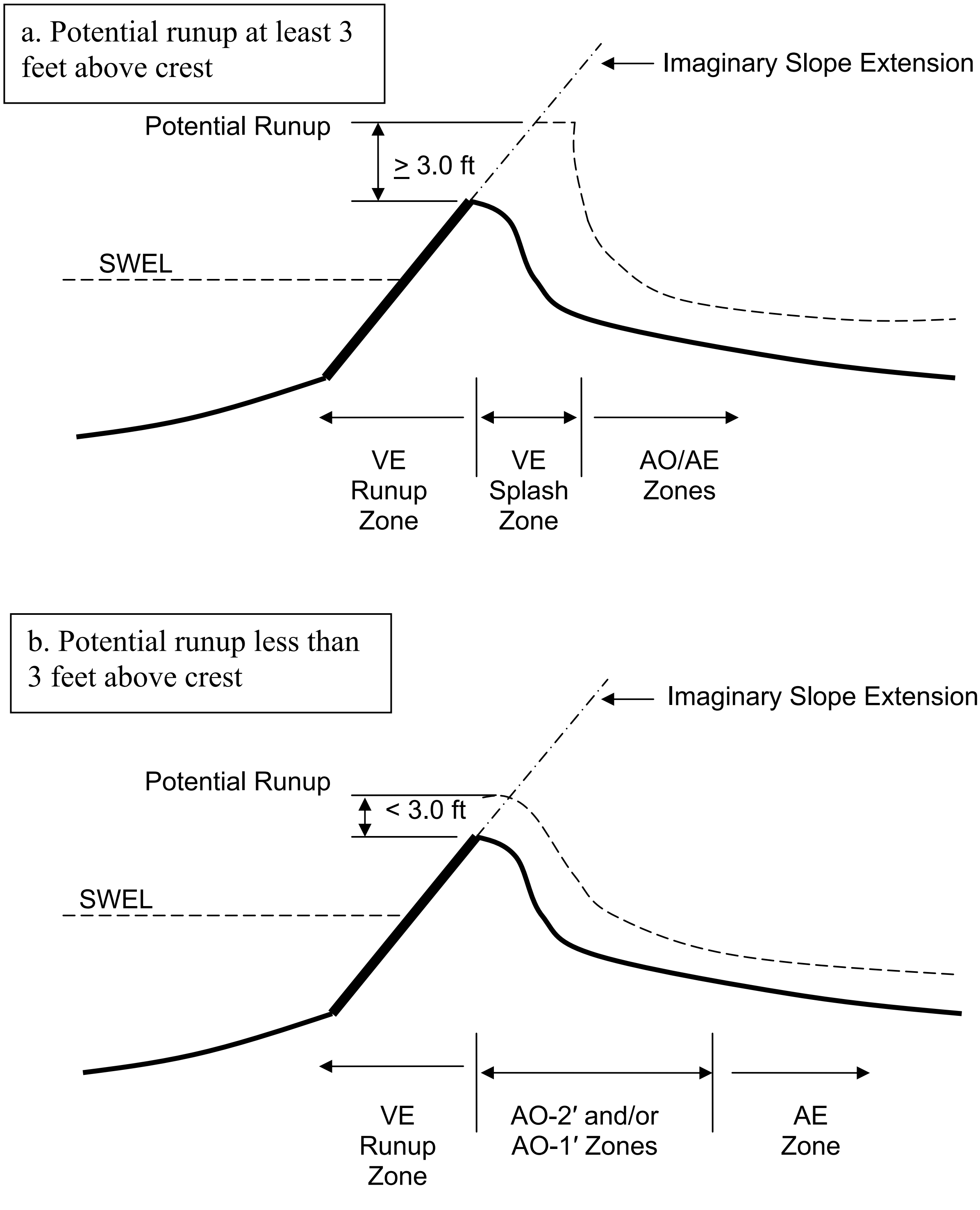


Figure D.2.8-9. Simplified Runoff Procedures (Zone AO)

A distinct type of overflow situation can occur at low bluffs or banks backed by a nearly level plateau, where calculated wave runup may appreciably exceed the top elevation of the steep barrier. A memorandum entitled *Special Computation Procedure Developed for Wave Runup Analysis for Casco Bay, FIS - Maine, 9700-153* provides a simple procedure to determine realistic runup elevations for such situations, as illustrated in Figure D.2.8-10 (French, 1982). An extension to the bluff face slope permits the computation of a hypothetical runup elevation for the barrier, with the imaginary portion given by the excess height R' = (R-C) between the calculated runup and the bluff crest. Using that height (R') and the plateau slope (m), Figure D.2.8-11 defines the inland limit to a wave runup (X) corresponding to the runup above the bluff crest (mX) or an adjusted runup elevation of R_a = (C + mX). This procedure is based on a Manning's "n" value of 0.04, with some simplifications in the energy grade line, and is meant for application only with positive slopes landward of the bluff crest. A different treatment of wave overflow onto a level plateau, for possible Flood Map Project use, is provided in *Overland Bore Propagation Due to an Overtopping Wave* (Cox and Machemehl, 1986).

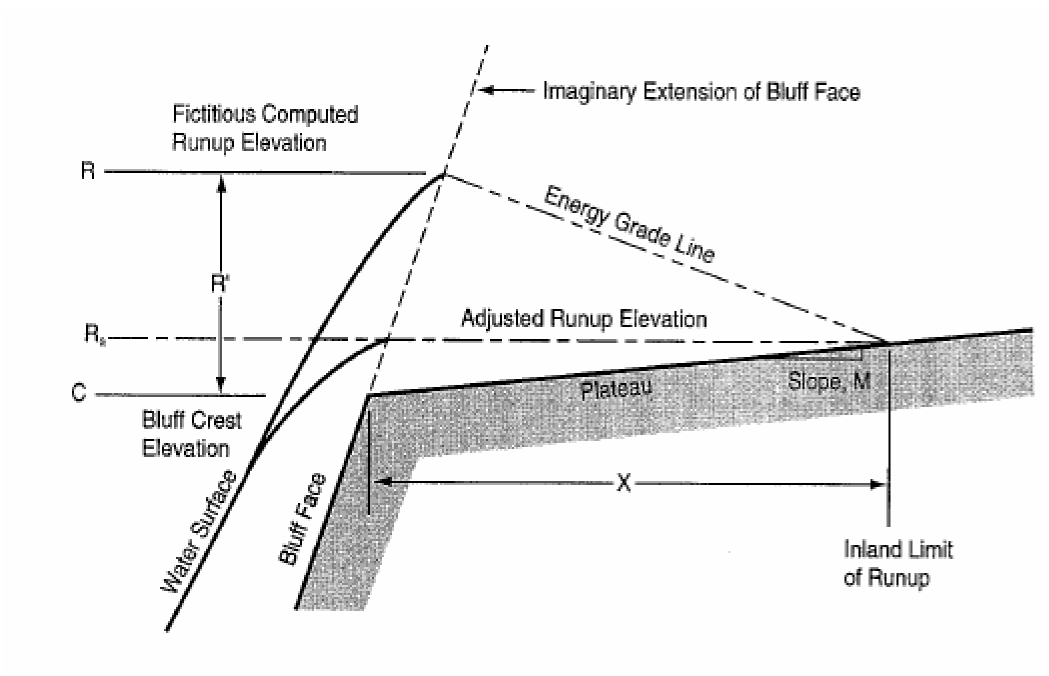


Figure D.2.8-10. Treatment of Runup onto Plateau above Low Bluff

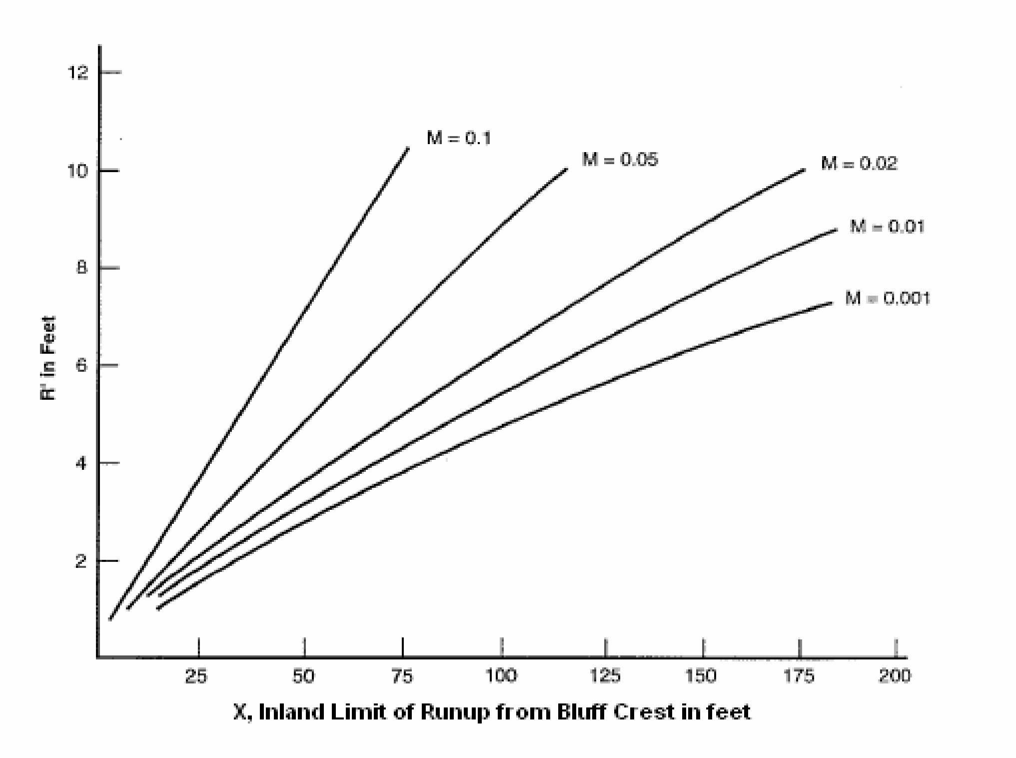


Figure D.2.8-11. Curves for Computation of Runup Inland of Low Bluffs

These runup assessment procedures are given for general guidance, but they may not be entirely applicable in certain situations. For example, runup elevations need to be fully consistent with the wave setup and wave overtopping assessments described in the subsubsections that follow. In problematic cases, the Mapping Partner shall use good judgment and rely on the historical data to reach a solution for the realistic flood hazards associated with a shore barrier. Subsection D.2.11 considers the integration of separately calculated wave effects into coherent hazard zones for the base flood. When a unique situation is encountered, the Mapping Partner shall prepare a Special Problem Report and discuss it with the FEMA Project Representative.

D.2.8.1.8 Advanced Wave Models

Wave models are becoming more sophisticated and able to account for the complexities of water waves. A rapidly developing class of these models is the Boussinesq group, which is both commercially and publicly available. The commercial models are generally more user friendly. In addition to wave setup, Boussinesq models can calculate wave runup. In conjunction with the development of these Guidelines and Specifications, 1-D Boussinesq models have been applied to calculate total wave runup, and the average and oscillating components were calculated separately. The comments below are based on an assessment of these Boussinesq results.

Compared to other methods, Boussinesq models yield generally realistic results. The main concern with Boussinesq modeling is the “learning curve” required to carry out these types of computations with confidence. Additionally, it was difficult to carry out calculations for deepwater waves with a small directional dependency. The reason for this difficulty lies in the associated substantial longshore wave lengths and the need for them to be represented by a 2-D model. One possible FEMA application that would avoid the repeated learning curve requirement would be to carry out computations on a regional basis using Boussinesq models. The rate of the improvement/development of Boussinesq models is moderate at present; however, it is likely that this type of model will be much more capable in 10 to 20 years than at present. Thus, at this stage, a Mapping Partner may elect to apply Boussinesq models; however, for application on a regional basis, it is preferable to wait for further developments and improvements. If a Boussinesq model is applied, the Mapping Partner shall obtain FEMA's approval, and it is suggested that calculations also be carried out using the DIM methodology for comparison of results.

With these more advanced wave models, the wave setup component is combined with the storm-surge modeling, resulting in SWELs that include both storm surge and wave setup. Care must be taken not to double count wave setup when calculating wave runup with one of the methods presented here. The wave runup methods are based on scale laboratory tests that are thought to include wave setup, so that calculated runup values shall be added to the storm surge, excluding wave setup.

D.2.8.1.9 Documentation

The Mapping Partner shall document the procedures and values of parameters employed to establish the 1-percent-annual-chance total wave runup on the various transects on natural beaches and barriers, which could include steep dunes and structures. In particular, the basis for establishing the runup reduction factors and their values shall be documented. The documentation shall be especially detailed if the methodology deviates from that described herein and/or in the recommendations of the supporting documentation. Any measurements and/or observations shall be recorded, as well as documented or anecdotal information regarding previous major storm-induced runup. Any notable difficulties encountered and the approaches to addressing them shall be described clearly. Additional information on required documentation criteria can be found in Subsection D.2.12.

D.2.8.2 Overtopping (Open Coast and Sheltered Waters)

D.2.8.2.1 Overview.

Wave overtopping occurs when a barrier crest height is lower than the potential wave runup level, as shown in Figure D.2.8-12. Waves will flow or splash over the barrier crest, typically to an elevation less than the potential runup elevation (R′). The exact overtopping water surface and overtopping rate will depend on the incident water level and wave conditions and on the barrier geometry and roughness characteristics. Moreover, overtopping rates can vary over several orders of magnitude, with only subtle changes in hydraulic and barrier characteristics, and are difficult to predict precisely.

The assessment of potential wave overtopping for flood hazard mapping purposes must rely on readily available empirical guidance, historical effects, and engineering judgment. Except for very heavy overtopping, useful guidance is typically derived from laboratory tests with irregular waves, because the intermittently large overtopping discharges in storm situations are difficult to reproduce in the laboratory. Recent numerical modeling and field experiments are advancing the state of the art in overtopping predictions, but applying those methods in routine flood hazard mapping purposes is still problematic. Therefore, the Mapping Partner shall estimate only the order of magnitude of mean overtopping rates, because there are clearly documented thresholds below which wave overtopping may be classified as negligible. While this approach does not account explicitly for highly variable peak overtopping rates and does not offer a complete specification of overtopping hazards, its use is recommended until overtopping rate calculation guidance is improved significantly.

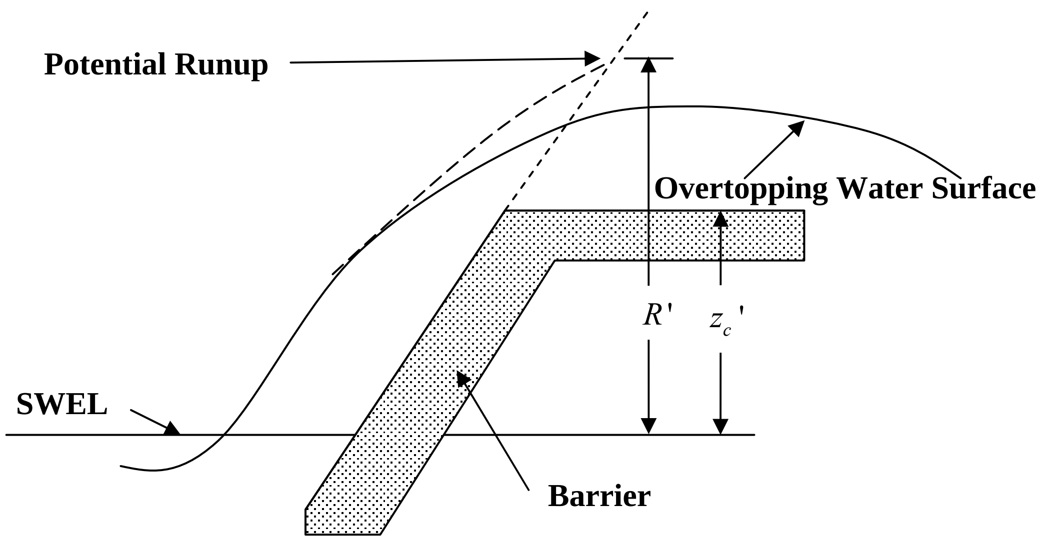


Figure D.2.8-12. Definition Sketch for Wave Overtopping

If a preliminary estimate indicates severe overtopping that threatens the stability of a given structure, that structure might be removed from the transect for analyses of the base flood, and further overtopping consideration may not be required. Two publications, Design of Seawalls Allowing for Wave Overtopping (Owen, 1980) and Random Seas and Design of Maritime Structures (Goda, 1985), appear to provide trustworthy and wide-ranging summaries of mean overtopping rates with storm waves. The former publication addresses smooth-plane or bermed slopes, and the latter publication considers vertical walls with or without a fronting rubble mound. Before surveying those primary sources of overtopping guidance, however, some introductory considerations can help to determine whether a detailed wave overtopping assessment is needed for base flood conditions at a specific shore barrier.

The initial consideration is an interpretation of the mean runup elevation already calculated ($\bar{R}$), in terms of likely extreme elevations according to the Rayleigh probability distribution usually appropriate for wave runups. To parallel the extreme wave height addressed in coastal studies (NAS, 1977), a controlling (base flood) runup magnitude may be defined as 1.6 times the significant runup, or 2.5 times the mean runup, according to the Rayleigh distribution.

The first overtopping calculation the Mapping Partner should make is a comparison of the potential mean runup ($\bar{R}$) to the freeboard (F) offered by the barrier. If the elevation of the barrier crest above the base flood total stillwater (MWL) elevation (freeboard) equals or exceeds

2.5 $\overline{R}$, then the landward area is considered not subject to wave overtopping discharges during the base flood. If F ≤ (2.0 $\overline{R}$), then wave overtopping can be appreciable during the base flood, and the Mapping Partner shall assess overtopping rates and potential ponding behind the barrier. The extreme runups introduced here (2.0 $\overline{R}$ and 2.5 $\overline{R}$) bracket the elevation exceeded by the extreme 2 percent of wave runups, which is a value commonly considered in structure design[9].

D.2.8.2.2 Mean Overtopping Rates

Once the need for quantitative overtopping assessment is established, wave overtopping estimates for a specified situation must generally be based on measurements in a similar configuration. Before considering some implications of quantitative guidance for idealized cases, an overview of overtopping magnitudes gives a useful introduction (Goda, 1985; Gadd *et al.*, 1984).

Wave overtopping is often specified as a mean discharge: water volume per unit time and per unit alongshore length of the barrier, commonly in cubic feet per second per foot (cfs/ft). By interpreting or visualizing a given mean overtopping rate, the Mapping Partner may take into account actual discharges that are generally intermittent and isolated, being confined to some portion of occasional wave crests at scattered locations.

Distinct regimes of wave overtopping may be described as spray, splash, runup wedge, and waveform transmission, in order of increasing intensity. Flood discharges corresponding to those regimes naturally depend on the incident wave size, but certain overtopping rates have been associated with various characteristics (Goda, 1985). The right axis of Figure D.2.8-13 shows this association.

The mean overtopping rate of 0.01 cfs/ft seems to correspond to a value that generally should be considered appreciable, and a 1-cfs/ft mean overtopping rate appears to define an approximate threshold where the structural stability of even well-constructed shore barriers becomes threatened by severe overtopping. The 1-cfs/ft mean overtopping rate also appears to be well within the range where buildings exposed to overtopping are damaged.

Figure D.2.8-13 summarizes some empirical overtopping guidance for storm waves, in a schematic form meant to help Mapping Partners determine the likely significance of flooding behind a coastal structure. Variables describing the basic situation are cotangent of the front slope for a smooth structure with ideally simple geometry, and freeboard of the structure crest above total stillwater (mean water) level, as normalized by incident significant wave height (F/H_s). The mean overtopping rate ($\overline{Q}$) is provided in dimensionless form as

$$Q^* = \overline{Q}/(gH_s^3)^{0.5} \qquad \text{(D.2.8-17)}$$

with test results shown for structure slopes of 1:1, 1:2, and 1:4 (Owen, 1980), and for a smooth vertical wall (Goda, 1985). These results pertain to significant wave steepness of approximately $2\pi H_s/gT_p^2 = 0.035$, fairly appropriate for extreme extratropical storms or hurricanes; water depth near the structure toe of approximately $d_t = 2H_s$, so that incident waves are not appreciably

[9] According to Walton (1992), the Rayleigh distribution would result in a 2-percent wave runup height that is approximately 2.2 times the mean runup and a maximum wave runup height (for levee analyses) that is approximately 2.9 times the mean wave runup.

attenuated; and moderate approach slopes of 1:30 for a vertical wall or 1:20 for other structures. The major feature of interpolated curves is fixed as a maximum in overtopping rate for a structure slope of 1:2, corresponding to the gentlest incline producing (at this wave steepness) total reflection rather than breaking, and thus peak waveform elevations (Nagai and Takada, 1972).

These measured results for smooth and simple geometries clearly show severe or "green water" overtopping even at relatively high structures ($F \geq H_s$) for a wide range of common inclinations (cotangents between 0 and 4). Also, for freeboards considered here, a vertical wall (cotangent 0) permits less overtopping than common sloping structures with cotangent less than approximately 3.5. Gentler barriers are uncommon, because the construction volume increases with the cotangent squared, so steep coastal flood-protection structures usually face attenuated storm waves and/or have rough surfaces. The basic effects of those differences can be outlined for use in simplified overtopping assessments.

For sloping structures sited within the surf zone ($d_t < 2H_s$), *Design of Seawalls Allowing for Wave Overtopping* indicates that basic overtopping guidance in Figure D.2.8-13 can be used with attenuated rather than incoming wave height (Owen, 1980). A simple estimate basically consistent with other analyses of the base flood is that significant wave height is limited to

$H_s' = d_t/2$at the structure toe. The value of $2F/d_t$ describes the effectively increased freeboard in entering Figure D.2.8-13, and the indicated Q* value is then converted to $\overline{Q}$ using H'_s. The presumed wave attenuation ignores any wave setup as a small effect with the partial barrier, and d_t should always correspond to the scour condition expected in wave action accompanying the base flood.

Figure D.2.8-13 might also be made applicable to rough slopes, using a roughness coefficient (r) from Table D.2.8-1 to describe the effectively increased freeboard with greater wave dissipation on the structure. *Design of Seawalls Allowing for Wave Overtopping* proposed formulating effect of structure roughness as *F/r*, and *Beach and Dune Erosion during Storm Surges* confirmed a similar dependence of overtopping on roughness in measured results for irregular waves (Owen, 1980; Vellinga, 1986). The overtopping relation reported as reliable in *Wave Runup and Overtopping on Coastal Structures* is

$$Q^* = 8 \bullet 10^{-5} \ \exp[3.1(rR^* - F / H_s)] \qquad \text{(D.2.8-18)}$$

where $R_* = [1.5\ m/(H_s/L_{op})^{0.5}]$, up to a maximum value of 3.0, is an estimated extreme runup normalized by Hs, for a barrier slope given as the tangent m (de Waal and van der Meer, 1992). Equation 3 is meant to pertain to very wide ranges of test situations with moderate overtopping, but it appears very approximate in comparison with specific results for $r = 1$, shown in Figure D.2.8-13. It may be advisable to evaluate Equation D.2.8.1-18 for both smooth and rough barriers, then to use the ratio to adapt a value from Figure D-19 for the case with roughness. *Design of Seawalls Allowing for Wave Overtopping* (Owen, 1980) and *Wave Runup and Overtopping on Coastal Structures* (de Waal and van der Meer, 1992) provide further overtopping guidance on the effects of composite profiles, oblique waves, and shallow water with sloping structures.

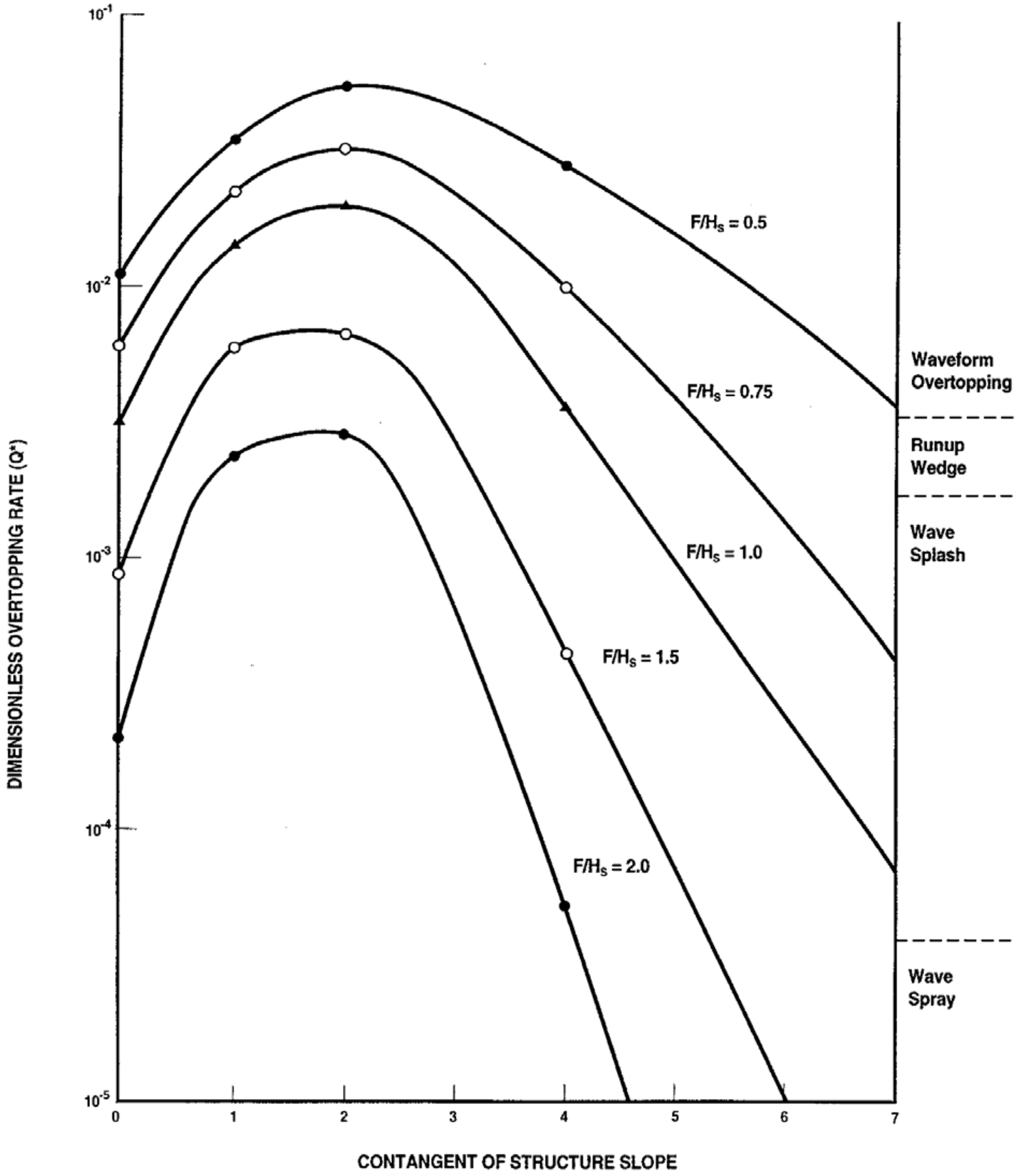


Figure D.2.8-13. Schematic Summary of Storm-Wave Overtopping at Structures of Various Slopes and Freeboards,

For overtopped vertical walls, the effects of wave attenuation appear relatively complex, but *Random Seas and Design of Maritime Structures* (Goda, 1985) provides extensive empirical guidance on various structure situations with incident waves specified for deepwater. Figure D.2.8-14 converts basic design diagrams for wave overtopping rate at a vertical wall, to display wall freeboard required for rates of 1 cfs/ft and 0.01 cfs/ft with various incident wave heights. Goda (1985) also provides a convenient summary on the effect of appreciable fronting roughness in storm waves: the required freeboard of a smooth vertical wall for a given overtopping rate is approximately 1.5 times that needed when a sizable mound having concrete block armor is installed against the wall. With this information, a specific vertical wall can be categorized as having only modest overtopping ($\overline{Q}$< 0.01 cfs/ft), intermediate overtopping, or severe overtopping ($\overline{Q}$ > 1 cfs/ft) expected for the base flood. Likely runoff or ponding behind the wall must then be identified; severe overtopping requires a delineation of the landward area susceptible to wave action and velocity hazard.

Considering Figure D.2.8-14 with respect to common wall and wave heights, wave overtopping that endangers structural stability appears usual during the base flood.

An assessment of failure during the base flood for typical walls would be fully consistent with one recommendation of *Criteria for Evaluating Coastal Flood-Protection Structures*, which states that "FEMA not consider anchored bulkheads for flood-protection credit because of extensive failures" (Walton et al., 1989).

D.2.8.2.3 Overtopping Rate Considerations for Establishing Flood insurance risk zones

An interpretation of the estimated overtopping rate in terms of flood hazards is complicated by the projected duration of wave effects, the increased discharge possible under storm winds, the varying inland extent of water effects, and the specific topography and drainage landward of the barrier. However, Table D.2.8-6 provides guidance that is potentially applicable to typical coastal situations.

Table D.2.8-6. Suggestions for Interpretation of Mean Wave Overtopping Rates

$\overline{Q}$ **Order of Magnitude**	**Flood insurance risk zone Behind Barrier**
<0.0001 cfs/ft	Zone X
0.0001-0.01 cfs/ft	Zone AO (1 ft depth)
0.01-0.1 cfs/ft	Zone AO (2 ft depth)
0.1-1.0 cfs/ft	Zone AO (3 ft depth)
>1.0 cfs/ft*	30-ft width+ of Zone VE (elevation 3 ft above barrier crest), landward Zone AO (3 ft depth)

*With estimated $\overline{Q}$ much greater than 1 cfs/ft, removal of barrier from transect representation may be appropriate.
+Appropriate inland extent of velocity hazards should take into account barrier characteristics, incident wave conditions, overtopping flow depth and velocity, and other factors.

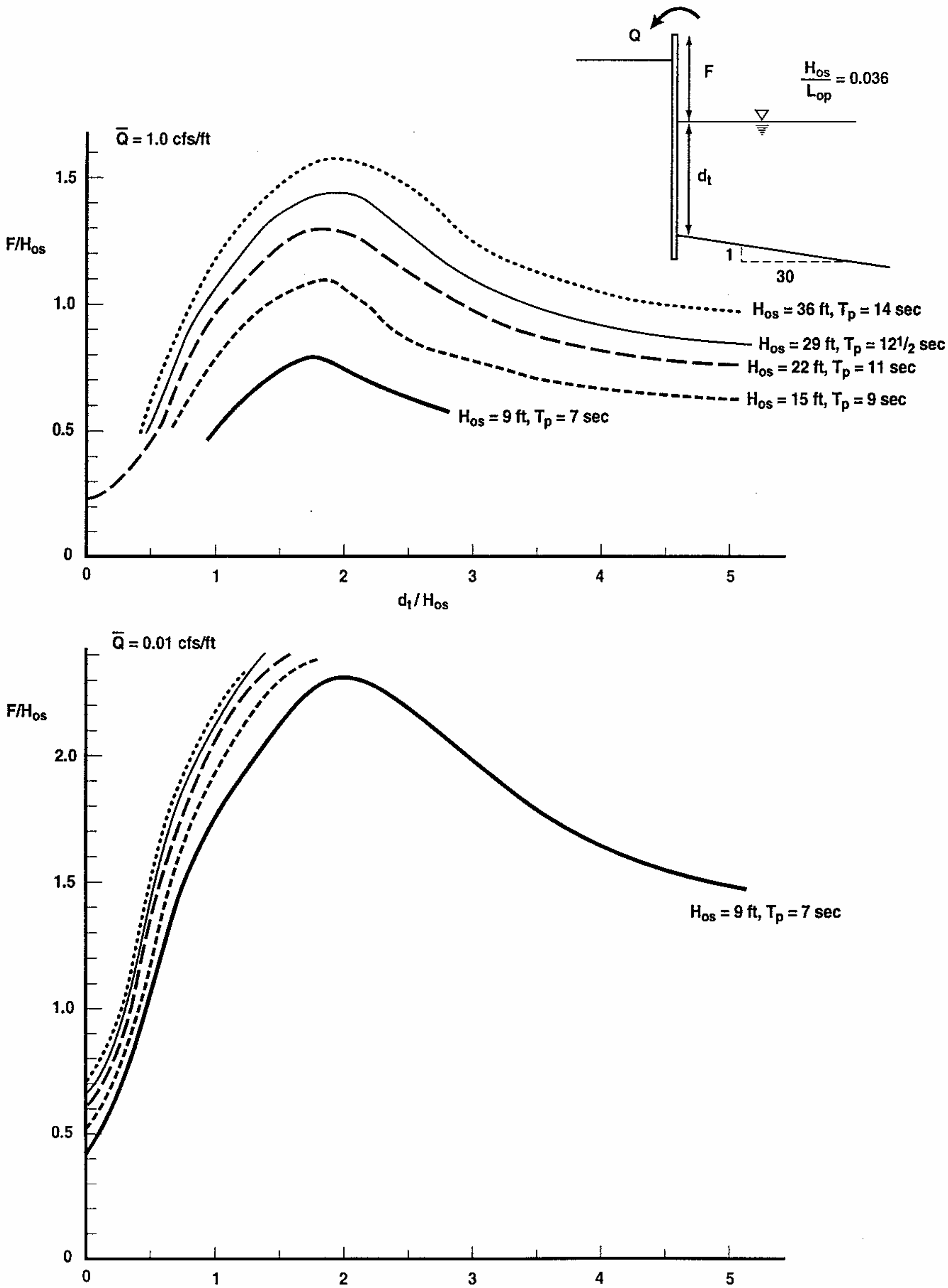


Figure D.2.8-14. Required Freeboard of Vertical Wall to Limit Mean Overtopping Rate to Certain Values, Based on Design Curves of Random Seas and Design of Maritime

D.2.8.2.4 Ponding Considerations

Once the mean overtopping rate has been estimated for the base flood, determining the resultant flooding landward of the barrier will require the Mapping Partner to evaluate several parameters, including the duration of overtopping, topography, and drainage landward of the overtopped barrier. By integrating the volume of overtopping (mean rate times duration) and comparing this to the available storage landward of the barrier, an estimated ponding elevation can be determined. This elevation should be adjusted by the Mapping Partner depending upon rainfall rates associated with the overtopping event, drainage features and systems landward of the barrier, and crest elevations of any features that may allow ponded water to escape. Ponding assumptions and calculations should be reviewed carefully to ensure that overtopping and other potential sources of water trapped behind the barrier are accounted for appropriately.

The duration of overtopping can vary widely, depending on the coastal flood cause, from a fast-moving hurricane to a nearly stationary extratropical storm. The final guidance is offered: a minimum assumption for the duration of flood-peak overtopping would generally be 2 hours. Durations of 10 hours or more could be appropriate for the cumulative effects of an extratropical storm causing flooding over multiple high tides.

D.2.8.2.5 Overtopping Depth and Velocity Considerations

In cases where the potential runup exceeds a barrier crest by 3.0 feet or more, the Mapping Partner will map a VE splash zone landward of the crest (see Subsection D.2.8.1.7). The Mapping Partner may consider the overtopping depth and velocity as one factor to determine the landward limit of the VE splash zone.[10] .

[10] This new mapping procedure was introduced in the *Pacific Mapping Guidelines*. More details are provided in Sections D.4.9.2.1 and D.4.5.2.5.

D.2.9 Coastal Erosion

This subsection provides methods for Mapping Partners to define the shape and location of eroded beach profiles, upon which 1-percent-annual-chance flood conditions (waves and water levels) will act and from which flood insurance risk zones and BFEs will be mapped.

D.2.9.1 Overview

Erosion processes and consequences of erosion can either be "episodic" or "chronic." These two descriptors assign a very important temporal component to erosion processes and their results. *Episodic erosion* is the shore and backshore adjustment that results from short duration, high intensity meteorologic and oceanic storm events. This type of event response results in shore adjustment and occurs during a single storm or during a series of closely spaced storm events within a storm season. Shore and backshore profile changes during intense storms and hurricanes can result in dramatic beach and dune erosion, retreat, breaching, or removal of backshore dunes; cause retreat and collapse of bluff and cliff formations; and culminate in greater landward encroachment of waves and flooding from the ocean. *Chronic erosion* is associated with slow, long-term processes such as gradual shoreline adjustment associated with: (1) sea-level rise, (2) land subsidence, (3) changes in sediment supply due to watershed modifications or dam building, and (4) decadal adjustments in rainfall, runoff, and wave climate associated with global warming.

Current FEMA regulations are limited to risks and losses occurring as the direct result of a storm event. The NFIP does not address long-term chronic erosion, but focuses on episodic, flood-related erosion due to coastal storm events.[11] FEMA does not currently map long-term erosion hazard areas as some local or State agencies do. FEMA FIRMs do not inform property owners of erosion risks. FIRMs only indicate risks from flooding hazards in the form of BFEs and flood insurance risk zones. Therefore, flood assessment guidelines in this subsection only include methods for estimating eroded shore and backshore profiles during single, large storm events; the resulting profiles are then used in runup and overtopping computations to determine flood risks associated with these events. Subsection D.2.8 discusses how results from event-based erosion assessments are to be used by Mapping Partners to determine flood risks and delineate hazard zones.

D.2.9.2 Atlantic Coast Characteristics Related to Storm-Induced Erosion

Atlantic Coast beaches undergo typical seasonal changes in profile and location from summer to winter conditions. During winter months, increased total water levels, along with high-energy, steep waves, tend to move sand offshore. Throughout the summer and early fall, during months of calm seas, the beach recovers and the berms and dunes rebuild as sand moves back onshore. Figure D.2.9-1 provides a sketch of generalized, seasonal profile changes that occur on sand beaches of the Atlantic Coast.

[11] Discussions of long-term erosion and the potential consequences of chronic erosion are found in materials listed in the reference section of this document and in many of the support documents referenced herein.

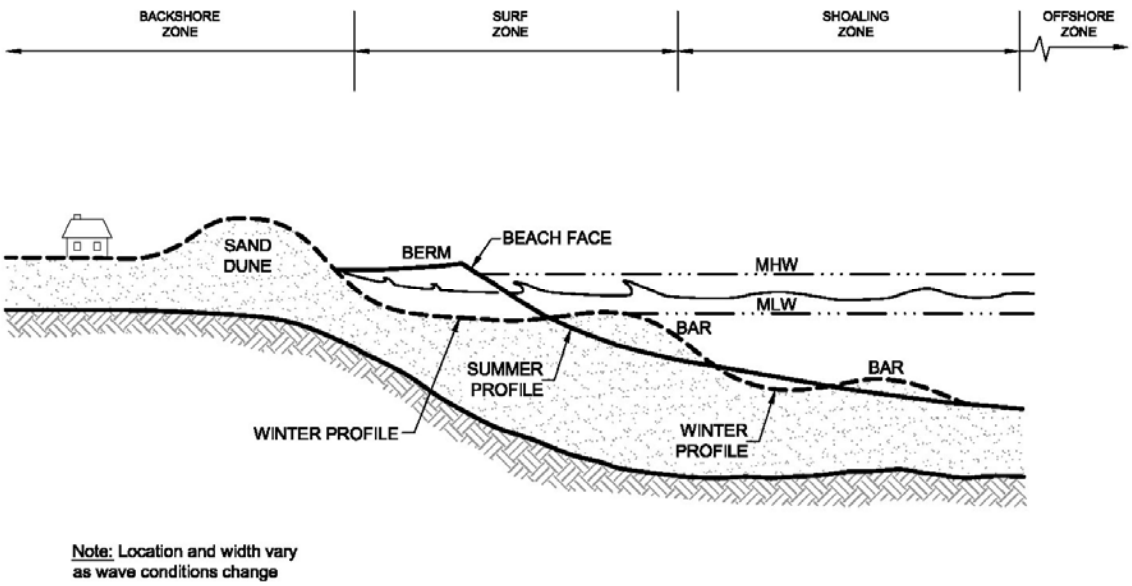


Figure D.2.9-1. Typical Atlantic Coast Summer and Winter Beach Profiles (after Bascom, 1964)

To estimate beach erosion and profile changes for a specific coastal setting, it may be important to consider during which season the potential flooding hazard event will likely occur. Many Atlantic and Gulf sandy beaches exhibit significant seasonal changes in their profiles due to seasonal differences in weather and wave climate. As a result, physical aspects such as beach width, berm height, and dune size can be reduced significantly at certain times of year. The severity of flooding associated with a given storm can be much greater if that storm strikes when the beach is in its seasonally eroded condition. To the extent practicable and where appropriate, Mapping Partners should consider the impacts of seasonal beach profile variability and the timing of severe storms when assessing flood hazards.

Northeasters can be particularly dangerous storms because they tend to occur during the time of year when beaches are in their most depleted or eroded condition. When determining the flooding and erosion hazard that a northeaster poses, it is important first to estimate the initial beach profile conditions that exist just before the occurrence of the storm (Figure D.2.9-2). Where significant storms occur during the winter, it may be appropriate for the Mapping Partner to consider use of the Most Likely Winter Profile (MLWP) approach. The concept of MLWP and how it can be used in assessing storm-induced erosion in appropriate study settings is discussed in greater detail in Subsection D.4.6.4. Mapping Partners conducting studies in areas subject to northeasters should consider estimating an MLWP prior to determining beach profile changes for a particular winter storm event. Generally, beaches south of Virginia are seldom affected by northeasters and therefore the MLWP approach is unlikely to be helpful when assessing coastal flood hazards.

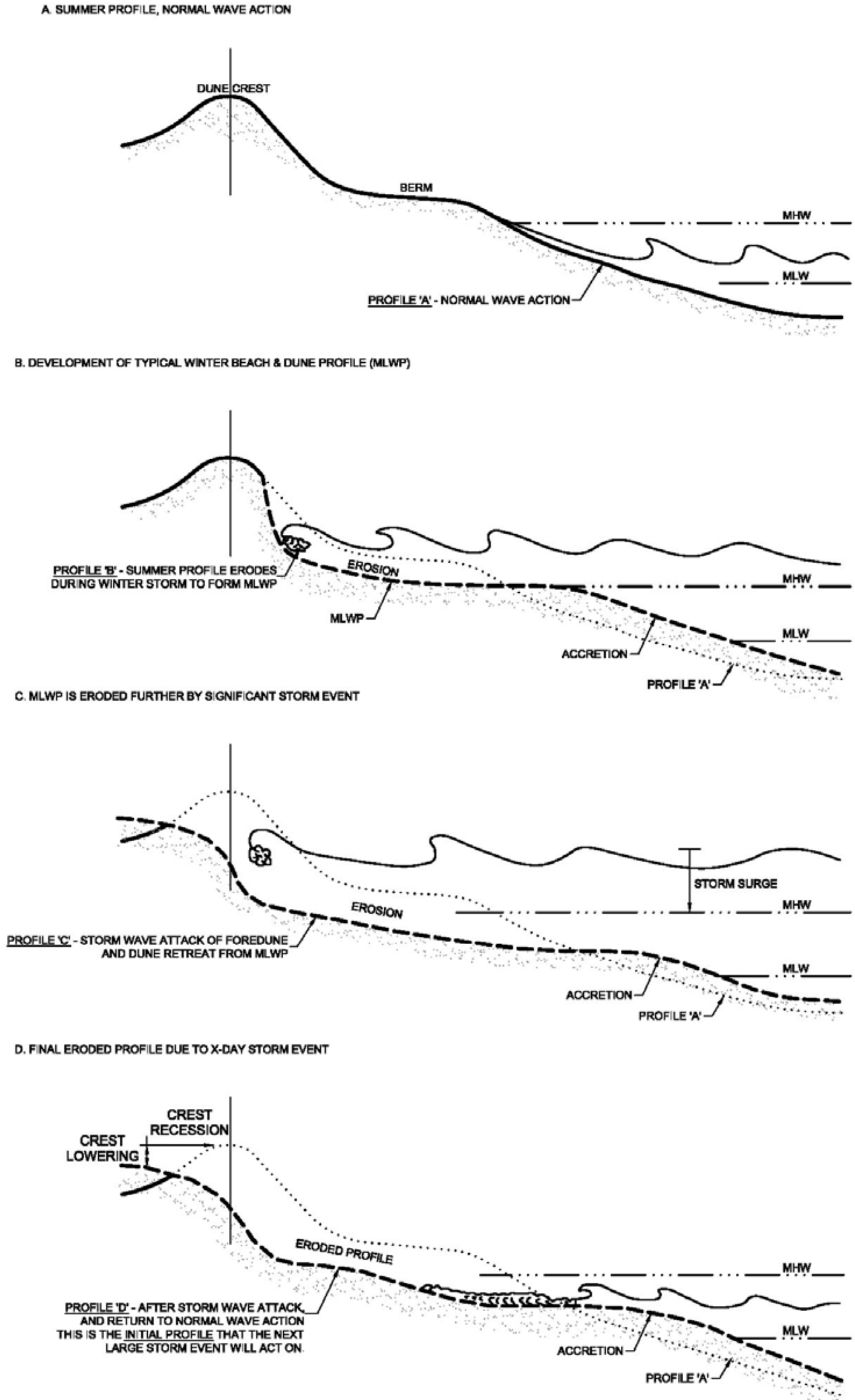


Figure D.2.9-2. Evolution of the Initial Beach Profile Before Occurrence of Large Storm Event (after SPM, 1984)

The following list provides references and websites where pertinent data may be obtained for use in event-based erosion analyses:

- Barton, C. C. 2004. U.S. National Coastal Assessment, USGS, Geologic Division, St. Petersburg, FL, website: <http://coastal.er.usgs.gov/national_assessment/>.

- Carr, E. E. 2002. Database of Federal Inlets and Entrances. U.S. Army Corps of Engineers, Coastal Inlets Research Program. <http://cirp.wes.army.mil/cirp/inletsdb/inletsdbinfo.html>. June 19.

- Good, J. W. (ed.). 1992. Coastal Natural Hazards, Science, Engineering and Public Policy. Oregon Sea Grant, Oregon State University, Publication No. ORESU-B-92-001, 162 pages.

- NOAA. 2000a. Tidal Datums and Their Applications. NOAA Special Publication NOS CO-OPS 1, Silver Spring, MD. June. <http://www.co-ops.nos.noaa.gov/publications/tidal_datums_and_their_applications.pdf>.

- NOAA. 2000b. Nautical Chart Symbols, Abbreviations and Terms, Chart No. 1, Eleventh Edition. Lighthouse Press, Annapolis, MD. 99 pp. NOAA Nautical Chart Users Manual. <http://chartmaker.ncd.noaa.gov/staff/ncum/ncum.htm>.

- NOAA. 2003. Computational Techniques for Tidal Datums Handbook, NOAA Special Publication NOS CO-OPS 2, Silver Spring, MD. September. <http://www.co-ops.nos.noaa.gov/publications/Computational_Techniques_for_Tidal_Datums_handbook.pdf>.

- Links to Other Information Sites Regarding Coastal Zone Management Topics:

  - <http://coastal.er.usgs.gov/lidar/AGU_fall98/>: Coastal Erosion (NOAA).

  - <http://geodesy.noaa.gov/RSD/coastal/cscap.shtml>: Remote Sensing (NOAA).

  - <http://gis.sfsu.edu/data.htm>: GIS Data Bases for Various Types of Data.

  - <http://www.csc.noaa.gov/shoreline/>: Shoreline Data (NOAA).

  - <http://www.csc.noaa.gov/crs/tcm/missions.html>: Topographic Data (NOAA).

D.2.9.3 Description of Beach Settings and Erosion Assessment Procedures

By their nature, coastlines are extremely complex and dynamic environments. The type and magnitude of coastal erosion are closely related to general coastal exposure and beach setting.

Coastal exposure refers to: (1) whether the coastline and beach are situated on the open coast, i.e., exposed to the undiminished waves, water levels, tides, winds, and currents associated with the open coast, or (2) whether the coastline is located within a sheltered area that is fully or partially protected from the direct action of ocean waves, winds, tides, water levels, and currents. The latter condition is referred to as a *sheltered water area*. Beach erosion processes resulting from changes in total water level and wave action are similar along the open coast and within sheltered water areas; however, the magnitude, rate, and ultimate beach response may be quite different. Sheltered water areas typically have reduced wave energy and smaller runup. Some sheltered water areas found in confined embayments or estuaries may, however, experience higher still-water elevations as a result of the combined effects of astronomical tides and fresh water runoff from streams and rivers and modified tidal and surge conditions.

The primary differences in estimating coastal erosion for these two types of beach exposures relate to how waves and water levels are determined for the 1-percent-annual-chance storm condition. Refer to Subsection D.2.2 for guidance on how the 1-percent-annual-chance storm is determined and to Subsections D.2.4 and D.2.5 for guidance on how waves and water levels are estimated for these two coastal exposures.

Beach setting refers to localized geomorphic characteristics of the shore and backshore zone related to site-specific geology, profile shape, material composition, and material erodibility; proximity to other dominant features such as coastal inlets, storm outfalls, streams, and creeks; harbors and coastal structures; littoral sediment supply; pocket beaches; and seasonal changes in beach width due to changes in wave direction. Four common beach settings representative of those along the Gulf of Mexico and Atlantic Ocean shorelines are addressed in these guidelines:

1. Sandy beach backed by a low sand berm or high sand dune formation
2. Cobble, gravel, shingle, or mixed grain sized beach and berms
3. Erodible and non-erodible coastal bluffs or cliffs
4. Sheltered waters (e.g., tidal marsh or other reduced-energy basins)

Beach Setting No. 1 is likely to be the most important coastal setting for the Atlantic and Gulf coasts from a hazards mapping perspective. This setting tends to experience the most erosion and flooding during large storm events.

The main erosion-related factors affecting all beach profiles during storms are:

Antecedent conditions of the beach and back beach (profiles and beach-dune juncture elevation) before the occurrence of the specified storm event;

Forcing processes that include the duration and time histories of wave characteristics, water levels, and runup; and

Response elements that include the beach setting and the dune/bluff characteristics, including material erodibility.

To estimate profile changes for beaches and back-beach dunes and bluffs during storms, Mapping Partners need erosion-assessment methods that account for the unique morphologies of each beach setting and the general effects of the above processes. The following four subsections briefly describe the characteristics of each Atlantic and Gulf coast beach setting that influence erosion assessments. The procedures for estimating storm-induced erosion for each setting are then provided in detail.

D.2.9.3.1 Sandy Dunes

Figures D.2.9-3a and D.2.9-3b provide sketches of typical beach profiles for broad sandy beaches backed by dunes or low sand berms. The primary factor controlling the basic type of dune erosion is the pre-storm cross section lying above the 1-percent-annual-chance SWEL (frontal dune reservoir). The Mapping Partner shall determine this area to assess the stability of the dune as a barrier. If the elevated dune cross-sectional area is very large, erosion will result in retreat of the seaward duneface with the dune remnant remaining as a surge and wave barrier. On the other hand, if the dune cross-sectional area is relatively small (for example, Figure D.2.9-4), erosion will remove the pre-storm dune leaving a low, gently sloping profile. Different treatments for erosion are required for these two distinct situations because no available model of dune erosion suffices for the entire range of coastal situations.

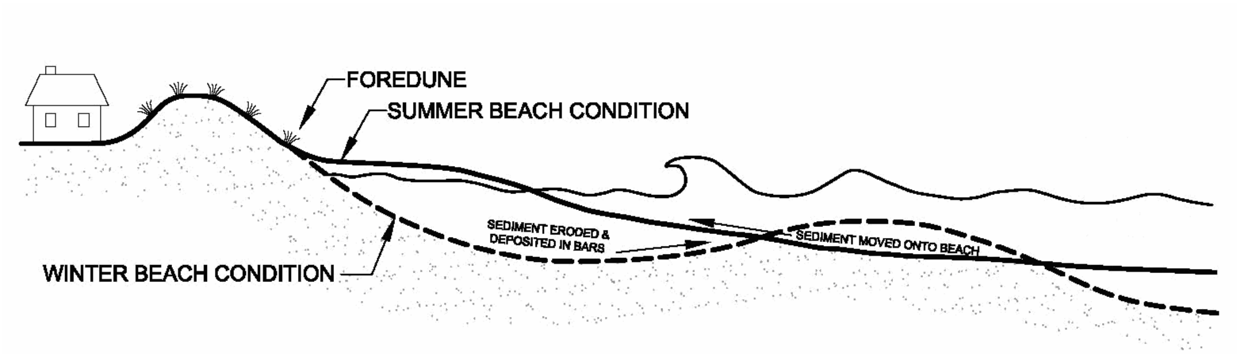


Figure D.2.9-3a. Sand Beach Backed by High Sand Dune (Beach Setting No. 1) (after Griggs, 1985)

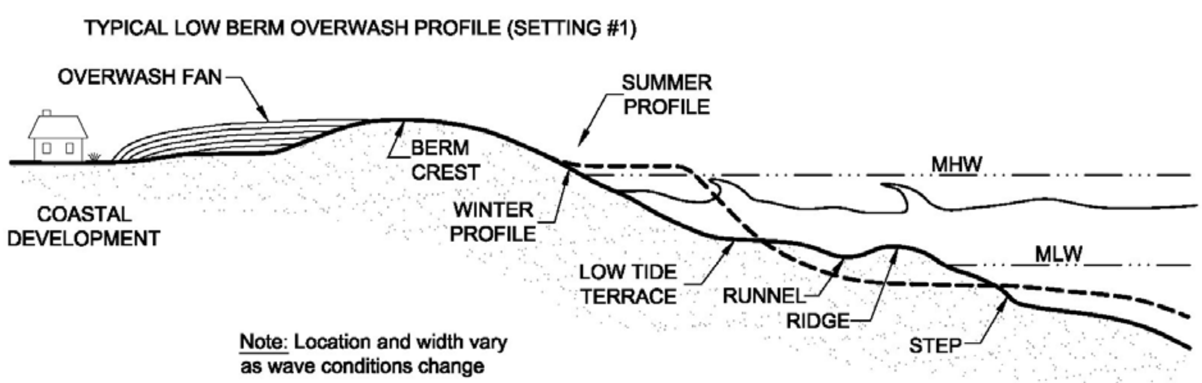


Figure D.2.9-3b. Sand Beach Backed by Low Sand Berm (Beach Setting No. 1) (after Bascom, 1964)

Figure D.2.9-4. Sandy Beach Backed by Low Dune

Figure D.2.9-5 introduces terminology for two representative dune types. A frontal dune is a ridge or mound of unconsolidated sandy soil, extending continuously along the shore landward of the sand beach. The dune is defined by relatively steep slopes abutting markedly flatter and lower regions on each side. For example, a barrier island dune has inland flats on the landward side, and the beach or back beach berm on the seaward side. The dune toe is a crucial feature and can be located at the junction between gentle slope seaward and a slope of 1:10 or steeper, marking the front dune face. The rear shoulder, as shown on the mound-type dune in Figure D.2.9-5, is defined by the upper limit of the steep slope on the dune's landward side.

The rear shoulder of mound-type dunes corresponds to the peak of ridge-type dunes. Once erosion reaches those points, the remainder of the dune offers greatly lessened resistance and is highly susceptible to rapid and complete removal during a storm. Figure D.2.9-5 shows the location of the “frontal dune reservoir,” above the 1-percent-annual-chance SWEL and seaward of the dune peak or rear shoulder. The amount of frontal dune reservoir determines dune integrity under storm-induced erosion.

As a result of changes to the NFIP regulations, coastal flood studies undertaken since the 1990s have analyzed and mapped dune ridge systems and assessed whether these features are able to withstand storm-induced erosion and remain as barriers to coastal flooding. Those dunes meeting specific NFIP criteria are designated as PFDs. Section 59.1 of the NFIP regulations defines a PFD as, “a continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent to the beach and subject to erosion and overtopping from high tides and waves during major coastal storms.” The regulations further state that the inland limit of the PFD, also known as the heel of the dune, “occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope.” The inland limit of the PFD establishes the minimum landward limit of the V zone. Section 65.11 of the NFIP regulations explains the criteria by which PFDs will be evaluated to determine if they are of sufficient volume to act as barriers to storm surge and waves during the base flood.

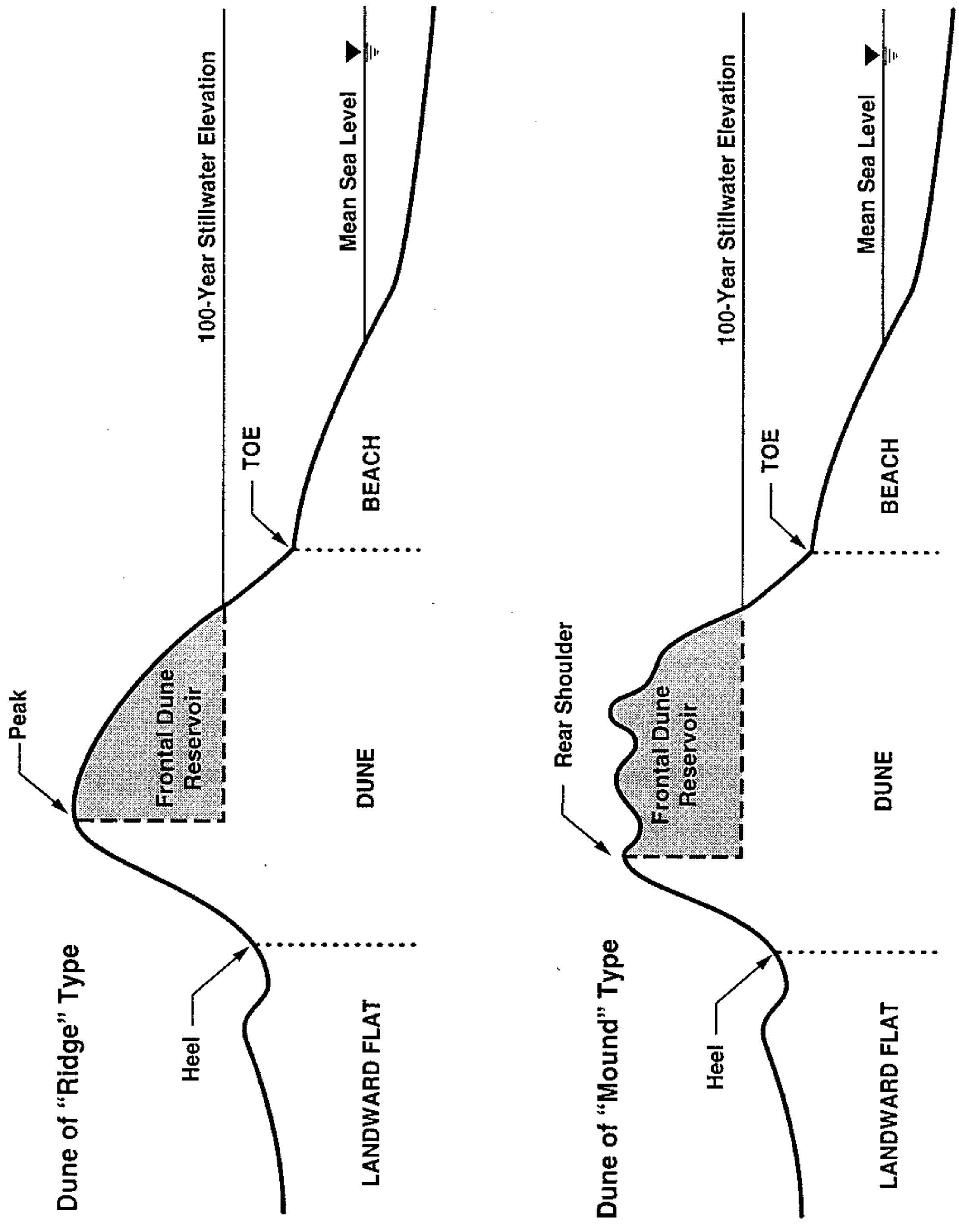


Figure D.2.9-5. Dune Features and Present Terminology

To prevent dune removal during the 1-percent-annual-chance storm, the frontal dune reservoir must typically have a cross-sectional area of at least 540 square feet (or 20 cubic yards volume per foot along the shore) above the 1-percent-annual-chance SWEL without wave setup (FEMA, September 1986; FEMA, November 1988). For more massive dunes, erosion will result in duneface retreat, with an escarpment formed on the seaward side of the remaining dune. To compute the eroded profile in such cases, FEMA has adopted a simplified version of the dune retreat model developed by Delft Hydraulics Laboratory of the Netherlands. This treatment is also appropriate in cases with sandy bluffs or headlands extending above the 1-percent-annual-chance SWEL. The simplified treatment of dune face retreat is described in Subsection D.2.9.3.1.2.

If a dune has a frontal dune reservoir less than 540 square feet in cross-sectional area, storm-induced erosion can be expected to obliterate the existing dune with sand transported both landward and seaward. The Mapping Partner shall estimate the eroded profile using procedures presented in Subsection D.2.9.3.1.1. Those procedures provide a realistic eroded profile across the original dune, but do not determine detailed sand redistribution by dune erosion, overwash, and breaching. Quantitative treatment of overwash processes is not yet feasible (Birkemeier et al., 1987), so the frontal dune is simply removed.

The initial decision in treating erosion as duneface retreat or as dune removal is based entirely on the size of the frontal dune reservoir. For coastal profiles more complicated than those in Figure D.2.9-5, the Mapping Partner shall use judgment to separate the sand reservoir expected to be effective in resisting dune removal from the landward portion of the pre-storm dune. The Mapping Partner shall complete the erosion assessment for the shoreline conditions representative of either the summertime shore profile for hurricane effects or the wintertime shore profile for northeaster storm effects, whichever is the appropriate and predominant source of coastal flooding that has been selected for use in the coastal hydraulic analyses and erosion assessment.

Figure D.2.9-6 presents a complete flowchart of necessary erosion considerations, outlining the major alternatives of duneface retreat and dune removal. Figure D.2.9-7 provides schematic sketches of the different geometries of dune erosion arising in coastal flood hazard assessments.

One additional factor complicating erosion assessment is the dissipative effect of wide sand beaches that shelter dunes from the full storm impact and retard retreat or removal. If the existing slope between mean level and the 1-percent-annual-chance SWEL is 1:50 or gentler, overestimation of erosion is possible during the 1-percent-annual-chance flood; therefore, the Mapping Partner shall examine this carefully. This effect and other variables, such as sand size, dune vegetation, and actual storm characteristics at a specific site, make thorough comparison of estimated erosion to documented historical effects in extreme storms necessary.

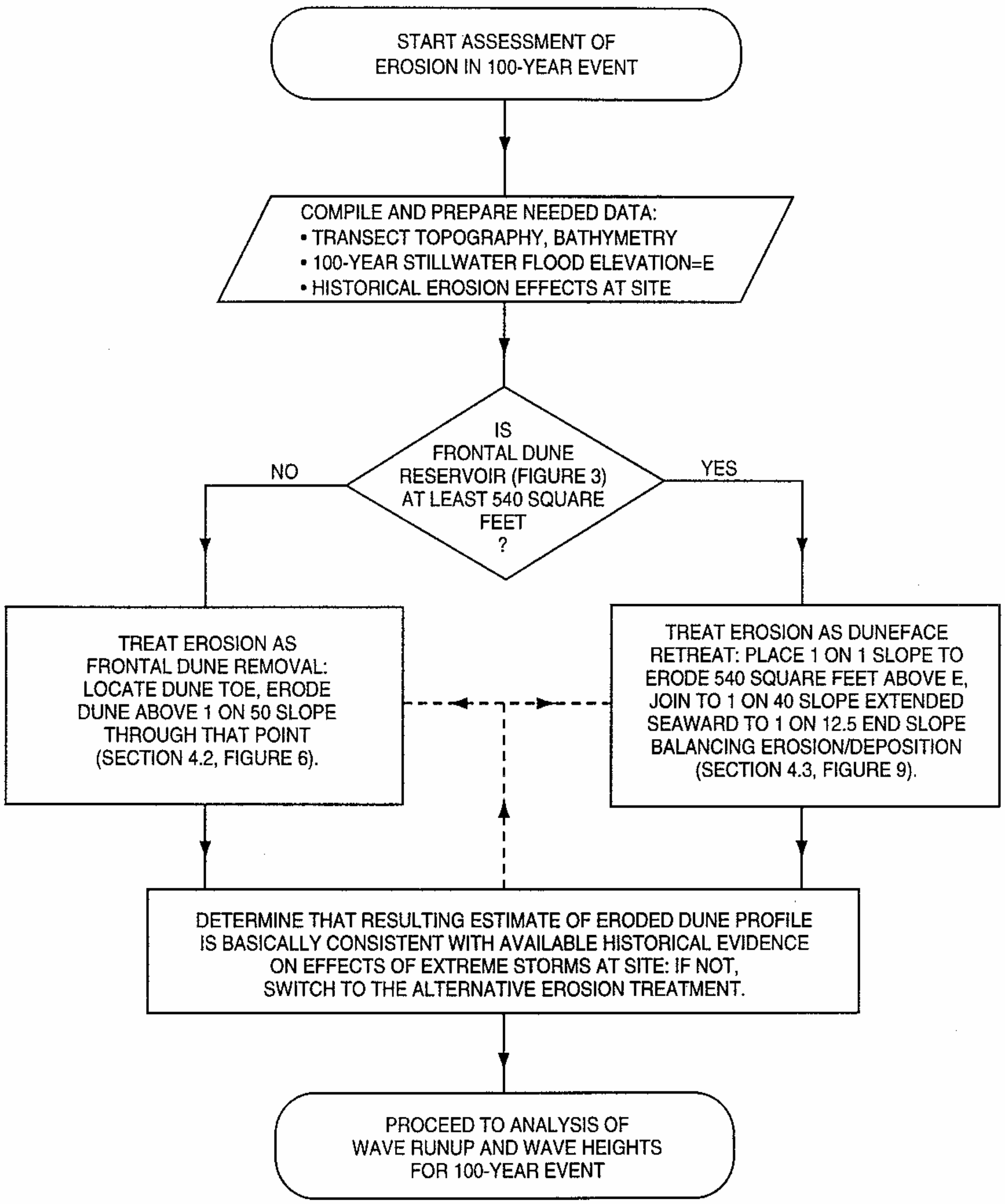


Figure D.2.9-6. Flowchart of Erosion Assessment for a Coastal Flood Map Project

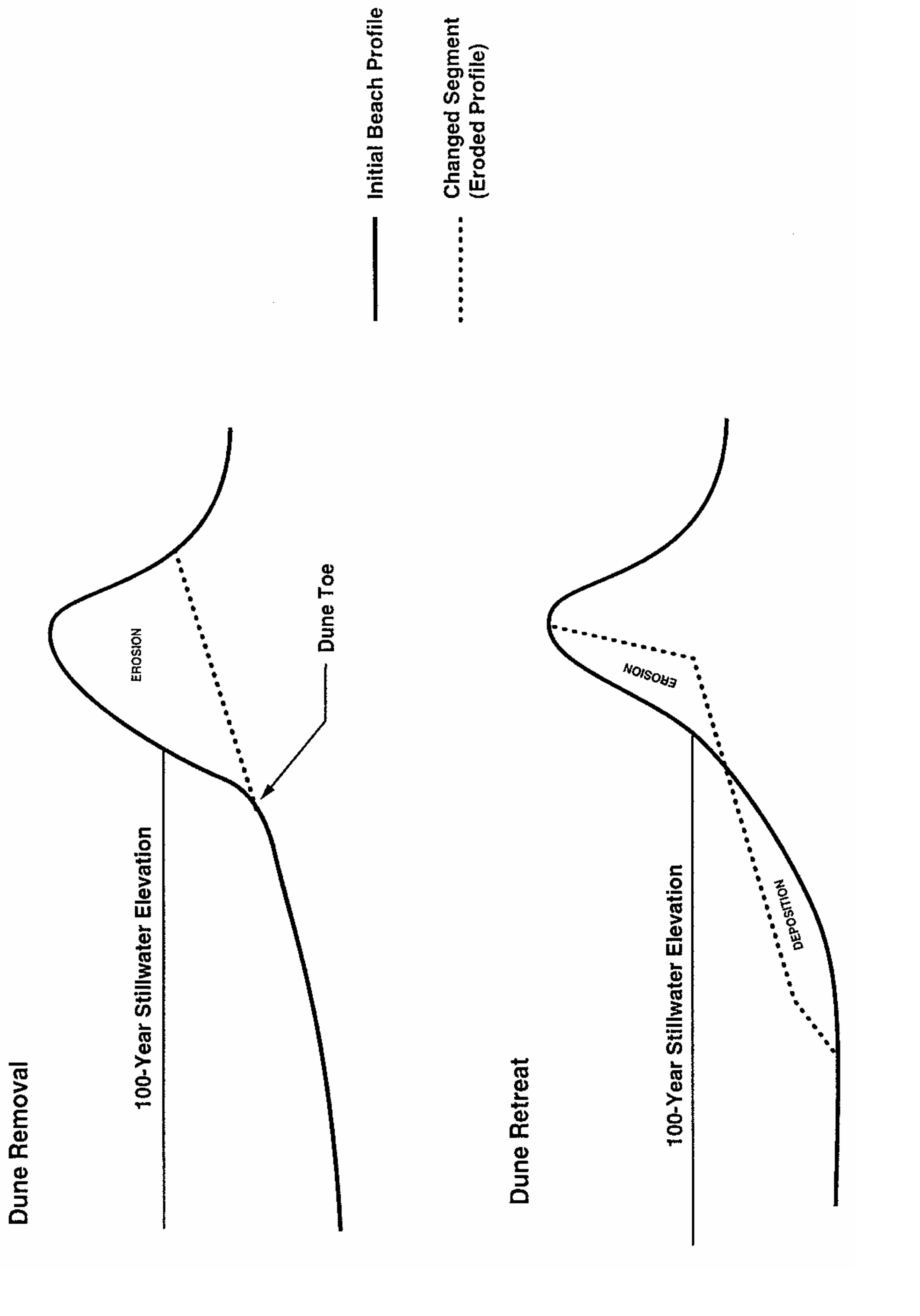


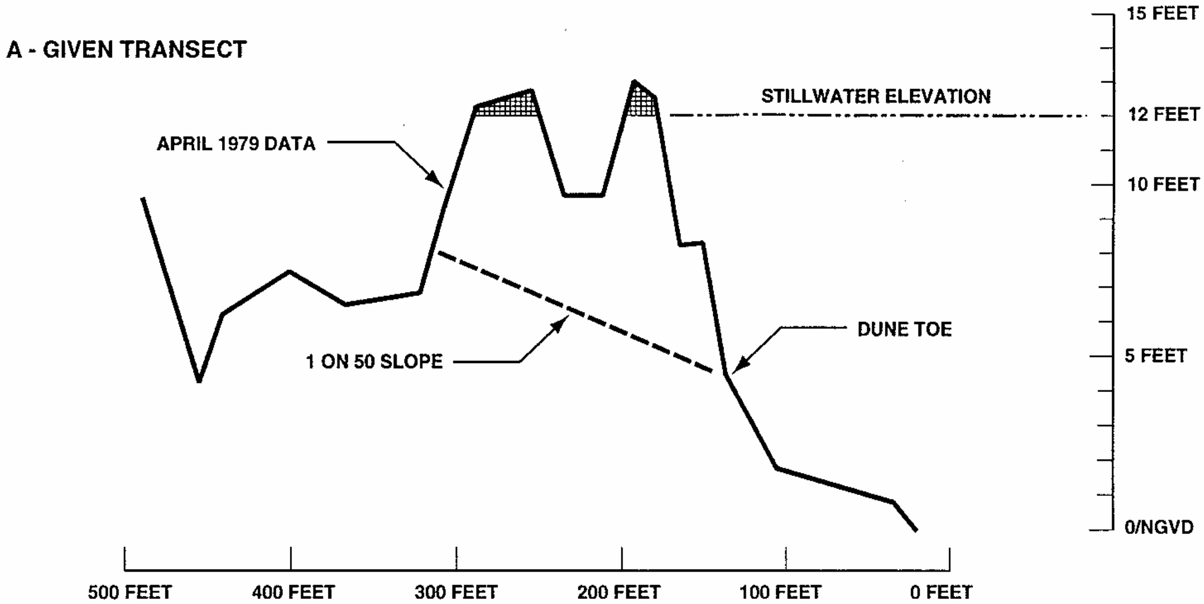
Figure D.2.9-7. Schematic Cases of Eroded Dune Geometries with Planar Slopes

D.2.9.3.1.1 Treatment of Dune Removal

Determining the dune reservoir requires an assessment of the profile area located above the 1-percent-annual-chance still-water flood level and seaward of the crest of the primary dune (see Figure D.2.9-5). Where the frontal dune reservoir is less than 540 square feet, construction of the eroded profile is extremely simple: dune removal is effected by means of a seaward-dipping slope of 1:50 running through the dune toe. The eroded profile is taken to be that slope across the pre-storm dune spliced onto the flanking segments of a given transect. This gives a gentle ramp across the extended storm surf zone, which is adequate as a first approximation to the profile existing at the storm's peak. This treatment simply removes the major vertical projection of the frontal dune from the transect.

Construction of an eroded profile focuses on the usually distinct feature termed the dune toe. The dune toe is taken to be the junction between the relatively steep slope of the front duneface and the notably flatter seaward region of the beach or the back-beach berm (including any minor foredunes). If a clear slope break is not apparent on a given coastal transect, its location should be taken at the typical elevation of definite dune toes on nearby transects within the study area. Alternatively, the dune toe may be set at the local 10-percent SWEL, which has been shown to be an adequate approximation along the Atlantic and Gulf coasts. In every case, the dune toe must be taken at an elevation above that of any beach berms on local shores.

Figures D.2.9-8, D.2.9-9, and D.2.9-10 show examples of the dune removal method described above. These simple constructions give appropriate estimates for the limits of high ground removed during the 1-percent-annual-chance flood, but cannot provide accurate representations of eroded profiles because of the complicated processes of dune failure. One example of overly simplified results is seen when deeper scour appears to occur where the frontal dune reservoir is relatively large.



B - ANALYSIS

1 - FRONTAL DUNE RESERVOIR ABOVE STILLWATER FLOOD ELEVATION (SHOWN CROSS-HATCHED) TOTALS ABOUT 35 SQUARE FEET, SO DUNE REMOVAL IS EXPECTED IN THIS 100-YEAR EVENT.
2 - DUNE TOE IS LOCATED AT JUNCTION BETWEEN 1 ON 3.8 AND 1 ON 11.5 SLOPES ON DUNEFACE.
3 - 1 ON 50 SLOPE THROUGH DUNE TOE PROVIDES APPROPRIATE ESTIMATE OF ERODED PROFILE ACROSS REMOVED DUNE.

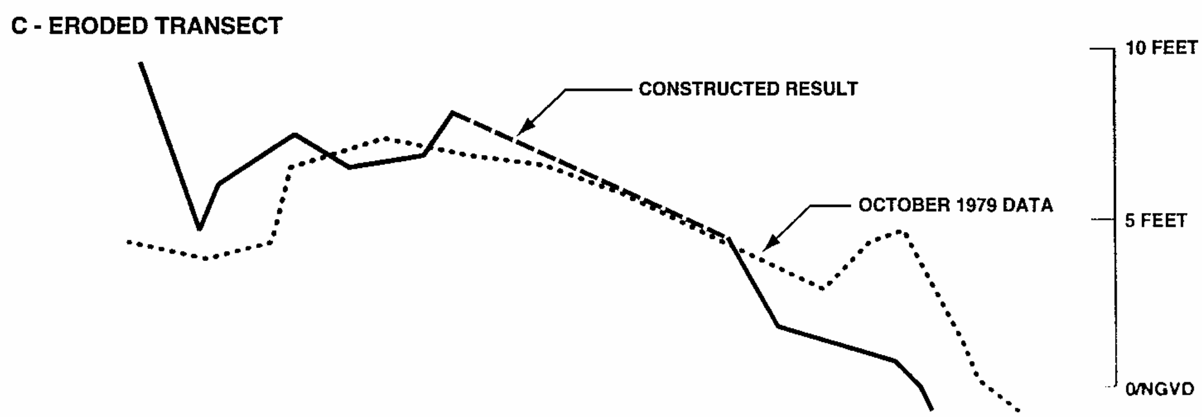


Figure D.2.9-8. Quantitative Example of Dune Removal Treatment for Alabama Profile Eroded by 1979 Hurricane Frederic. Situation Is Profile B-35 in Baldwin County, Alabama

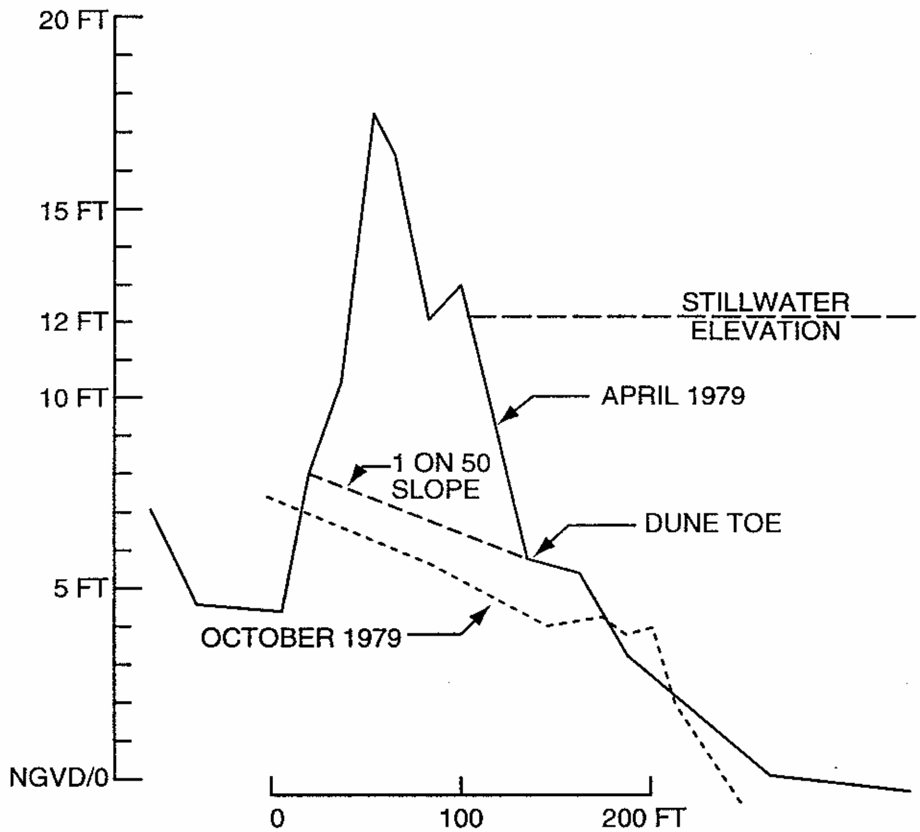


Figure D.2.9-9. Case of Relatively Large Dune Removed by 1979 Hurricane Frederic in Baldwin County, Alabama

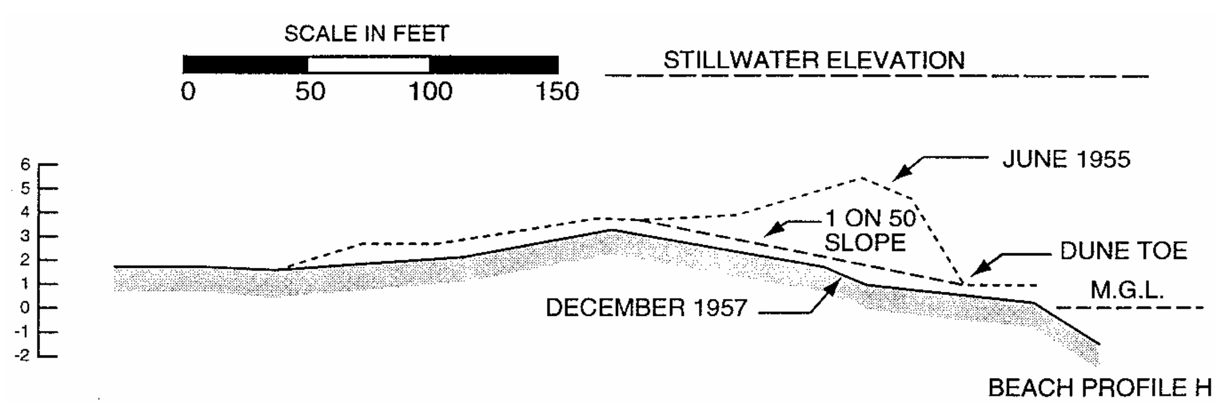


Figure D.2.9-10. Erosion of Relatively Low Profile by 1957 Hurricane Audrey in Cameron Parish, Louisiana

D.2.9.3.1.2 Treatment of Duneface Retreat

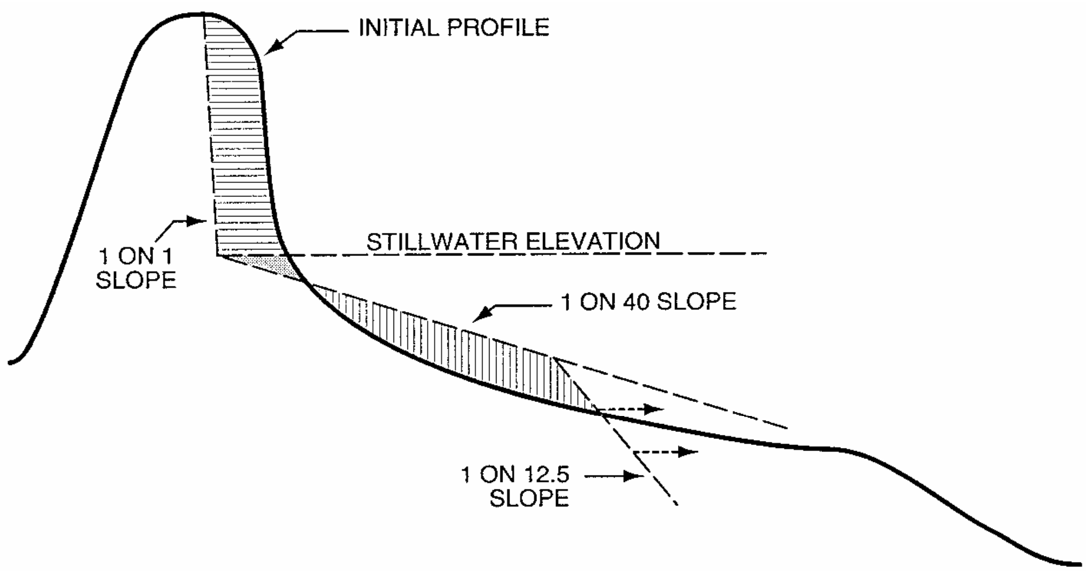
The procedure described here pertains to cases in which the frontal dune reservoir is at least 540 square feet. It yields an eroded profile for a beach impacted by duneface retreat during the 1-percent-annual-chance flood. During such retreat, the frontal dune barrier remains basically intact and eroded sand is transported in the seaward direction (Figure 2.9-11). The post-storm profile provides a balance between sand eroded from the duneface and sand deposited at lower elevations seaward of the dune.

Figure D.2.9-11. Dune Retreat on the South Shore of Long Island, New York

The following procedure for constructing the eroded profile constitutes a simplification of the dune retreat model developed by Delft Hydraulics Laboratory (DHL) of the Netherlands (DHL, 1986). Erosion above the 1-percent-annual-chance SWEL is fixed at 540 square feet to guarantee an appropriate amount for the Atlantic and Gulf coasts (FEMA, 1986 and November 1988). (In the DHL model, erosion is determined as the variable depending on specified storm and site conditions.)

The simplification of the DHL model eliminates potential problems associated with computation sensitivity to storm wave height and with uncertain capabilities for situations dissimilar to the Netherlands coast (Birkemeier et al., 1987; FEMA, November 1988). Other modifications of the model and treatment of duneface retreat have been implemented in an attempt to simplify the treatment by ignoring the variation of sand size and approximating the planar slope to the curved segment of the DHL post-storm profile.

Figure D.2.9-12 summarizes the simplified procedure adopted by FEMA to treat cases of duneface retreat. The eroded profile consists of three planar slopes: uppermost is a retreated duneface slope of 1:1, joining an extensive middle slope of 1:40, which is terminated by a brief segment with a slope of 1:12.5 at the limit to storm deposition. Upper dune erosion is specified to be 540 square feet above the 1-percent-annual-chance SWEL and in front of the 1:1 slope. Geometrical construction balances the nearshore deposition with the total dune erosion of somewhat more than 540 square feet by an appropriate seaward extension of the 1:40 slope. The resulting eroded profile is spliced onto the unchanged landward and seaward portions of the pre-storm profile. This procedure gives a complete profile suitable for use with the Wave Runup Model in assessing an appropriate flood elevation on the dune remnant.



PROCEDURE:
1 - CONSTRUCT RETREATED DUNEFACE WITH 540 FT^2 EROSION [ ] ABOVE 100-YEAR STILLWATER ELEVATION AND SEAWARD OF 1 ON 1 SLOPE.
2 - DETERMINE ADDITIONAL DUNE EROSION QUANTITY, SHOWN DOTTED, IN WEDGE BETWEEN STILLWATER ELEVATION, 1 ON 40 SLOPE, AND INITIAL PROFILE.
3 - BALANCE TOTAL DUNE EROSION WITH POSTULATED DEPOSITION [ ] BY APPROPRIATE PLACEMENT OF 1 ON 12.5 SLOPE AS LIMIT TO DEPOSITION.

Figure D.2.9-12. Procedure Giving Eroded Profile in Cases of Duneface Retreat, and Simplification of Dune Retreat Model Developed by Delft Hydraulics Laboratory of the Netherlands

Figure D.2.9-13A presents an example of duneface retreat according to the present procedure. This simple construction of a retreated dune profile gives appropriate eroded slopes important to the wave runup analysis of the remaining barrier. Where historical data on duneface retreat are available for comparison, agreement of estimated erosion slopes with those recorded should be considered of primary importance in verifying the present treatment. Actual quantities of dune erosion are subject to large variations in natural situations, and this procedure presumes a generally representative value for the 1-percent-annual-chance flood condition. For the example in D.2.9-13C taken from a study in Walton County, Florida, the estimated erosion and deposition proved greater than the observed. However, based on the reported characteristics of Hurricane Eloise, the associated flood was less severe than the 1-percent-annual-chance flood. So, while this specific profile would indicate another slope may be a better fit, the representative erosion slope proved to be valid, on average, for this reach of the coast and this storm.

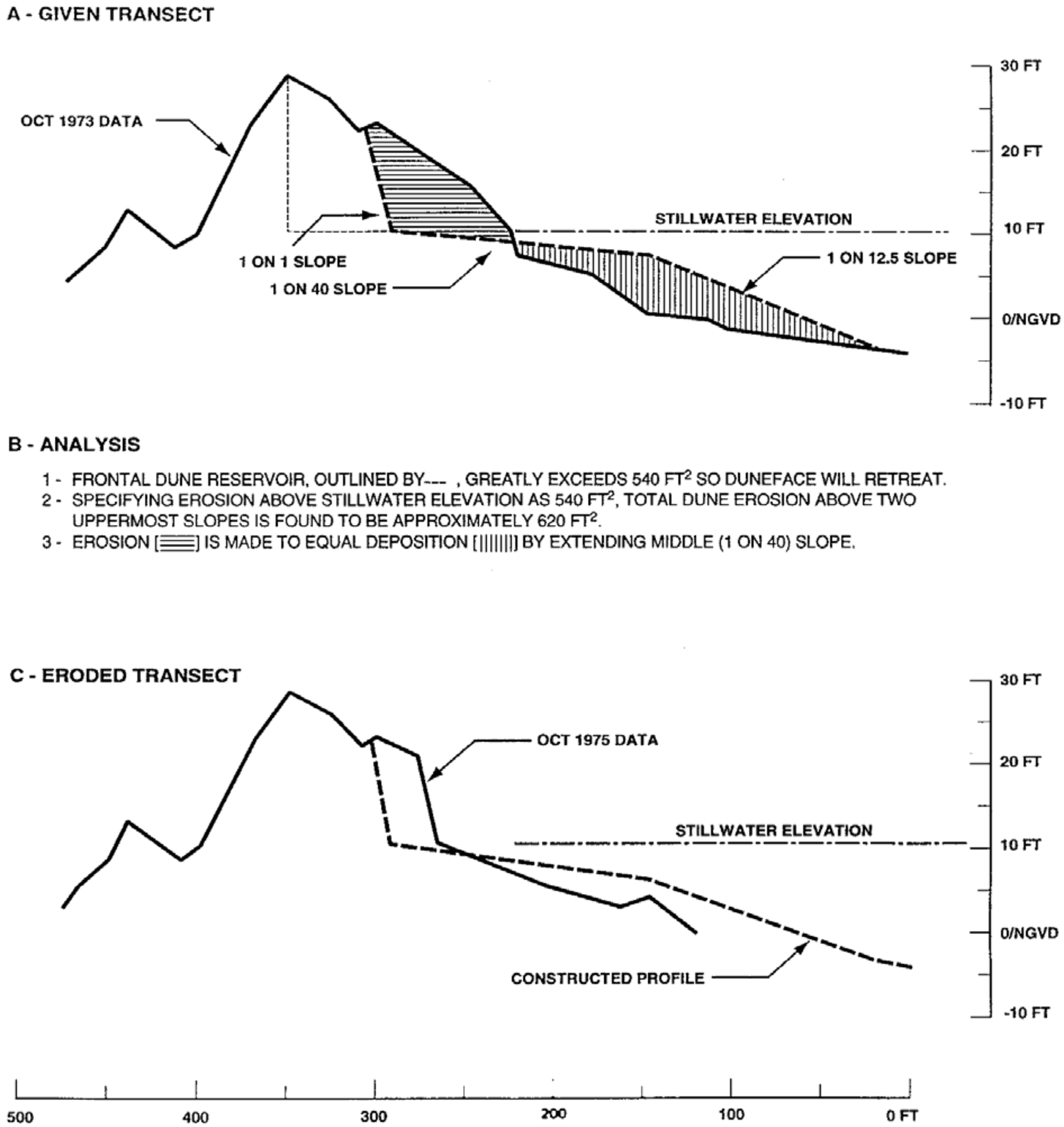


Figure D.2.9-13. Example of Duneface Retreat Treated by Simplified Version of DHL Model, with Erosion above SWEL Fixed at 540 Square Feet. Situation Is Profile R-105 in Walton County, Florida, Surveyed Before and After 1975 Hurricane Eloise

D.2.9.3.1.3 Finalizing Erosion Assessment

Based on measured erosion along the Atlantic and Gulf coasts, the demarcation between duneface retreat and dune removal in a 1-percent-annual-chance flood has been set at a frontal dune reservoir of 540 square feet (FEMA, 1986 and November 1988). This quantitative criterion might appear too precisely stated in view of potential inaccuracies in available dune topography, possible complications in delineating the effective frontal dune reservoir, and documented variability of dune erosion during extreme storms. In fact, the likelihood of duneface retreat or dune removal cannot be assessed with full certainty. Validating the present erosion assessment for a specific site by means of available evidence is advisable.

At many sites, historical evidence may be available regarding the extent of flooding, erosion, and damage in an extreme event comparable to the local 1-percent-annual-chance flood. In these instances, the erosion treatment giving results more consistent with historical records must be selected as appropriate. That choice may be relatively clear-cut given potential differences in expected erosion and inland flood penetration for duneface retreat versus dune removal. Where available historical evidence is not definitive, the decision between retreat and removal on a given transect should be based solely on size of the frontal dune reservoir. Present procedures for erosion assessment are highly simplified, but provide an unbiased estimation and a level of detail appropriate to coastal flood map projects.

D.2.9.3.1.4 Wave Overtopping for Cases of Duneface Retreat

Where the erosion assessment indicates duneface retreat, an eroded dune remnant persists as an appreciable barrier to the 1-percent-annual-chance flood. However, storm wave action can result in occasional extreme runup overtopping that barrier, yielding floodwater run off or ponding landward of the dune. DHL (1983) has determined the mean overtopping rate with storm waves incident on a typical duneface retreat geometry to be:

$$\overline{Q} = 5.26 \exp [-0.253 F] \quad \text{(D.2.9-1)}$$

Here the overtopping rate $\overline{Q}$1 has units of cubic feet per second per foot alongshore (cfs/ft), and F is maximum height (in feet) of the dune remnant above SWEL. This result was measured in DHL tests scaled to reproduce a specific extratropical storm on the Dutch seacoast, with a significant deep-water wave height of 25 feet and a peak wave period of 12 seconds. Those wave conditions seem roughly representative for the 1-percent-annual-chance flood along U.S. seacoasts, although expected wave characteristics will differ between hurricanes and extratropical storms at various sites. Recorded rates of overtopping can show sizable departures from the expected mean, even with steady flood conditions (Goda, 1985; Owen, 1980).

Despite uncertainties about actual overtopping rates for a dune remnant, the equation gives a useful basis for outlining expected effects. The threshold for severe overtopping, associated with jeopardizing the structural integrity of bare soil behind steep barriers exposed to storm waves, is on the order of magnitude of 1 cfs/ft (Goda, 1985). From Equation D.2.9-1, $\overline{Q}$2 of approximately 1 cfs/ft corresponds to F of approximately 7 feet, so retreated remnants with less relief above the 1-percent-annual-chance SWEL certainly require consideration of possible flood hazards landward of the dune. Appropriate treatments for ponding or runoff behind barriers are outlined in Subsection D.2.8.2.

D.2.9.3.2 Mixed / Coarse Sediment Systems

Beaches armored with cobbles, gravel, or other coarse sediments (Figures 2.9-14 and 2.9-15) develop in two distinct coastal environments. Often, these mixed-sediment beaches are prevalent in areas with slowly eroding bluffs that provide coarse sediment to the coastal system. Along the Atlantic Coast, these beaches are most common in New York, Massachusetts, and other New England States. In particular, mixed-sediment beaches are typically found along the shores of relatively sheltered bodies of water, where development of sandy beaches is inhibited by the absence of significant wind and wave action and by limited amounts of erodible sand. The other environment in which mixed-sediment beaches develop is one in which the coastline is exposed to high energy wave action, and, as a result, the finer sediments are winnowed away. Consideration of the wind and wave action to which the beach is exposed is necessary to determine whether the cobbles and gravel will provide a protective armoring against the 1-percent-annual-chance event, or whether wave action will exert sufficient force to erode them away.

Figure D.2.9-14. Mixed-sediment beach with materials ranging from sand to large cobbles, Peggotty Beach, Massachusetts (Photo courtesy of R. Haney, Massachusetts Coastal Zone Management)

Figure D.2.9-15. Mixed-sediment beach with well defined berm crest in Mann Hill Beach, Massachusetts (Photo courtesy of R. Haney, Massachusetts Coastal Zone Management)

Mixed-sediment beaches can vary significantly in overall morphology and sediment size distribution (i.e., size fractionation). These characteristics make it difficult to identify a "typical" mixed-sediment beach profile in either fair-weather or post-storm conditions. Figure D.2.9-16 provides one example of a mixed-sediment profile, but the composition and spatial relationships of the various sediment types can vary significantly from beach to beach. Historical profile data, therefore, are essential for the assessment of event-based erosion in mixed-sediment systems.

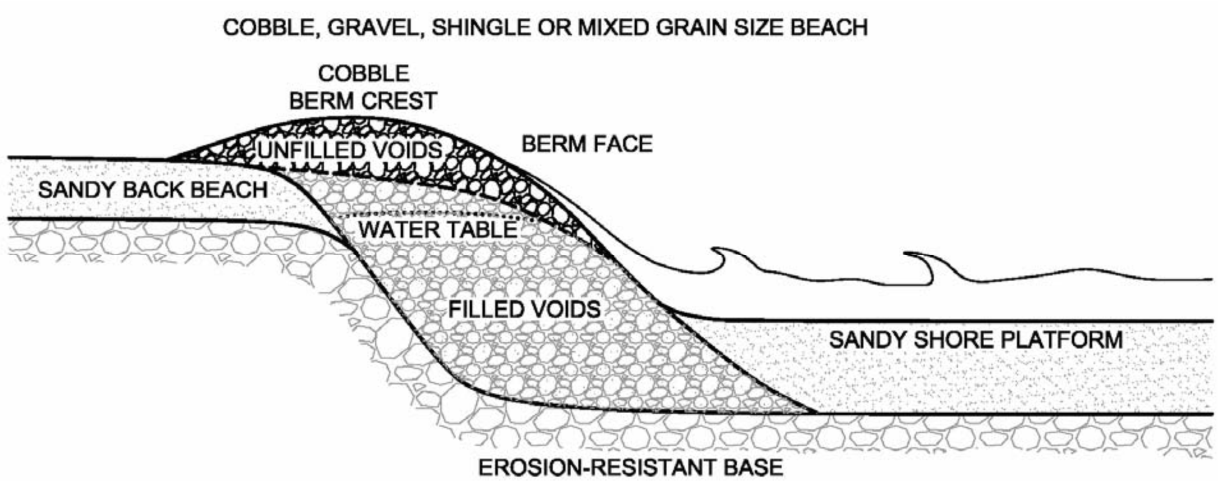


Figure D.2.9-16. Cobble, Gravel, Shingle, or Mixed Grain Sized Beach and Berms (Beach Setting No. 2)

Explicit procedures for determining beach and back beach profile changes on gravel and cobble beaches are not as well developed or documented as for sand beaches. Mixed-sediment beaches can be relatively stable in fair-weather conditions, but may behave dynamically or even breach in response to storms or other significant wave events (Figure D.2.9-17). Resorting of material, particularly between the Mean Higher High Water and Mean Lower Water elevations, is a common response to significant wave events (T. Ruthven, personal communication, 2006).

Figure D.2.9-17. Breach of mixed-sediment beach following a May 2005 northeaster, Mann Hill Beach, Massachusetts (Photo courtesy of R. Haney, Massachusetts Coastal Zone Management)

Given the lack of extensive observational data and prior coastal flood hazard analyses to date, there is currently no set of prescriptive profile geometries or erosion volumes that can be applied to all mixed-sediment beaches. Until such guidelines can be developed, Mapping Partners should use the following procedures and engineering judgment, as appropriate, to establish the typical eroded profile for Beach Setting No. 2, cobble, gravel, or shingle beaches and berms:

- Review the references listed in the support documents and literature on the design of and construction of dynamic revetments and cobble berms

- Examine photos and historical pre- and post-storm event LIDAR and beach profile data for the study area and develop a typical eroded profile from observed data, including a MLWP, if appropriate

- If a relatively broad sandy beach is located in front of the cobble berm, determine whether there is a history of significant erosion of the sand beach portion, and include that information in the beach profile data

- Survey the top-of-berm and back beach profile

- Splice the cobble berm profile, winter sand beach profile, top-of-berm, and back-beach profiles together to create a continuous beach profile that represents the complete profile for the beach, cobble berm, and back-beach areas

- Use this eroded-beach profile for subsequent runup computations during the selected storm event, unless other information indicates the profile may need further adjustment during large storm events

- Check results and try to validate them with observed information

- Document assumptions and results

D.2.9.3.3 Bluffs (Erodible and Erosion-Resistant)

Portions of the New England and U.S. island territory coasts have narrow to nonexistent beaches backed by high, steep, erodible coastal bluffs and cliffs, as illustrated in Figures D.2.9.-18 and D.2.9-19. The geomorphic evolution of this bluff-type shoreline is significantly different from that of the sandy beaches backed by either dunes or low-lying berms. A thin sand lens often overlies a rocky beach or bedrock platform fronting the bluff. These thin deposits of sand are removed during each winter storm season. If storm water levels reach sufficient elevations to intersect the toe of the bluff, storm waves can directly impinge upon the bluff face, causing bluff toe erosion (Figure 2.9-20). If enough material is eroded from the toe during a storm, the upper portion of the bluff can fail, resulting in bluff retreat. It should be noted that significant bluff failure may not occur during all storm events. However, if the bluff materials are erodible, toe erosion and bluff failure are possible during individual storm events.

Figure D.2.9-18. Erodible bluff fronted by narrow cobble beach, Scituate, Massachusetts (Photo courtesy of R. Haney, Massachusetts Coastal Zone Management)

Figure D.2.9-19. Erosion-resistant bluffs in Maine, with pocket beach of 0.5- to 1-m diameter boulders in the foreground (Photo courtesy of M. Honeycutt)

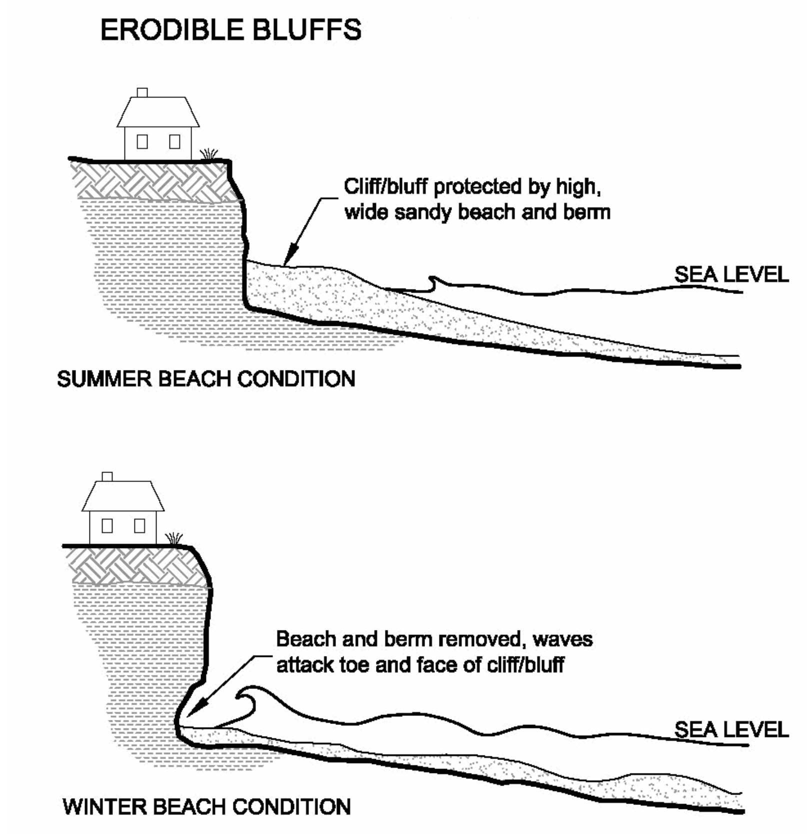


Figure D.2.9-20. Erodible coastal bluffs, showing seasonal profile variations and bluff-toe erosion (after Griggs, 1985)

The rate and extent of bluff erosion and failure depends on the site-specific bluff profile conditions at the time of the event (e.g., toe elevation and setback distance from the surf zone) and on the erodibility of the bluff materials. In some locations, it may take several storms to cause sufficient toe erosion to lead to bluff failure, or only one significant event with sufficient total water level, storm duration, and wave orientation to result in significant storm erosion.

Previous estimates of coastal bluff retreat have typically resorted in temporally averaged rates over a long period. Though the average annual rate of coastal cliff erosion is a reasonable indicator of the gradual retreat of the bluff top, it does not adequately predict the episodic nature of bluff failure that can result in 3 to 50 feet (or more) of retreat during a single storm event. The average annual retreat rate provides a misleading indication of the hazards of coastal bluff or cliff erosion because the occurrence of storms of sufficient magnitude and duration to cause significant buff retreat are episodic. At some locations, coastal bluffs have fairly low elevations and may be overtopped by large wave events. Therefore, assessment of coastal flood and erosion hazards in coastal settings with erodible bluffs (Beach Setting No. 3) requires special methods and data.

D.2.9.3.3.1 Erodible Bluffs and Cliffs

Once the Mapping Partner has determined through reconnaissance that a bluff or cliff is susceptible to erosion, it is important to investigate the coastal setting and history of episodic and chronic bluff erosion for the study area. The Mapping Partner should then follow the steps below:

1. Obtain reliable beach and bluff profile data (surveyed cross-shore profiles or LIDAR) for existing conditions. Try to obtain data reflecting conditions near the end of the winter season (March or April).
2. Determine whether bluff erosion and failure-monitoring data are available for the study area. Obtain and examine that information to determine the magnitude of episodic toe erosion and bluff retreat.
3. Estimate top-of-bluff elevations and compare to potential significant storm, total water level (stillwater plus waves). Determine whether the bluff is subject to overtopping, frequent wave attack, or toe erosion.
4. If potential damage to structures or public safety is determined to be insignificant, the Mapping Partner shall document those results and determine whether any further analyses of potential coastal flooding are recommended.
5. If further analysis of bluff erosion or overtopping is not recommended, or the site is determined to be non-eroding, the Mapping Partner should apply methods listed below for Erosion-Resistant bluffs.
6. Perform all additional runup and overtopping analyses on the surveyed existing winter conditions beach and bluff profiles for the site.
7. Document results and summarize the data, methods used, and assumptions associated with the analyses.

If it is determined that the study site experiences significant erosion and retreat during large storm events, the Mapping Partner shall document those findings and discuss with the FEMA Study Representative whether there are sufficient data, time, and budget to perform a more detailed bluff erosion analysis. Depending on the site-specific characteristics of the setting and bluff materials, a detailed bluff erosion analysis will likely require detailed geologic sampling, bluff erosion monitoring data, and bluff erosion simulation analyses. Data requirements and procedures for conducting detailed bluff erosion analyses are presented in Subsection D.4.6.8.2 of this Appendix.

D.2.9.3.3.2 Erosion-Resistant Bluffs or Cliffs

Erosion-resistant bluffs and cliffs are often fronted by rock terraces, rocky beaches, or narrow rock platforms capped with thin layers of sand or gravel. Once the thin sand cap is eroded from the rocky beach, this beach setting is stable. Therefore, Mapping Partners shall assume the sand cap is removed from the beach profile before a significant storm event, and use the adjusted rocky beach profile, along with measured profiles, for the non-erodible bluffs or cliffs for all subsequent runup and overtopping computations. All assumptions, methods, data resources, and results should be well documented.

D.2.9.3.4 Sheltered Waters

"Sheltered waters," for the purposes of FEMA coastal flood hazard analyses, are defined in detail in Subsection D.2.1.2.3. Generally, "sheltered waters" are water bodies or regions that experience diminished forces from wind and/or wave action relative to the open coast due to the presence of physical barriers, both natural and human, either on land or under water. Coastal reaches in this beach setting include (but are not limited to) low-energy beaches, tidal flats, and wetlands (Figure D.2.9-21).

Figure D.2.9-21. Photograph of Tidal Flats and Wetlands Complex (Photo by Northwest Hydraulic Consultants, Inc.)

Tidal flats and wetlands are low-gradient coastal features, usually comprised of fine cohesive silts and clay (Figure D.2.9-22). Sedimentation processes in this beach setting are typically depositional. Over time, these coastal landforms may become capped with wetland vegetation and detrital deposits, and sand or debris from overland wave propagation during storm events (Figure D.2.9-23).

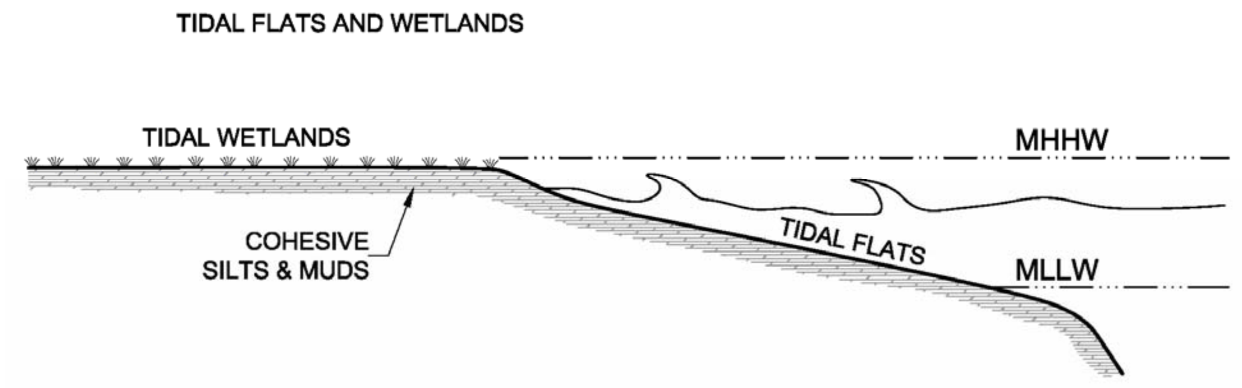


Figure D.2.9-22. Sheltered Water: Tidal Flats and Wetlands (Beach Setting No. 4)

Figure D.2.9-23. Virginia Tidal Salt Marsh, with Overwash Deposit After Hurricane Floyd

For the purposes of assessing storm-induced erosion, the classification of an area as ”sheltered” suggests that shoreline or wetland response to the storm surge and wave action is expected to be minimal. Therefore, Mapping Partners may assume that tidal mudflats and wetland profiles do not erode over the time-scale of a single storm event.

Mapping Partners should compare existing tidal flat and wetland profiles with recent post-storm profiles to verify this assumption. If it is determined that no measurable erosion occurs during single storm events, then the Mapping Partner shall use the existing profile information to determine runup, overtopping, and overland propagation. However, if it is found that measurable changes can occur during a single storm, the Mapping Partner should document the observed changes experienced at the site and propose to the FEMA Study Representative a procedure for using that information to adjust the existing profiles before determining runup, overtopping, and overland propagation. The Mapping Partner shall document assumptions, data used, and methods implemented to prepare the final profiles and summarize the results.

D.2.9.3.5 Special Considerations

D.2.9.3.5.1 Beach Nourishment and Constructed Dunes

Policy and criteria for evaluating the stability and performance of costal beach nourishment projects are not yet developed, and only basic guidance is provided in Subsection D.2.1.2.5.

The presence of constructed dunes in the study area may raise questions as to whether the material will affect flood insurance risk zones or BFEs. If the Mapping Partner determines that beach renourishment and/or dune construction have occurred in the study area, it should be brought to the attention of the FEMA Study Representative. As outlined in Subsection D.2.1.2.5, further analysis may be needed to assess the performance of the beach renourishment and/or dune construction, and what impact, if any, these actions will have on flood zones and BFEs.

For all practical purposes, the Mapping Partner shall treat constructed or reconstructed dunes (referred to as “artificial” dunes by FEMA) as natural dunes during the FIS. In the erosion assessment, this means assessing the dune reservoir and applying the dune removal (Subsection 2.6.3.1.1) or dune retreat (Subsection 2.6.3.1.2) procedures as appropriate. Note, however, that the condition of the artificial dune may alter this procedure. NFIP regulations [44 CFR 65.11(a)] do not allow an artificial dune to be considered as an effective barrier to coastal flooding, regardless of its size and cross-sectional area, unless it has well-established, longstanding vegetative cover. Pre-nourishment topography shall be used for non-vegetated artificial dunes and nourished beaches unless special consideration is granted by the FEMA Study Representative.

D.2.9.4.5.2 Erosion Assessment in the Vicinity of Coastal Structures

The erosion assessment procedures that the Mapping Partner must complete for coastal structures are dependent on whether the structure is certified or uncertified, and in the latter case, whether the structure will completely or partially fail during the 1-percent-annual-chance flood.

If a coastal structure is certified to withstand the 1-percent-annual-chance flood (see Subsection D.2.10.2), but the structure will be inundated by the 1-percent-annual-chance flood, the ground profile landward of the structure (including any PFD identified) must be evaluated for storm-induced erosion. Because the structure provides protection against the full forces associated with the base flood, application of the 540-square-foot rule for open coast settings would not be appropriate. The Mapping Partner shall, at a minimum, erode the land surface immediately landward of the structure to the crest elevation. The Mapping Partner will need to exercise professional judgment in constructing the remainder of the eroded profile, connecting the crest elevation to natural ground elevation at a point landward where erosion is likely to be negligible.

If a coastal structure cannot be certified, the Mapping Partner must determine whether the structure will completely or partially fail during the base flood. When failure will be complete, the Mapping Partner shall remove the structure entirely from the analysis transect. The remaining soil profile should be altered to achieve its likely slope immediately after structure failure. Information on slopes behind failed structures is limited. These slopes may vary from 1:100 (v:h) for unconsolidated sands to 1:1 or steeper for consolidated material landward of the failed structure. The post-failure slope for this analysis should be in the range of 1:1 to 1:1.5. Note that the post-failure slope may not necessarily match the long-term stable slope, but will serve as the basis for subsequent site-specific, event-based, erosion wave height, wave runup, and wave overtopping analyses. If the Mapping Partner determines that the structure will partially fail, storm-induced erosion must still be evaluated both seaward and landward of the structure. Subsection D.2.7.3 of this document provides further information on how to determine an appropriate post-failure profile for use in subsequent coastal flood hazard modeling.

D.2.10 Coastal Structures

This subsection provides guidance for certifying coastal protection structures for use in the NFIP and outlines methods for analyzing the stability and effects of coastal structures during 1-percent-annual-chance flood conditions

D.2.10.1 Purpose and Overview

Because coastal structures can significantly affect local topography and flood hazards, the evaluation of coastal structures is a necessary part of any flood hazard study. The evaluation should, where possible, determine whether a coastal structure will survive the 1-percent-annual-chance flood and provide protection to upland areas.

- If a particular structure is expected to remain intact through the 1-percent-annual-chance flood, the structure geometry shall be used in all ensuing FIS analyses that accompany the flood event (e.g., event-based erosion, wave runup and overtopping, and determination of wave crest elevations).
- If a particular structure is expected to fail during the 1-percent-annual-chance flood, the coastal structure shall either be removed entirely before ensuing analyses, or be replaced by an appropriate failed configuration before ensuing analyses (D.2.10.3.2).
- If the performance of a particular structure is uncertain, both intact and failed configurations should be analyzed, and the most hazardous flood conditions should be mapped.

For the purposes of these Guidelines and Specifications, coastal structures are classified as follows:

- **Coastal Armoring Structures:** Generally shore-parallel structures constructed to prevent erosion of uplands and mitigate coastal flood effects (e.g., seawalls, revetments, bulkheads, and levees). Please note that coastal levees are classified as armoring structures here, but are often referred to as flood control structures.
- **Beach Stabilization Structures:** Structures intended to stabilize or reduce erosion of the beach, which, by doing so, afford some protection to upland areas (e.g., groins, breakwaters, sills, and reefs).
- **Miscellaneous Structures:** Structures not included above that can affect flood hazards, especially in sheltered waters (e.g., piers, port and navigation structures, bridges, culverts, and tide gates).

Criteria for evaluating the stability and performance of coastal armoring structures for FIS purposes are well-developed and are discussed in detail. Criteria for evaluating beach stabilization structures have not been developed yet, and only basic guidance is provided (beach nourishment is addressed in Subsections D.2.1.2.5 and D.2.9.3.5.1). Criteria for evaluating miscellaneous structures are not standardized, and only basic guidance is provided.

D.2.10.2 Evaluation Criteria

Mapping Partners are not required to perform detailed engineering evaluations of all coastal structures within the study area, and, in fact, rarely do so. However, when such an evaluation is performed, there are specific evaluation criteria that must be applied.

D.2.10.2.1 Detailed Engineering Evaluation of Coastal Armoring Structures

Specific criteria for evaluating coastal armoring structures are contained in an April 23, 1990, FEMA memorandum (FEMA, 1990), "Criteria for Evaluating Coastal Flood Protection Structures for National Flood Insurance Program Purposes."[12] The evaluation criteria from the 1990 memorandum are provided below[13]:

> General
>
> For purposes of the NFIP, FEMA will only recognize in its flood hazard and risk mapping effort those coastal flood protection structures that meet, and continue to meet, minimum design and maintenance standards that are consistent with the level of protection sought through the comprehensive floodplain management criteria established by 44 CFR Part 60.3. Accordingly, the procedure describes the types of information FEMA needs to recognize, on NFIP maps, that a coastal flood protection structure provides protection from the base flood. This information must be supplied to FEMA by the community or other party seeking recognition of such a coastal flood protection structure at the time a flood risk study or restudy is conducted, when a map revision under the provision of 44 CFR Part 65 is sought based on a coastal flood protection structure, and upon request by the Administrator during the review of previously recognized structures. The FEMA review will be for the sole purpose of establishing appropriate risk zone determinations for NFIP maps and shall not constitute a determination by FEMA as to how a structure will perform in a flood event.
>
> Design Criteria
>
> For coastal flood protection structures to be recognized by FEMA, sufficient evidence must be provided that adequate design, construction, and maintenance have been undertaken to provide reasonable assurance of durable protection from the base flood. The following requirements must be met:
>
> 1. Design Parameters. A coastal flood protection structure must be designed using physical parameters that fully represent the base flooding event, including the following:

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[12] The criteria discussed in this memorandum are based in large part on Technical Report 89-15 prepared by the U.S. Army Corps of Engineers, Coastal Engineering Research Center (USACE CERC) for FEMA, Criteria for Evaluating Coastal Flood-Protection Structures (Walton et al., 1989). The criteria in the memorandum have been adopted as the basis for National Flood Insurance Program (NFIP) accreditation of new or proposed coastal structures to reduce the flood hazard areas and elevations designated on NFIP maps, but can be applied to existing coastal structures.

[13] The use of the term *stillwater* in this memorandum shall be understood to refer to total stillwater, or MWL.

i. Design water levels evaluated should range from the mean low water at the site, to the 1-percent-annual-chance stillwater elevation. The full range of elevations must be examined to determine the critical water level because the most severe conditions may not occur at either extreme.

ii. Wave heights and periods must be calculated for each water level analyzed. At a minimum, significant wave heights and periods should be used for "flexible" structures such as revetments, and larger wave heights, up to the 1-percent-annual-chance wave height (1.67 times the significant wave height), used for more rigid structures such as seawalls and bulkheads. The U.S. Army Corps of Engineers (USACE) Shore Protection Manual (1984 or later edition), provides guidance and procedures for determining appropriate wave heights and periods.

iii. Breaking wave forces under structure-perpendicular loading must be considered in the design unless it can be demonstrated that the structure will not be subject to breaking waves. The very high, short duration "shock" pressures must be used for low mass structures such as bulkheads, while only the secondary "non-shock" pressures need to be used for massive structures such as gravity seawalls. Analyses of the breaking wave forces using methods such as those identified in the USACE report "Criteria for Evaluating Coastal Flood Protection Structures," (WES TR CERC-89-15) must be submitted.

2. Minimum Freeboard. The minimum freeboard for coastal flood protection structures to be recognized on FEMA flood maps for protection against the storm surge component of the base flood shall be 2 feet above the 1-percent-annual-chance stillwater elevation [and 1 foot above the height of the 1-percent-annual-chance wave or the maximum wave runup (whichever is greater)].

3. Toe Protection. The loss of material and profile lowering seaward of the structures must be included in the design either through the incorporation of adequate toe protection or an evaluation of structural stability with potential scour equal to the maximum wave height on the structure. Engineering analyses such as those recommended in the USACE's "Geotechnical Engineering in the Coastal Zone" (WES IR CERC-87-1) or "Design of Coastal Revetments, Seawalls, and Bulkheads" (COE EM 1110-2-1614) must be submitted for toe protection, or an analysis of scour potential such as found in "Criteria for Evaluating Coastal Flood Protection Structures," (WES TR CERC-89-15) must be submitted.

4. Backfill Protection. Engineering analyses of wave runup, overtopping, and transmission must be performed using methods provided in the USACE report "Criteria for Evaluating Coastal Flood Protection Structures," (WES TR CERC-89-15). Where the structure height is not sufficient to prevent overtopping and/or wave transmission, protection of the backfill must be included in the design. This should address prevention of loss of backfill by rundown over the structures, by drainage landward, under, and laterally around the ends of the structure as well as through joints, seams, or drainage openings in the structures.

5. Structural Stability, Minimum Water Level. Analyses of the ability of the structures to resist the maximum loads associated with the minimum seaward water level, no wave action, saturated soil conditions behind the structures, and maximum toe scour must be submitted. For coastal dikes and revetments, geotechnical analyses of potential failure in a landward direction by rotational gravity slip must be submitted.
   i. For coastal dikes and revetments, geotechnical analyses of potential failure in a landward direction by rotational gravity slip must be submitted.
   ii. For gravity and pile-support seawalls, engineering analyses of seaward sliding, seaward overturning, and foundation adequacy using the maximum pressures developed in the sliding and overturning calculations must be submitted.
   iii. For anchored bulkheads, engineering analyses of shear failure, moment failure, and the adequacy of the tiebacks and deadmen to resist the loadings must be submitted.
6. Structural Stability, Critical Water Level. Analyses of the ability of the structure to resist the maximum loads associated with the critical water level, which may be any water level from the mean low water level to the 1-percent-annual-chance stillwater elevation, including hydrostatic and hydrodynamic (wave) loads, saturated soil conditions behind the structure, and maximum toe scour must be submitted.
   i. For coastal dikes and revetments, geotechnical analyses of potential failure in a seaward direction by rotational gravity slip and of foundation failure due to inadequate bearing strength must be submitted.
   ii. For revetments, engineering analyses of the rock, riprap, or armor block stability under wave action; uplift forces on the rock, riprap, or armor blocks; toe stability; and adequacy of the graded rock and geotechnical filters must be submitted.
   iii. For gravity and pile-supported seawalls, engineering analyses of landward sliding, landward overturning, and foundation adequacy using the maximum pressures developed in the sliding and overturning calculations must be submitted.
   iv. For anchored bulkheads, engineering analyses of shear failure and moment failure using “shock” pressures must be submitted.
7. Material Adequacy. Documentation and/or analyses must be submitted that demonstrate that the materials used for the construction of the structure are adequate and suitable, including life expectancy considerations, for the conditions that exist at the site.
8. Ice and Impact Alignment. Where appropriate, analyses of ice and impact forces must be submitted.
9. Structure Plan Alignment. A shore protection project should present a continuous structure with redundant return walls at frequent intervals to isolate locations of

failure. Isolated structures, or structures with a staggered alignment, must submit analyses of the additional forces from concentrated, diffracted, and/or reflected wave energy on the different sections and ends.

10. Other Design Criteria. FEMA will require that flood protection structures described above, regardless of type, be evaluated on the basis of how they may react structurally to applied forces. Therefore, analyses normally required of one structure type may also be required by another type that would react in a similar manner to applied forces. In unique situations, FEMA may require that other design criteria and analyses be submitted to show that the structure provides adequate protection. In such situations, sound engineering practice will be the standard on which FEMA will base its determinations. FEMA will provide the rationale for requiring any additional information.

<u>Adverse Impact Evaluation</u>

All requests for flood map revisions based upon new or enlarged coastal flood control structures shall include an analysis of potential adverse impacts of the structure on flooding and erosion within, and adjacent to, the protected area.

<u>Community and/or State Review</u>

For coastal flood protection structures to be recognized, evidence must be submitted to show that the design, maintenance, and impacts of the structures have been reviewed and approved by the affected communities and by any Federal, State, or local agencies that have jurisdiction over flood control and coastal construction activities.

<u>Maintenance Plans and Criteria</u>

For a coastal flood protection structure to be recognized as providing protection from the base flood, the structure must be maintained in accordance with an official adopted maintenance plan. A copy of this plan must be provided to FEMA by the owner of the structure when recognition is being sought or when the plan for a previously recognized structure is revised in any manner. All maintenance activities must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or any agency of a community participating in the NFIP that must assume ultimate responsibility for maintenance. This plan must document the formal procedure that ensures that the stability and overall integrity of the structure and its associated structures and systems are maintained. At a minimum, maintenance plans shall specify the maintenance activities to be performed, the frequency of their performance, and the person by name or title responsible for their performance.

<u>Certification Requirements</u>

Data and analyses submitted to support that a given coastal flood protection structure complies with the structural design requirements set forth in paragraphs 1 through 10 above must be certified by a registered professional engineer. Also, certified as-built plans of the structure must be submitted. Certifications are subject to the definition given at § 65.2 of 44 CFR Part 65. In lieu of these certification requirements, a Federal agency with responsibility for design of coastal flood protection structures may certify that the structure has been adequately designed and constructed to provide protection against the base flood.

Where a Mapping Partner chooses to perform a detailed engineering evaluation of an existing coastal armoring structure during an FIS, FEMA requires the evaluation to be based upon the criteria outlined above from the April 23, 1990, FEMA memorandum, and upon as-built documentation. When as-built documents are not available, the evaluation should be based upon best available data, standard design and engineering assumptions, and conservative estimates of material properties. The evaluation should be confirmed and documented by past performance during severe storm events. The underlying requirement is that the evaluation must yield an accurate assessment of coastal structure performance during the 1-percent-annual-chance flood, based upon available evidence.

It should be noted, however, that the art of coastal structure evaluation is constantly evolving. Thus, the Mapping Partner may choose to propose evaluation criteria that differ from those contained in the April 23, 1990, FEMA memorandum (e.g., from the CEM [USACE, 2003], or from other authoritative and accepted references). However, permission should be obtained from the FEMA Study Representative prior to utilizing alternative evaluation procedures and criteria.

D.2.10.2.2 Coastal Armoring Structure Evaluation Based on Limited Data and Engineering Judgment

For the purposes of an FIS, the Mapping Partner may not have sufficient resources and time to conduct a detailed evaluation of each coastal armoring structure within the study area. In such cases, the Mapping Partner can apply engineering judgment (albeit, guided by the FEMA memorandum and USACE CERC *Technical Report 89-15 "Criteria for Evaluating Coastal Flood Protections Structures*") to determine the likely stability of each structure during the 1-percent-annual-chance flood. These conclusions may be based largely on available archive information and local observations, including historic evidence of storm damage and maintenance. Note that any data and procedures used in the evaluations shall be documented (see Subsections D.2.10.6 and D.2.10.7), and communities and property owners shall be made aware that these evaluations are for mapping purposes only.

If the available information does not clearly point to survival or failure of a coastal structure, the Mapping Partner may either:

1. Conduct a detailed evaluation based on the FEMA criteria (April 23, 1990) (see the previous subsection).
2. Perform the erosion and wave analyses for both the intact and failed structure cases and map the flood hazards associated with the more hazardous case.

If option 1 is selected, the Mapping Partner shall clearly document the results of all cases investigated and specify which case is used for mapping purposes. It should be noted that a failed coastal structure may or may not yield the greatest flood hazards. Therefore, coastal flood analyses for the intact and failed conditions should be performed, with the greatest resulting flood hazard being mapped. Maintaining results of all analyses may be useful in the event map revisions are requested by property owners based upon certified structures[14].

[14] Often, property owners request revisions to the FIRM based upon existing, new, or proposed coastal structures. Map revisions based upon coastal structures require a detailed evaluation and certification by a professional engineer

D.2.10.2.3 Evaluation of Beach Stabilization Structures

Guidance on how to predict the survival or failure of groins, which usually fail by loss of profile (through settlement, displacement, or deterioration) and/or by becoming detached at their landward ends, is not readily available. Likewise, guidance on how to predict the failure of breakwaters, sills, and reefs (usually through loss of profile) is not readily available. Some information on failure modes may be available in technical or historical literature, and should be consulted by the Mapping Partner.

If a Mapping Partner chooses to evaluate beach stabilization structures during an FIS, the proposed evaluation methods and procedures should be discussed with the FEMA Study Representative, in advance, and approval by FEMA must be obtained before the evaluations can be carried out.

D.2.10.3 FIS Treatment of Coastal Armoring Structures

Technical Report 89-15 identifies four primary functional types of coastal flood protection structures: gravity seawalls, pile-supported seawalls, anchored bulkheads, and dikes or levees. A fifth type, revetment, is added here (see Figure D.2.10-1b).

Technical Report 89-15 recommends as a general policy that “FEMA not consider anchored bulkheads as providing flood protection during large storms.” Thus, the default assessment for open coast anchored bulkheads should be that they are assumed to fail during the 1-percent-annual-chance flood. Mapping Partners may choose to treat some anchored bulkheads as surviving the flood and/or providing some degree of flood protection, but those instances should be limited (e.g., to sheltered waters, where the bulkhead may be stable during 1-percent-annual-chance flood conditions).

Many seawalls, revetments, and (some) bulkheads may be recognized on flood hazard maps if analysis based on the evaluation criteria in Subsection D.2.10.2 shows they will remain intact during the 1-percent-annual-chance storm (in some cases, even if overtopped). These structures may provide total or limited protection against flooding, erosion, and waves, depending upon their location, strength, and dimensions.

D.2.10.3.1 Failure and Removal of Coastal Armoring Structures

In the event that a coastal structure is determined to fail, the Mapping Partner shall remove the structure entirely from the analysis transect, or estimate the partial collapse of the structures where appropriate (see Subsection D.2.10.3.2). If the failed structure is removed entirely, the remaining soil profile should be altered to achieve its likely slope immediately after structure failure. Information on slopes behind failed structures is limited. These slopes may vary from 1 on 100 (v:h) for unconsolidated sands, to 1:1 or steeper for consolidated material landward of the failed structure.

The post-failure slope for this analysis should be in the range of 1:1 to 1:1.5 (v:h). Note that the post-failure slope may not necessarily match the long-term stable slope, but will serve as the

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registered in the subject State. FEMA has distributed the *Coastal Structure Form* (MT-2, Form 5, available at <http://www.fema.gov/pdf/fhm/mt2_f5.pdf>) to evaluate coastal structures as the basis for map revisions.

basis for subsequent site-specific, event-based erosion (D.2.9), wave height (D.2.7), wave runup (D.2.8), and wave overtopping (D.2.8) analyses.

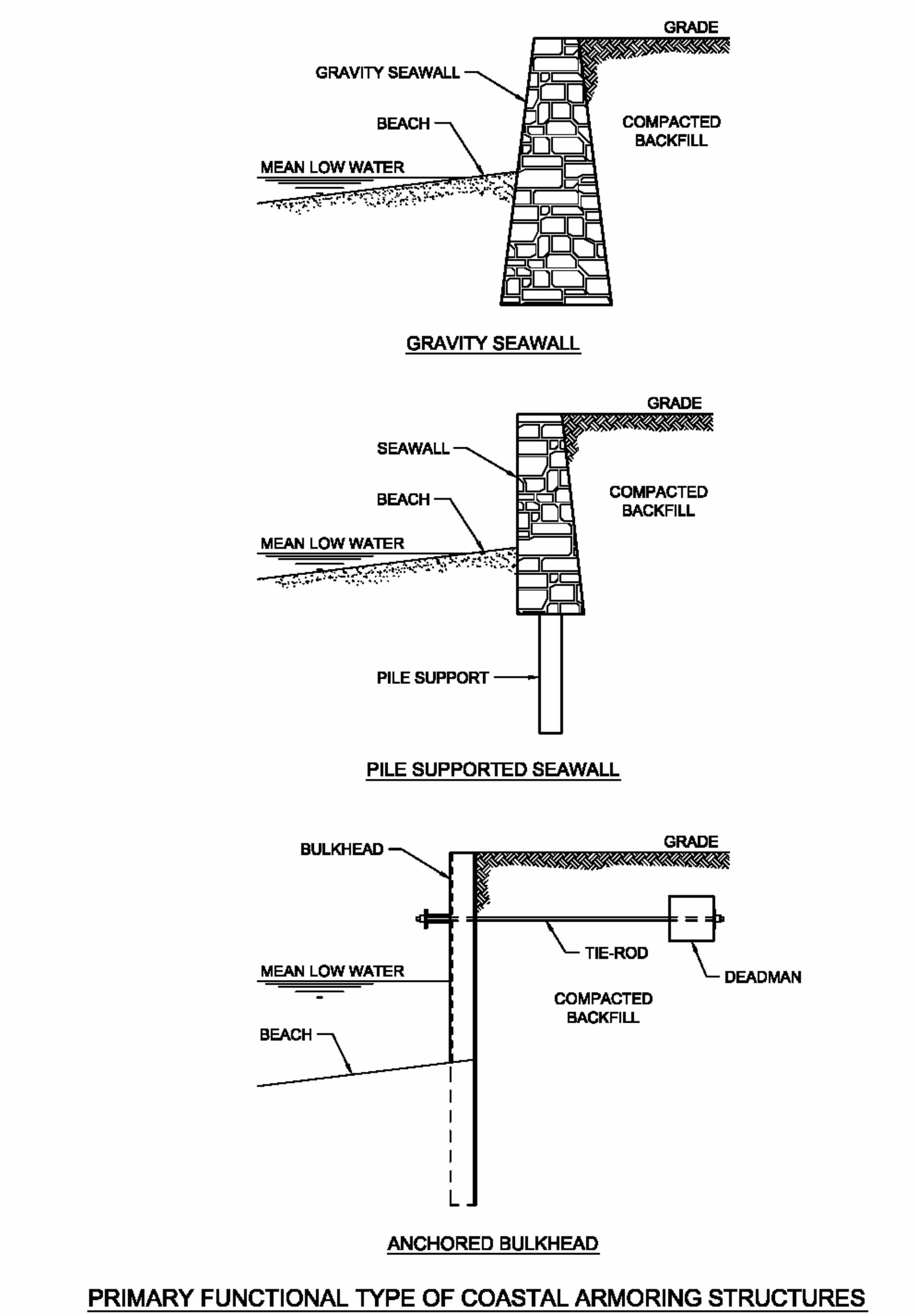

Figure D.2.10-1a. General Classification of Coastal Armoring Structures

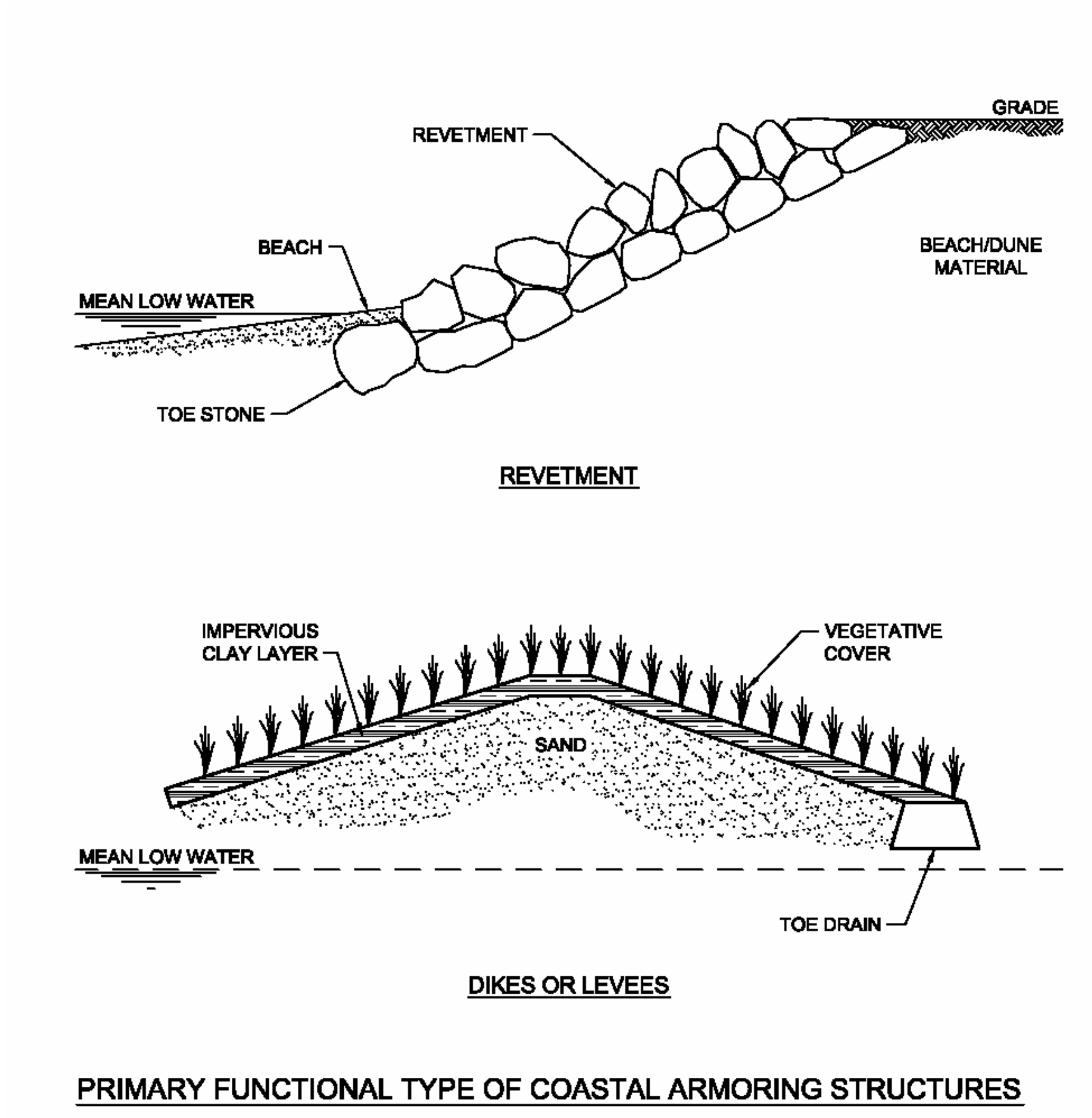

Figure D.2.10-1b. General Classification of Coastal Armoring Structures

D.2.10.3.2 Partial Failure of Coastal Armoring Structures

Coastal structures are frequently constructed of either concrete or large individual armor units. Consequently, it is improbable that the structural components will be completely destroyed or removed from the vicinity during the 1-percent-annual-chance flood. It may be appropriate to assume partial failure of such structures and to model accordingly.

A recommended simple geometric approach for approximating partial failure of a vertical or near-vertical coastal armoring structure is as follows (see Figure D.2.10-2):

1. Estimate toe scour at the subject structure based upon the methods described in the CEM (USACE, 2003).
2. Assume the structure fails and falls into a rough, porous slope at 1:1.5 (v:h).
3. Extend the 1:1.5 failure slope from the depth of scour at the structure toe landward to the point where it intersects the existing grade.

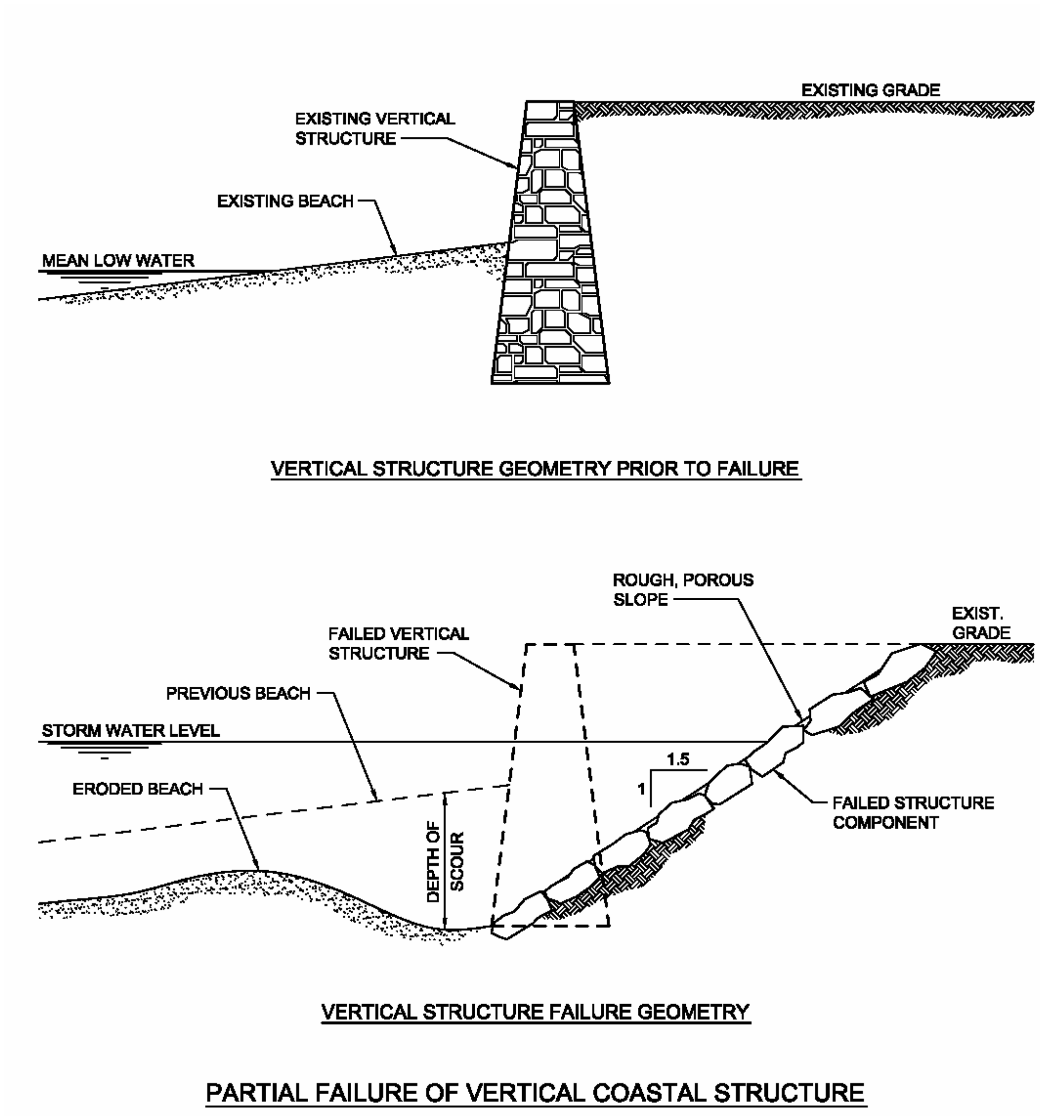

Figure D.2.10-2. Partial Failure of Vertical Coastal Structure

A recommended approach for approximating partial failure of a sloping revetment (due to undermining at the toe, or to collapse at the top due to erosion behind the structure) is as follows (see Figure D.2.10-3):

1. Assume scour at the base of the structure is equal to the depth of the armor layer.
2. Assume the structure will collapse in place into a triangular section throughout the structure footprint, with side slopes equal to the original structure slope.
3. Assume the landward side of the failed configuration will be half exposed and half buried. Approximate the soil slope landward from the failed structure at a slope in the range of 1:1 to 1:1.5 (v:h).

After determining an appropriate failure configuration, the Mapping Partner shall conduct overland wave height propagation (D.2.7) and wave runup (D.2.8) analyses upon the failed structure, as discussed in preceding subsections. The Mapping Partner shall select an appropriate roughness factor when conducting runup and overtopping analyses on the failed structure.

In some cases, the assumed failed slope may result in the undermining of buildings located landward of the coastal structure. If this occurs, the building shall be removed from the analysis transect and not considered during subsequent wave effects modeling.

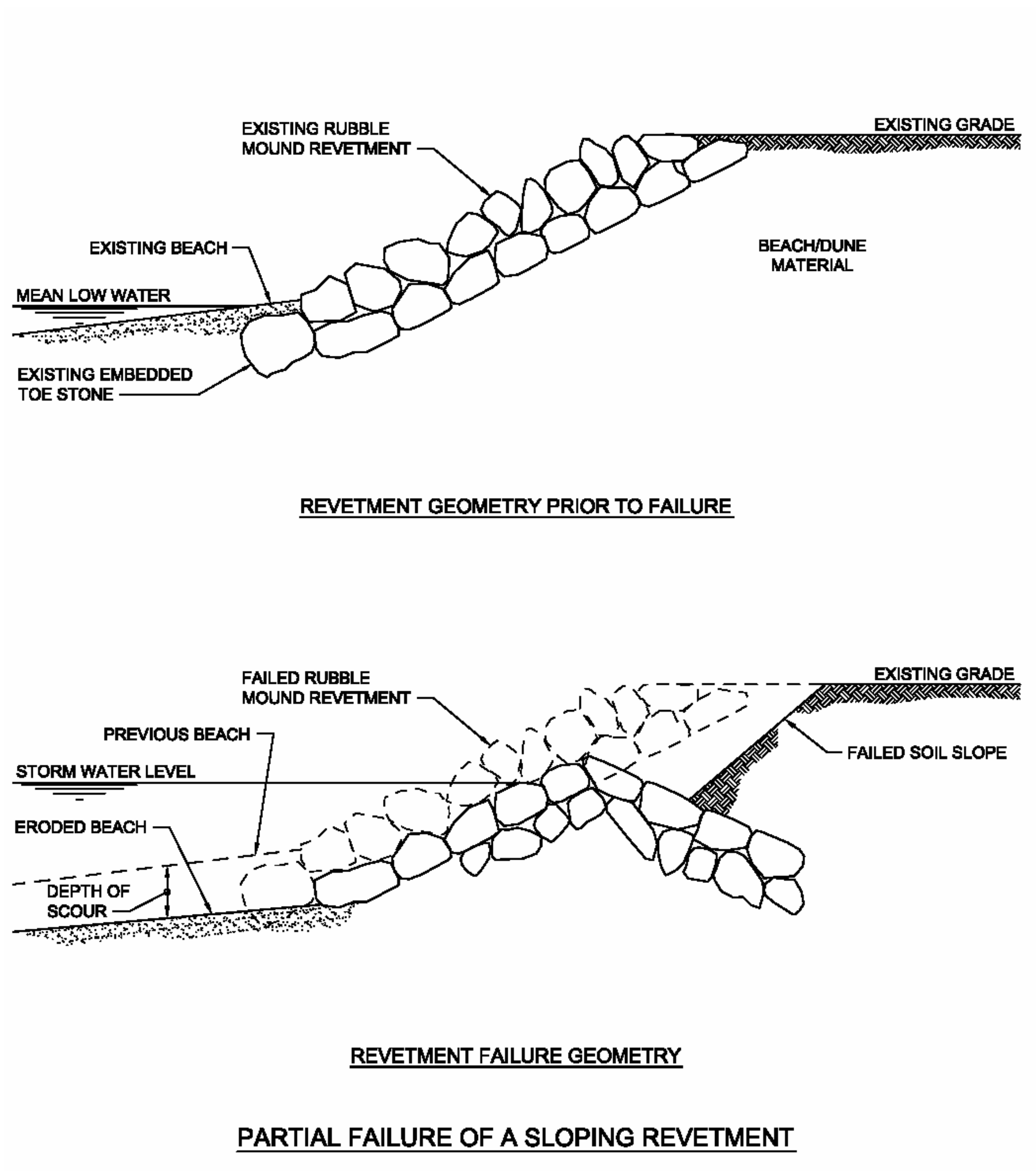

Figure D.2.10-3. Partial Failure of a Sloping Revetment

D.2.10.3.3 Buried Coastal Structures

In some instances, coastal structures may be covered or buried by sediments and not readily observable during an FIS site reconnaissance. For example, Figure D.2.10-4 shows two photographs of nearly buried structures on the Atlantic coast. The top photo shows a revetment, the bottom a buried seawall. This is one example where a dune is building up in front of the structures and will one day cover the structures. Some buried structures are of a size and construction to possibly affect coastal flood hazards, and should—like exposed structures—be considered during the FIS.

The Mapping Partner is responsible for determining whether buried coastal structures exist within the study area during the preliminary investigation phase of the FIS. The Mapping Partner should include information from the community and carefully review aerial photographs of the study area to locate buried structures.

Once the Mapping Partner has determined that a coastal structure is likely buried at a site, the next steps are to collect information about the structure and follow the study process outlined in Figure D.2.10-5. The erosion analysis will result in one of the following two scenarios: 1) the buried structure will remain buried during the 1-percent-annual-chance flood (see Figure D.2.10-6), or 2) the buried structure will be exposed by the 1-percent-annual chance flood (see Figure D.2.10-7).

Note that the buried structure study process need not be followed unless the presence of buried structures is known or is highly likely. The *Guidelines and Specifications* do not require field investigations to identify buried coastal structures. There may be some instances where limited field work (such as soil probes to locate the structure) might be useful, but this should be limited to cases where large buried structures are known to exist.

Figure D.2.10-4. Examples of a Buried Coastal Structure that Could Affect Flood insurance risk zones and BFEs

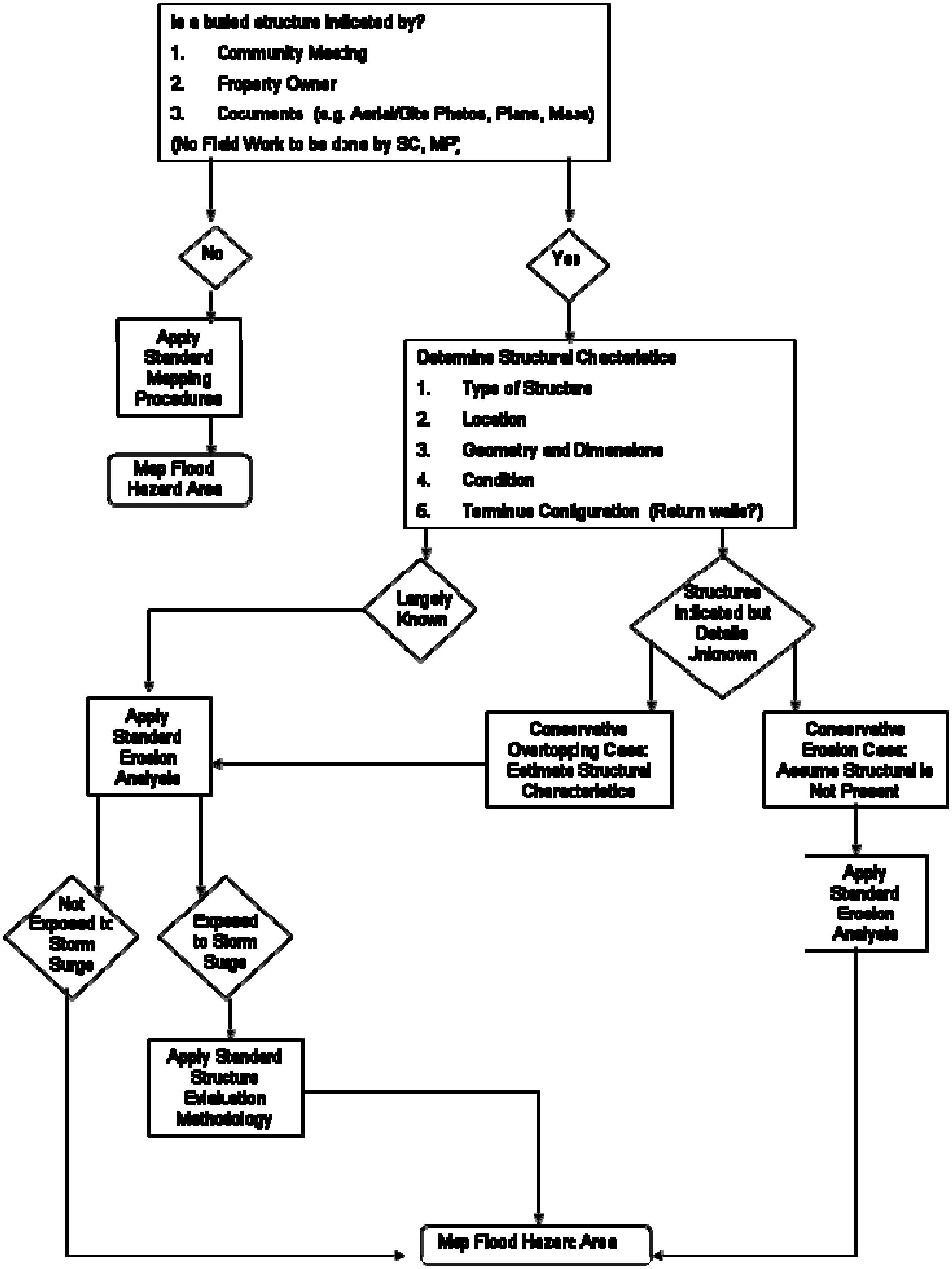

Figure D.2.10-5. Methodology for Evaluating Buried Coastal Structures

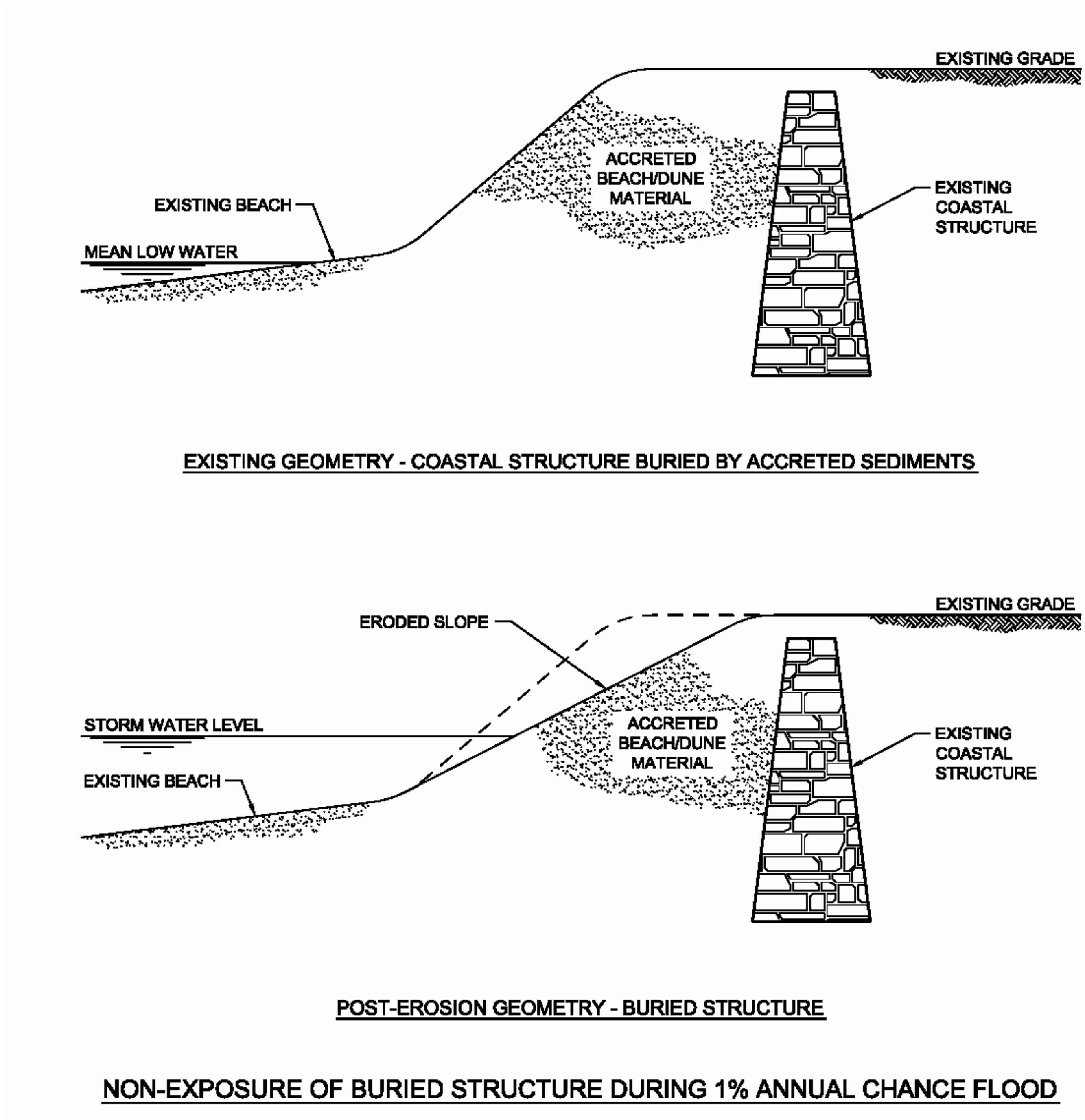

Figure D.2.10-6. Buried Structure Remains Buried During 1-Percent-Annual-Chance Flood

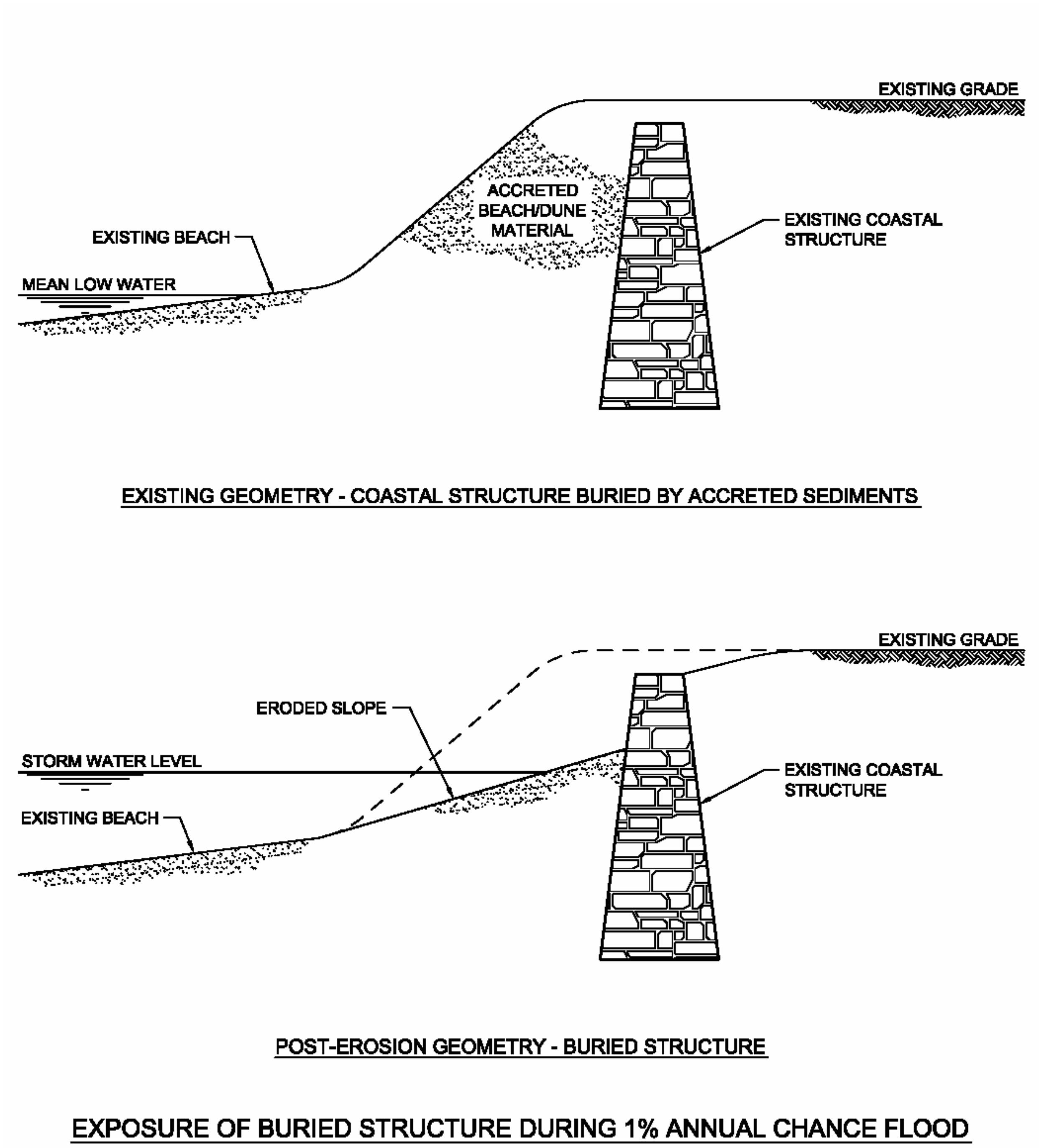

Figure D.2.10-7. Buried Structure Exposed During 1-Percent-Annual-Chance Flood

D.2.10.3.4 Coastal Levees

Levees are man-made structures (usually earthen embankments that may or may not have their slopes and crest armored) that prevent flooding of low-lying areas. A *levee system* consists of a levee, or levees, or a floodwall and associated structures, such as closure and drainage devices, that are constructed and operated to prevent flooding of interior areas. FEMA has issued guidance on levees in Procedure Memorandum Number 34 "Interim Guidance for Studies including Levees" dated August 22, 2005. The Mapping Partner should consult Procedure Memorandum Number 34 for guidance in any new study or revision in which a levee structure influences the BFEs or hazard mapping.

For coastal levees or levee systems to be recognized as providing protection against the base flood by the NFIP and incorporated into flood hazard maps, they must be designed, constructed, operated, and maintained to resist erosion and prevent any flooding or wave overtopping landward of the levee crest during 1-percent-annual-chance flood conditions. The levee or levee system also must be certified as providing that level of protection. NFIP regulations (44 CFR Part 65.10) detail the requirements for a levee to be recognized as providing protection from flooding, including a freeboard requirement specific to coastal levees — the crest elevation of the levee must be elevated at least 3 feet above the 1-percent-annual-chance total stillwater elevation (MWL), and 1 foot above the 1-percent-annual-chance wave height or the maximum wave runup elevation (whichever is greater)[15]. Data to support that a given levee system complies with the structural requirements described in 44 CFR Part 65.10 must be certified by a registered professional engineer. In lieu of these structural requirements, a Federal agency with responsibility for levee design may certify that a levee has been adequately designed and constructed to provide protection from the 1-percent-annual-chance flood. Occasionally, exceptions to the minimum coastal levee freeboard requirement may be approved. Appropriate engineering analyses demonstrating adequate protection with a lesser freeboard must be submitted to support a request for such an exception. The material presented must evaluate the uncertainty in the estimated base flood loading conditions. Particular emphasis must be placed on the effects of wave attack and overtopping on the stability of the levee. Under no circumstances, however, will a freeboard of less than 2-feet above the 1-percent-annual-chance total stillwater surge elevation (MWL) be accepted. Additional guidance for evaluating levees can be found in Appendix H of the *Guidelines and Specifications*.

The USACE utilizes a risk-based analysis to evaluate flood damage reduction projects such as levees. Freeboard requirements are not used in the risk-based approach but rather a level of assurance should be achieved that the levee provides protection from the 1-percent-annual-chance flood. Assurance is defined as the percent chance that flood waters associated with the 1-percent-annual-chance flood will not inundate any area landward of a levee system that would be inundated without benefit of the levee system. The levee must at least be of such height that there is a 90 percent assurance of containing the 1-percent-annual-chance wave height or maximum wave runup (whichever is greater) associated with the 1-percent-annual-chance stillwater elevation at the site. Risk-based analysis that demonstrates a 95 percent assurance of containing

[15] To be recognized by the NFIP, riverine levees require a minimum of 3 feet of freeboard above the 1-percent-annual-chance flood elevation and a minimum of 4 feet of freeboard within 100 feet of locations where the flow is constricted (e.g., a bridge). In addition, the upstream end of the levee must provide an additional 0.5 foot of freeboard added to the minimum.

the 1-percent-annual-chance wave height or maximum wave runup (whichever is greater) associated with the 1-percent-annual-chance stillwater elevation at the site is acceptable justification for the reduction in minimum freeboard to 2 feet as provided for in 44 CFR Part 65.10.

For a coastal levee to be considered as the basis of a map revision, the "Riverine Structure Form" (MT-2, Form 3, available at <http://www.fema.gov/pdf/fhm/mt2_f3.pdf>) must be completed in addition to the "Coastal Structure Form."

For consideration of levees that are subject to both coastal and riverine conditions, the Mapping Partner shall determine freeboard requirements using water levels determined using the methods contained in Subsection D.2.4 and Subsection D.2.5. Because BFEs are required to be mapped to within a 0.5-foot tolerance (Guidelines and Specifications Appendix C.6.3), the combined total stillwater (MWL) and riverine flood profile shall be adjusted to an inland extent where the effects of waves and/or runup diminish to 0.5 foot or less. The resulting flood profile shall be compared to the crest elevations of flood protection along the combined tidal-river reach to determine whether interior areas are sufficiently protected.

D.2.10.3.4.1 Levee Failure and Removal

Current FEMA policy states that in instances where levees cannot meet the requirements for recognition by the NFIP, the levees shall be "removed" from the analysis. Two scenarios are considered here: 1) a single levee on an analysis transect, and 2) multiple levees along an analysis transect.

Single Levee Case: If a community cannot provide the Mapping Partner with evidence that a levee is certified as meeting FEMA's requirements in 44 CFR 65.10, then the Mapping Partner shall remove the levee from subsequent analyses. In such a case, the Mapping Partner shall:

- Modify the topography along the transect by erasing the levee cross section and joining the ground elevations on each side of the levee with a straight line.
- If the Mapping Partner determines that the failed levee provides substantial (but not complete) protection against incident wave action during 1-percent-annual-chance flood conditions, the Mapping Partner shall assume no wave action penetrates beyond the failed levee, and that only stillwater flooding (tide + wind setup) and locally generated waves (i.e., waves generated in the region behind the levee) affect the flooded area behind the levee.
- If the Mapping Partner determines that the failed levee provides minimal protection against incident wave action during 1-percent-annual-chance flood conditions, the Mapping Partner shall consult with the FEMA Study Representative to determine whether subsequent analyses should assume incident wave action penetrates beyond the failed levee.

Multiple Levee Case: If a community cannot provide the Mapping Partner with evidence that the outer levee is certified as meeting FEMA's requirements in 44 CFR 65.10, then the Mapping

Partner shall remove the outer levee from subsequent analyses. In such a case, the Mapping Partner shall do one of the following:

- Modify the topography along the transect by erasing the outer levee cross-section and joining the ground elevations on each side of the levee with a straight line.
- If the Mapping Partner determines that the failed outer levee provides substantial (but not complete) protection against incident wave action during 1-percent-annual-chance flood conditions, the Mapping Partner shall assume no wave action penetrates beyond the outer levee, and that only mean water flooding (tide + wave setup) and locally generated waves (i.e., waves generated in the region behind the levee) affect the next landward levee (see Figure D.2.10-8).
- If the Mapping Partner determines that the failed outer levee provides minimal protection against incident wave action during 1-percent-annual-chance flood conditions, the Mapping Partner shall consult with the FEMA Study Representative to determine whether subsequent analyses should assume incident wave action penetrates beyond the failed outer levee.
- The Mapping Partner shall repeat steps 1 through 3 for each additional levee along the transect, for which the community cannot supply certification.

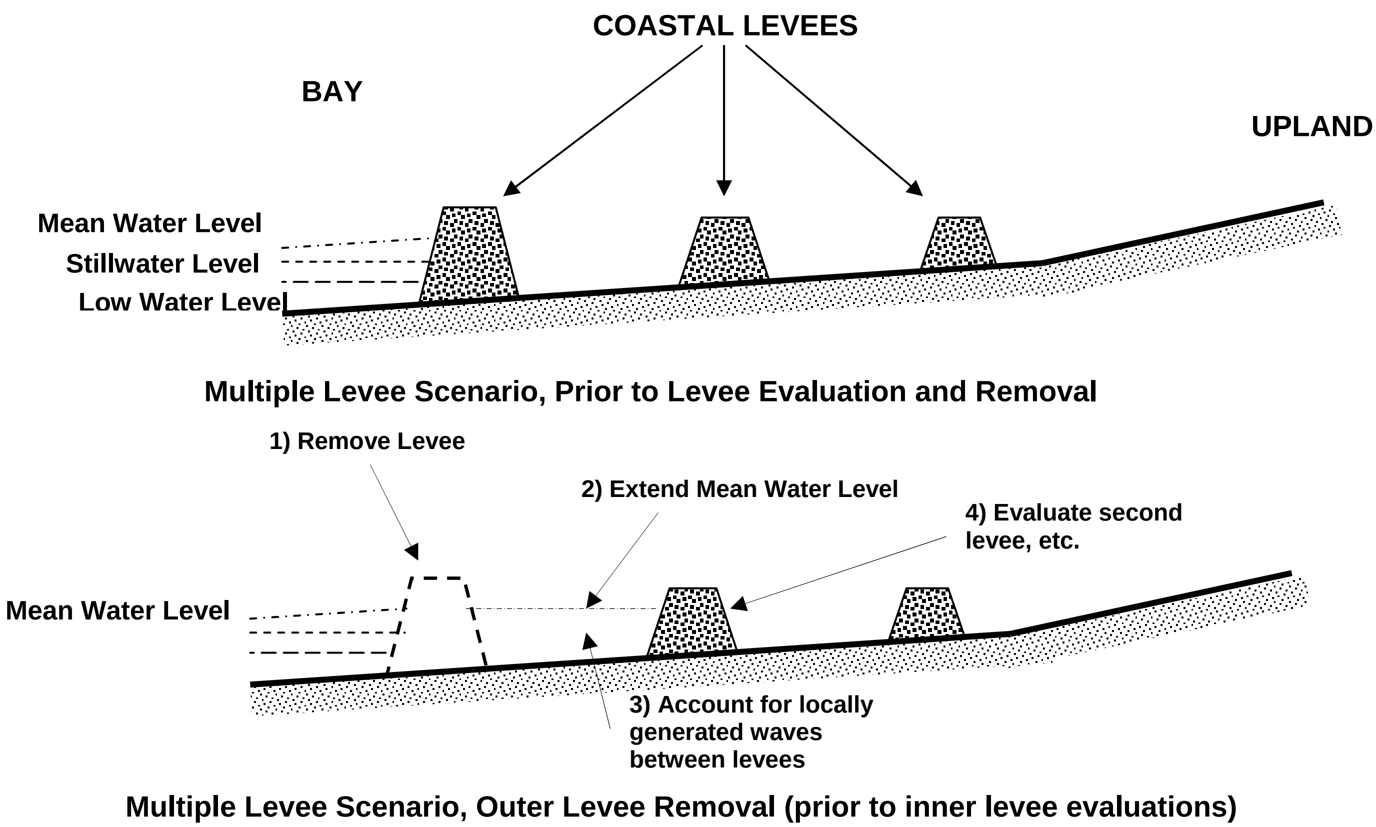

Figure D.2.10-8. Levee Removal, Multiple Levee Situation

D.2.10.3.5 Operation and Maintenance

Both the FEMA memorandum (April 23, 1990) (D.2.10.2.1) and the NFIP regulations indicate that an operation and maintenance plan is required as part of the certification that a coastal structure will withstand the base flood. At a minimum, the plan must document the formal procedure to maintain the stability, height, and overall integrity of the structure and its associated structures and systems.

The NFIP regulations (44 CFR Part 65.10) require that all maintenance activities must be under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of the community participating in the NFIP that must assume ultimate responsibility for maintenance. Often, the aforementioned government entities are unable to take responsibility for maintenance of private structures. However, a government agency can recognize private property owners as the responsible party for maintenance of an existing structure.

For the purposes of an FIS, the Mapping Partner shall ascertain (through discussions with the community and property owners) whether operation and maintenance plans exist for coastal structures that are expected to remain intact during 1-percent-annual-chance flood conditions. Mapping Partners may not have sufficient resources and time to conduct detailed evaluations of the operation and maintenance of each coastal structure within the study area. In such cases, the Mapping Partner shall make an engineering judgment about the adequacy of structure operation and maintenance. The Mapping Partner must document data, materials, and assumptions associated with the flood hazard determinations related to structure operation and maintenance. Communities and property owners should be made aware that these evaluations are for mapping purposes only.

D.2.10.4 FIS Treatment of Beach Stabilization Structures

If a Mapping Partner chooses to evaluate beach stabilization structures (e.g., groins, jetties, sills, or similar structures) during an FIS, the following approach is recommended:

- Identify any beach stabilization structures during the FIS reconnaissance phase
- Use historical evidence and engineering judgment to determine whether the structures (or similar structures nearby) have been damaged or detached (during prior storms or gradually over time)
- Document prior damage to the stabilization structures and any resulting shoreline recession attributable to the structural damage
- Notify the FEMA Study Representative if the Mapping Partner intends to remove beach stabilization structures or reduce their effects during the FIS analyses. Obtain FEMA concurrence before proceeding with the following steps.
- Use historical evidence and engineering judgment to predict the likely shoreline configuration (in plan view and elevation) if the structures fail during 1-percent-annual-chance flood conditions.

- Subject the modified shoreline and profile to typical FIS analyses (e.g., event-based erosion analysis, wave runup and overtopping analysis, and wave height analysis).

- Note that in the case of some stabilization structures, it is unlikely that their failure will require "removal" from analysis transects; the effects of the structure failure on the shoreline configuration, however, should be considered by the analyses.

D.2.10.5 FIS Treatment of Miscellaneous Structures

Current FEMA guidance does not address the effects of miscellaneous structures (e.g., piers, port and navigation structures, bridges, culverts, tide gates, etc.) on coastal flood hazard analysis and mapping. This subsection provides general guidance for identifying and analyzing the effects of miscellaneous structures on flooding in sheltered water areas as follows:

- The Mapping Partner shall identify structures – in addition to the coastal armoring and beach stabilization structures addressed above – that could exert a significant influence on nearshore waves and currents, coastal sediment transport, or ponding in backshore areas, during 1-percent-annual-chance flood conditions, particularly in sheltered waters. This should be done during the FIS reconnaissance phase.

- Once identified, the Mapping Partner shall use historical evidence, other readily available data, and engineering judgment to determine whether the miscellaneous structures are likely to survive the 1-percent-annual-chance flood conditions. If the structures are likely to fail, then they (and their effects on the shoreline and flooding) should be removed from subsequent analyses.

- The Mapping Partner shall notify the FEMA Study Representative as to how he/she intends to address miscellaneous structures and their effects during the FIS analyses, and obtain FEMA concurrence before proceeding.

D.2.10.5.1 Piers, Navigation Structures, and Port Facilities

The Mapping Partner shall review navigation charts, aerial photographs, and other information relative to piers, navigation structures, and port facilities (including dredged channels) that may affect the propagation and transformation or dissipation of waves within a sheltered water body, or that may affect littoral sediment transport. The Mapping Partner shall consider the range of possible effects of these structures and facilities during 1-percent-annual-chance flood conditions, using readily available data and site characteristics as a guide.

The Mapping Partner shall verify basic structure and facility information with local agencies and communities to determine the location, extent, and influence of these features. If there is any uncertainty concerning major features and their potential effects on upland flood hazards, limited field surveys or additional data collection shall be considered to augment existing data.

D.2.10.5.2 Bridges, Culverts, and Tide Gates

The shorelines of sheltered waters are often paralleled by roads and railroads in backshore areas. The effect of these structures on coastal flooding can be most pronounced where they intersect tidally influenced creeks, river channels, and floodplains. The Mapping Partner shall consider the presence and influence of roadways, railways, embankments and abutment fill, and bridge piers on flood hazards during 1-percent-annual-chance flood conditions.

The Mapping Partner shall identify the location and condition of culverts, tide gates, and other flow control structures in the vicinity of the study site and evaluate their potential to affect interior flood elevations. Design calculations and reports for individual culverts and tide gates, and storm drainage master plans for larger drainage systems shall be obtained and reviewed by the Mapping Partner to understand design criteria and provide data for hydraulic calculations and hazard zone delineation.

D.2.10.6 Data Requirements

The Mapping Partner shall obtain documentation for each coastal structure that could provide protection during 1-percent-annual-chance flood conditions, or significantly affect flood hazards in the study area. The documentation shall provide all information necessary to evaluate the structure according to the criteria set forth in Subsection D.2.10.2.1. Documentation should include, but is not limited to, the following:

- As-built design parameters: structure type, location, layout, dimensions, crest elevation of structure, etc.;
- Dominant site particulars (e.g., local water depth, tide, surge and wave conditions, erosion rate, sediment characteristics and geotechnical conditions, debris hazards, and ice climate);
- Construction materials and present integrity;
- Historical record for structure including: construction date, plans, and specifications; recent inspection reports and photographs; maintenance plan and responsible party; and dates and descriptions of damage, repairs, and modifications; and
- Clear indications of effectiveness or ineffectiveness.

The Mapping Partner shall develop much of this information through office activity, including a careful review of aerial and site photographs, reports and information provided by the community and property owners, and other readily available information. In the case of some major coastal structures, site inspection would be advisable to confirm preliminary judgments.

Note that the level and detail of the structure and site data collected should be consistent with the level of analysis undertaken by the Mapping Partner. An analysis based on engineering judgment, or multiple analyses assuming different structure responses during 1-percent-annual-chance flood conditions (e.g., structure survives intact, partial failure, complete failure) will require less detailed and precise information than a structural engineering and geotechnical evaluation of a coastal structure.

D.2.10.7 Study Documentation

If coastal structures are present in the study area, the Mapping Partner shall document the data, methods, and procedures used to evaluate the likelihood that the structures will survive 1-percent-annual-chance flood conditions (D.2.10.2.1). This documentation shall include any assumptions or approximations used in the analyses. The same documentation shall be required in the event that coastal structures are indicated by information collected during the FIS, but are apparently buried and not visible during the study.

The Mapping Partner shall document the results of all analyses of coastal structures conducted for the FIS. In cases where the study contractor could not determine whether a given structure would survive the 1-percent-annual-chance flood intact, and where multiple analyses were conducted for the structure (i.e., intact condition, failed condition, and removed from the analysis transect), the Mapping Partner shall document each analysis and record the structure condition that was used to map flood insurance risk zones and BFEs. This information will be useful in the event a map revision is requested based upon a structure condition different from that used as the basis for the FIRM. Subsection D.2.12.2 describes the intermediate data submission procedures during which the documentation and analysis will be submitted to FEMA for review and the requirements for the preparation of a Technical Support Data Notebook (TSDN). The TSDN will contain the data needed by FEMA or the community to reconstruct or defend the study results on technical grounds.

D.2.11 Mapping of Flood Insurance Risk Zones and Base Flood Elevations

This subsection provides guidance to Mapping Partners on the delineation of coastal flood insurance risk zones and BFEs.

D.2.11.1 Review and Evaluation of Basic Results

Before mapping the flood elevations and flood insurance risk zones, the Mapping Partner should review results from the models and assessments from a common-sense viewpoint and compare them to available historical flood data. When using models, there is the potential to forget that transects represent real shorelines being subjected to high water, waves, and winds. Familiarity and experience with the coastal area being modeled, or with similar areas, should provide an idea of a "reasonable" result.

The main point to be emphasized is that the results should not be blindly accepted. There are many uncertainties and variables in coastal processes during an extreme flood, and many possible adjustments to methodologies for treating such an event. The validity of any model is demonstrated by its success in reproducing recorded events. Therefore, the model results must be in basic agreement with past flooding patterns, and historical data must be used to evaluate these results.

It would be very convenient if data from a storm closely approximating the 1-percent-annual-chance flood were available, but this is seldom the case. Although most historical flood data are for storms less intense than a 1-percent-annual-chance flood, these data will still indicate, at a minimum, the areas that should be within a flood zone. For instance, if a storm that produced a flood below the 1-percent-annual-chance flood elevation generally caused structural damage to houses 100 feet from the shoreline, a "reasonable" VE Zone width must be at least 100 feet. Similarly, houses that collected flood insurance claims for the same storm (without building foundation or structural damages) should at least be located in an AE, AH, or AO Zone. If the analyses of the 1-percent-annual-chance flood produce flood zones and elevations indicating lesser hazards than those recorded for a more common storm, the analyses should be reevaluated. One possible explanation for changes in flood patterns because the historical flood event might be that a new coastal structure now acts to reduce flood hazards in the local area.

If there are indications that a reevaluation is needed, the Mapping Partner should determine whether the results of the assessment are appropriate. The Mapping Partner should attempt to compare all aspects of the coastal hazard assessment to past effects, whether in the form of data, profiles, photographs, or anecdotal descriptions. The Mapping Partner should examine other data input to the assessments for wave effects (wave setup, wave height, wave runup, and wave overtopping). This includes checking that the stillwater levels are correct and that the results of wave analyses are consistent with the historical data. The Mapping Partner should use judgment and experience to project previous storm effects onto the 1-percent-annual-chance conditions and to ensure that the coastal assessment results are consistent with previous observed events.

D.2.11.2 Identification of Flood Insurance Risk Zones

The Mapping Partner should identify the flood insurance risk zones and BFEs, including the wave effects, to be identified on each transect plot before delineating the flood insurance risk zones on the work maps. The existing topography, eroded topography, presence of PFDs, coastal structure effects, combined wave analyses (wave runup, overtopping, and overland propagation) are all important for the proper identification of flood insurance risk zones. Hazard zones that are generally mapped in coastal areas include VE, AE, AH, AO, and X.[16]

D.2.11.2.1 VE Zone

VE Zones are coastal high hazard areas where wave action and/or high-velocity water can cause structural damage during the 1-percent-annual-chance flood. VE Zones are identified using one or more of the following criteria for the 1-percent-annual-chance flood conditions:

- The ***wave runup zone*** occurs where the (eroded) ground profile is 3.0 feet or more below the 2-percent wave runup elevation.
- The ***wave overtopping splash zone*** is the area landward of the crest of an overtopped barrier, in cases where the potential 2-percent wave runup exceeds the barrier crest elevation by 3.0 feet or more (ΔR>3.0 feet). (See Subsection D.2.8.2.)
- The ***breaking wave height zone*** occurs where 3-foot or greater wave heights could occur (this is the area where the wave crest profile is 2.1 feet or more above the total stillwater level).
- The ***primary frontal dune zone***, as defined in 44 CFR Section 59.1 of the NFIP regulations (see Subsection D.2.9.3.1 of this document for more details).

The actual VE Zone boundary shown on the FIRM is defined as the farthest inland extent of any of the four criteria listed above. VE Zones are subdivided into elevation zones, and whole-foot BFEs should be assigned (see Subsection D.2.11.4).

When the potential runup is at least 3.0 feet above the barrier crest (criterion 2), a VE Zone is delineated landward of the barrier. The BFE for that VE Zone is capped at 3 feet above the crest of the barrier. When the runup depth in excess of the barrier crest is 0.1 to 1.5 feet, the VE Zone BFE is the runup elevation (rounded to the nearest whole foot), and an AO Zone with a depth of 1 foot should be mapped landward until another flooding source is encountered (Zone AE) or the floodplain limit is reached (Zone X). Similarly, for a runup depth of 1.5 to 2.9 feet above the barrier crest, the VE Zone BFE is the runup elevation (rounded to the nearest whole foot). In this case, however, an AO Zone with a depth of 2 feet should be mapped, then transitioned landward into an AO Zone with a depth of 1 foot, and into subsequent flood insurance risk zones, if any.

VE Zone criterion 3, the designation of a 30-foot splash zone, should be applied to both vertical walls and sloping barriers upon the identification of wave overtopping hazards (D.2.8.2).

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[16] For a complete list of flood insurance risk zones, refer to Volume 1, Section 1.4.2.7, of the *Guidelines and Specifications*.

Delineation of the landward limit of the VE Zone based on the PFD (criterion 4) requires detailed topographic data and engineering judgment. Identifying the PFD heel, "the point where there is a distinct change from a relatively steep slope to a relatively mild slope" (per Section 59.1 of the NFIP regulations) can be particularly challenging when there are inadequate topographic data and/or encroachments into the dune ridge system that obscure this slope change.

The Mapping Partner should review the available topographic data and, if necessary, conduct field verification to delineate PFDs in the study area. Previous flood insurance studies may have identified PFDs and used these features as the basis of the effective FIRM's VE Zone; such information should be reviewed to aid in locating PFDs that exist at the time of the restudy. The Mapping Partner is cautioned to carefully evaluate any preexisting methods for PFD heel delineation to ensure that a reasonable approach is applied to the study area.

It is possible that a PFD may be identified landward of a shore protection structure. If the structure can be certified per the criteria in the April 23, 1990, FEMA memorandum (FEMA, 1990), *Criteria for Evaluating Coastal Flood Protection Structures for National Flood Insurance Program Purposes* (see Subsection D.2.10.2.1), the VE Zone should be delineated based on the wave analyses for that transect (criteria 1-3, as applicable), not on the PFD heel. If the structure cannot be certified and will partially or completely fail during the base flood, the VE Zone should be mapped to the PFD landward heel. Certified structures with a crest at or below the dune toe or the 10-year flood level will provide little more than protection from toe scour to a dune and will not protect inland areas or dunes from hazardous flood conditions. Low-crested structures would warrant PFD VE Zone determinations landward if deemed appropriate based on wave runup and wave height propagation analysis.

In all cases where the PFD is the basis of the VE Zone, the BFE to be applied will be the wave height or wave runup elevation encountered at the dune face; see Examples 1 and 2 in Subsection D.2.11.5 (Figures D.2.11-3 and D.2.11-4) for more information.

D.2.11.2.2 AE Zone

AE Zones are areas of inundation by the 1-percent-annual-chance flood, including areas with wave heights less than 3.0 feet and runup elevations less than 3.0 feet above the ground. These areas are subdivided into elevation zones, and BFEs are assigned. The AE Zone will generally extend inland to the limit of the 1-percent-annual-chance flood SWEL.

D.2.11.2.3 AH Zone

AH Zones are areas of shallow flooding or ponding, with average water depths between 1.0 foot and 3.0 feet. These areas are usually not subdivided, and a BFE is assigned.

D.2.11.2.4 AO Zone

AO Zones are areas of sheet-flow shallow flooding, or where the potential runup is less than 3.0 feet above an overtopped barrier crest (ΔR<3.0 feet). The sheet flow in these areas will either flow into another flooding source (AE Zone), result in ponding (AH Zone), or deteriorate because of ground friction and energy losses to merge into the X Zone. AO areas are designated with 1-, 2-, or 3-foot depths of flooding.

D.2.11.2.5 X Zone

X Zones are areas above the 1-percent-annual-chance flood level. On the FIRM, a shaded X Zone area is inundated by the 0.2-percent-annual-chance flood, and an unshaded X Zone area is above the 0.2-percent-annual-chance flood.

D.2.11.3 Wave Envelope

The seaward portion of the wave envelope is a combination of the potential wave runup elevation and the controlling wave crest elevation profile. The wave crest elevation profile is plotted along a transect (from the 0.0 map datum elevation landward) based on the results of the WHAFIS model or other methodology. A horizontal line is extended seaward from the potential wave runup elevation to its intersection with the wave crest profile to obtain the wave envelope, as shown in Figure D.2.11-2. If the runup elevation is greater than the maximum wave crest elevation, the wave envelope will be represented as a horizontal line (extending to the elevation 0.0 location on the transect) at the runup elevation, and the BFE for mapping purposes will be based on that elevation. Conversely, if the wave runup is negligible, the wave crest elevation profile becomes the wave envelope.

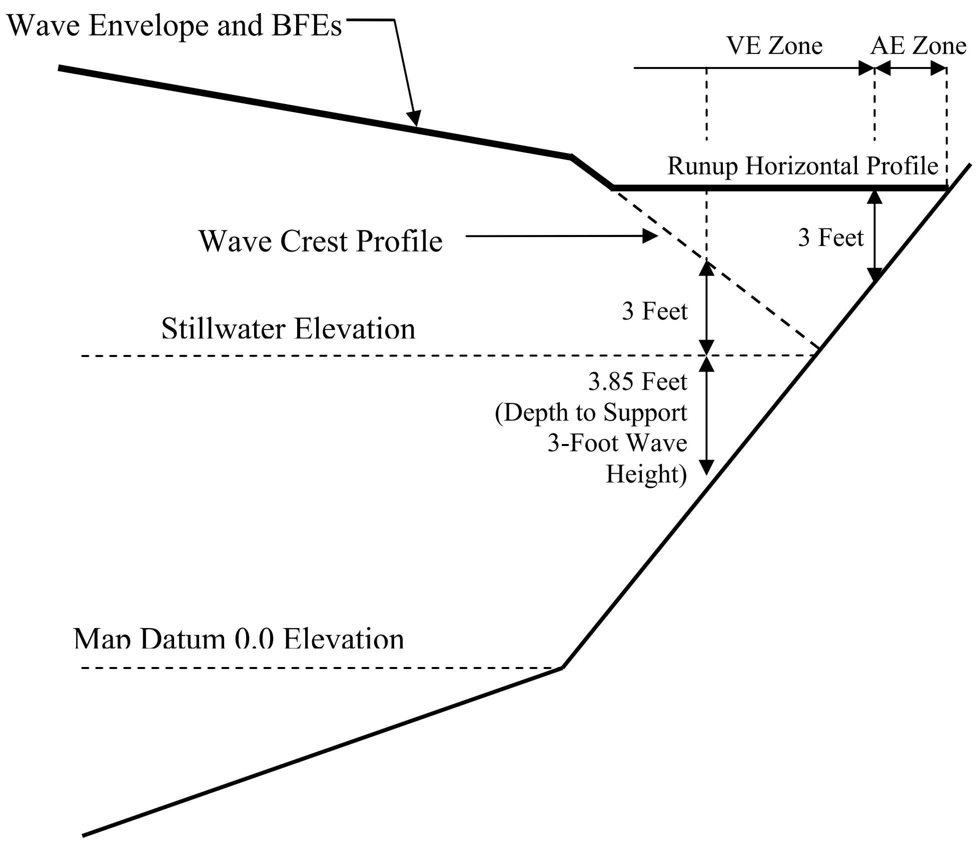

Figure D.2.11-2. Seaward Portion of Wave Envelope Based on Combination of Nearshore Crest Elevations and Shore Runup Elevation (figure not to scale)

D.2.11.4 Criteria for Flood Boundary and Hazard Zone Mapping

The first step in identifying the flood insurance risk zones along a transect is locating the inland extent of the VE Zone, also known as the VE/AE boundary. The mapped VE/AE Zone boundary is based on the most landward limit of the four criteria outlined in Subsection D.2.11.2.1. The Mapping Partner should extend the AE Zone from the VE/AE boundary to the inland limit of 1-percent-annual-chance inundation, which is a ground elevation equal to the potential runup elevation, or the 1-percent-annual-chance SWEL if runup is negligible. The Mapping Partner may designate additional areas of 1-percent-annual-chance flooding caused by wave overtopping sheet flow and shallow flooding or ponding as the AO Zone and/or the AH Zone. The Mapping Partner should label all areas above 1-percent-annual-chance inundation as the X Zone (shaded for areas affected by the 0.2-percent-annual-chance flood and unshaded for areas above the 0.2-percent-annual-chance flood level).

The Mapping Partner should then subdivide the VE and AE Zone areas into elevation zones, with whole-foot BFEs assigned according to the wave envelope. Generally, the VE Zone is subdivided first. Initially, the Mapping Partner should mark the location of all elevation zone boundaries on a transect. Because whole-foot BFEs are being used, these should always be mapped at the location of the half-foot elevation on the wave envelope. However, the Mapping Partner should not subdivide the horizontal runup portion of the seaward wave envelope (see Figure D.2.11-2). The BFE should simply be the runup elevation, rounded to the nearest whole foot.

Ideally, the Mapping Partner would establish an elevation zone for every BFE in the wave envelope; however, because these zones are mapped on the FIRM so that buildings or property can be located in a flood insurance risk zone, the Mapping Partner should use a minimum width for the mapped zone to provide a usable FIRM. For coastal areas, the general guidance is to have a minimum zone width of 0.2 inch on the FIRM. The mapping criteria and the ability to map all coastal BFE and hazard zone changes is dependent upon the scale of the FIRM. The minimum zone width is 0.2 times the final FIRM scale; for example, a width of 80 feet for a FIRM at a scale of 1 inch equals 400 feet, or a width of 100 feet for a FIRM at a scale of 1 inch equals 500 feet. Because digital FIRM data can easily be enlarged, the map scale limitations should be reviewed by the Mapping Partner with the FEMA Study Representative and community officials.

The Mapping Partner should combine elevation zones that do not meet the minimum width requirement with an adjacent zone or zones to yield an elevation zone equal to or wider than the minimum width. The BFE for this combined zone is a weighted average of the combined zones, rounded to the nearest whole foot. When combining VE Zones, the Mapping Partner should not reduce the maximum BFE at the shoreline by averaging.

The AE Zone, if wide enough, should be subdivided in the same manner. If the total AE Zone width is less than the minimum width requirement, the VE Zone with the lowest elevation is usually assigned to that area. This situation typically occurs for steep or rapidly rising ground profiles, and it is not unreasonable to designate the entire inundated area as a VE Zone. In some cases, however, it may be appropriate for the Mapping Partner to extend the AE Zone slightly into the next zone seaward to satisfy the minimum width requirement.

Relatively low areas landward of zones subject to wave effects may be subject to shallow flooding or the ponding of floodwater; the Mapping Partner should designate these areas as AO or AH Zones. Such designations can be relatively common landward of coastal structures, bluffs, ridges, and dunes, where wave overtopping occurs.

Identifying appropriate zones and elevations may require particular care for dunes, given that the entire PFD is defined as a coastal high hazard area. Although the analyses may have determined that a dune will not completely erode and that the wave action should stop at the retreated dune face with only overtopping possibly propagating inland, the Mapping Partner should designate the entire dune as a VE Zone, as defined in the NFIP regulations. The Mapping Partner should assign the last calculated BFE at the open-coast dune face (whether VE or AE Zone) to be the dominant VE Zone BFE for the entire PFD and should extend this value to the landward limit of the PFD. It may seem unusual to use a BFE lower than the ground elevation, but this is fairly common. Most of the BFEs for areas where the dune was assumed to be eroded are also below existing ground elevations. In these cases, it is the VE Zone designation that is most important to the NFIP because, under current regulations, structures in VE Zones must be built on pilings and alterations to the dunes are prohibited.

D.2.11.5 Transect Examples

Settings occurring along the Atlantic and Gulf coastlines include the following:

- Sandy beach backed by a low sand dune or sand berm
- Sandy beach backed by a high sand dune formation
- Sandy beach backed by shore protection structures
- Cobble, gravel, shingle, or mixed-grain-size beach and berms
- Erodible coastal bluffs
- Non-erodible coastal bluffs or cliffs
- Tidal flats and wetlands

The examples discussed below depict idealized transects for these beach settings, where erosion, wave runup, and overtopping are the dominant coastal processes, to illustrate common flood hazard zonations in a quantitative way. The BFEs shown are arbitrary and are included for illustrative purposes only.

> **Example 1.** Figures D.2.11-3a and D.2.11-3b illustrate flood hazard mapping for a transect where the dune or sand berm does not meet the 540-foot criterion and will be removed in the erosion assessment, allowing wave heights to dominate the flood insurance risk zones throughout the 1-percent-annual-chance floodplain. In this scenario, the WHAFIS (or a similar wave-height model) results can be mapped directly, with only zone averaging required for any flood zones that cannot be mapped at the final map scale.

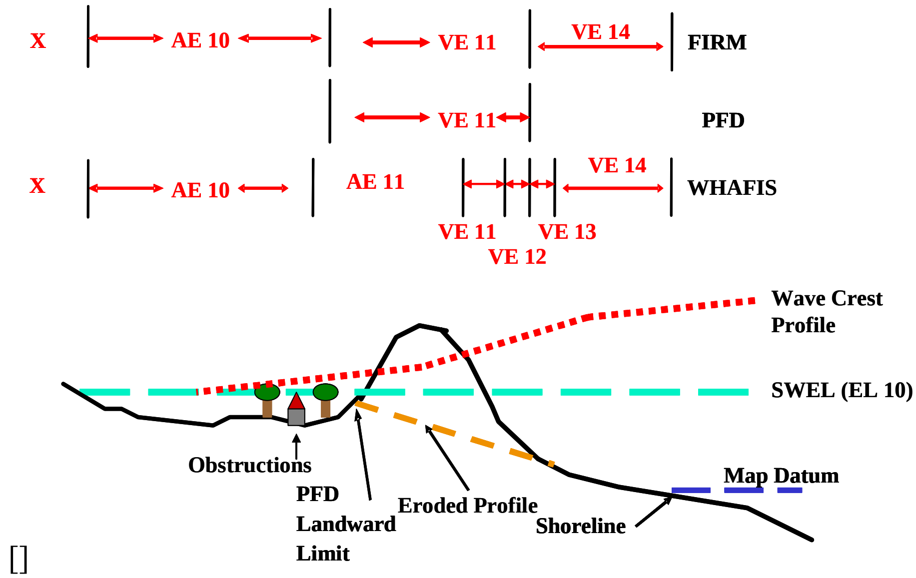

Figure D.2.11-3a. Example 1: Sandy Beach backed by Low Dune, with Wave Height Propagation and PFD Controlling the Flood insurance risk zone Mapping.

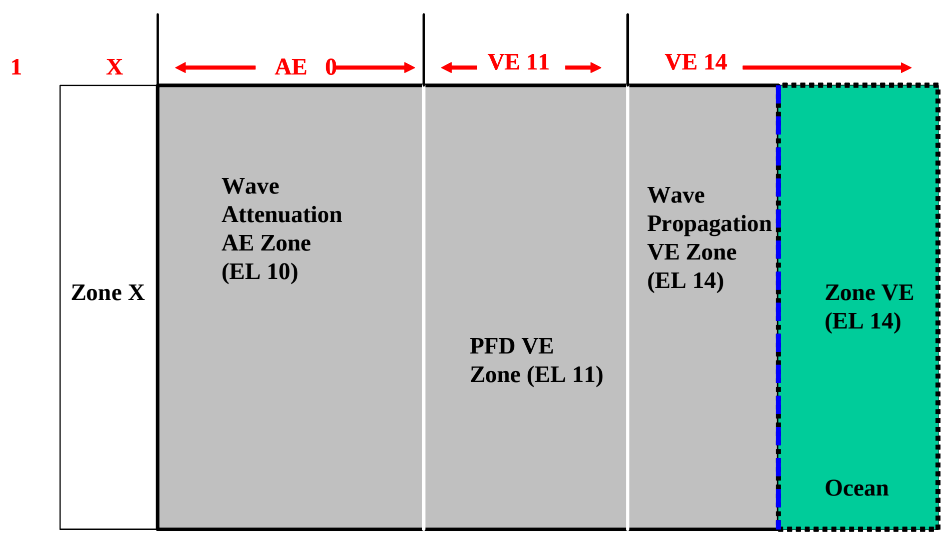

Figure D.2.11-3b. Example 1: Sandy Beach backed by Low Dune, with Wave Height Propagation and PFD Controlling the Flood insurance risk zone Mapping.

Example 2. Figures D.2.11-4a and D.2.11-4b illustrate flood hazard mapping for a low coastal dune where the dune cross section is insufficient to prevent removal by the 1-percent-annual-chance flood. The eroded profile is calculated and adjusted (see Subsection D.2.9), then the resulting profile is checked for inundation, overland wave propagation, wave runup, and overtopping. In the example shown, the remnant dune crest is not inundated, so overland wave propagation is not mapped. Instead, hazard zones are mapped based on the combined effects of wave runup, overtopping splash (runup extends more than 3.0 feet above the crest in this example), and PFD. Guidance for determining AO zone depths based on the overtopping rate is provided in Subsection D.2.8.2.3. In this example, the overtopping splash zone extends farther landward than the PFD and determines the VE/AO boundary.

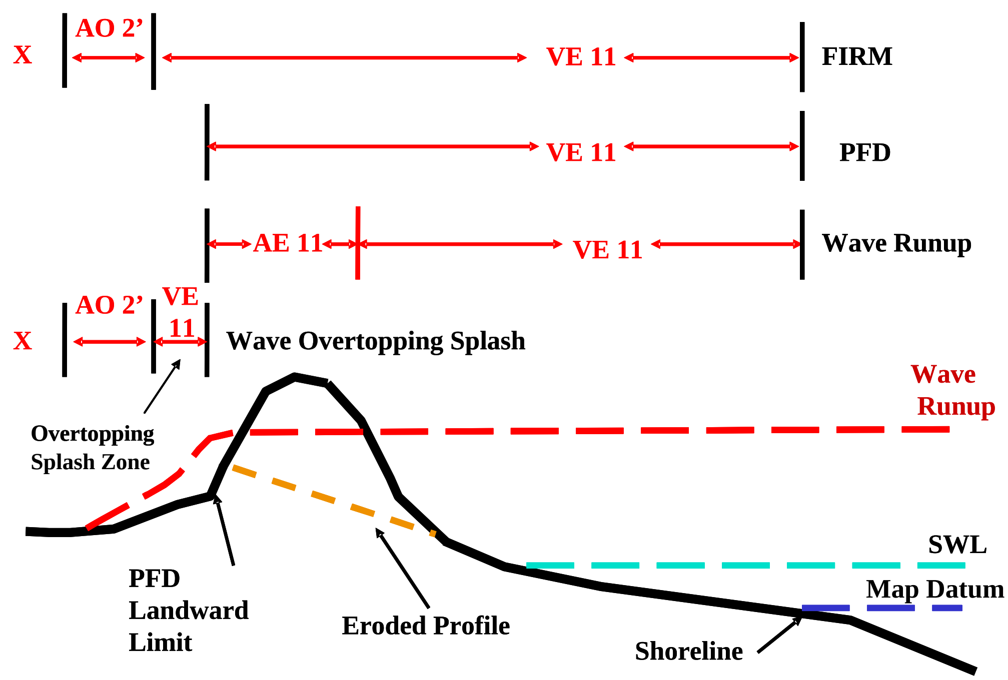

Figure D.2.11-4a. Example 2: Sandy Beach Backed by Low Sand Dune with Overtopping Splash Controlling VE Zone

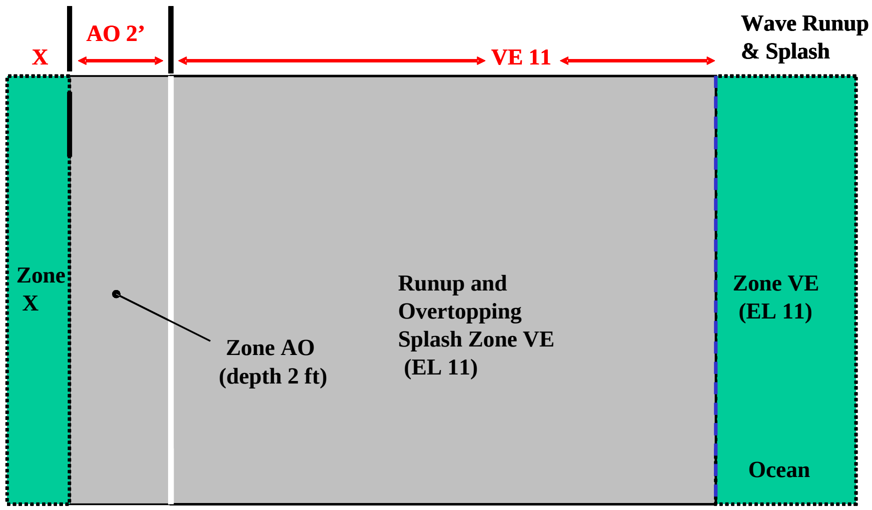

Figure D.2.11-4b. Example 2: Sandy Beach Backed by Low Sand Dune with Overtopping Splash Controlling VE Zone

Example 3. Figures D.2.11-5a and D.2.11-5b illustrate flood hazard mapping for a PFD that is large enough (in cross section) to prevent its removal and high enough to prevent overtopping during 1-percent-annual-chance flood conditions. In the example shown, the eroded profile is first generated according to the dune retreat erosion regime (D.2.9), then wave runup on eroded profile is calculated (D.2.8). The 2-percent wave runup elevation is mapped. In the absence of a PFD designation, the area seaward of the eroded dune face would be mapped as an AE Zone, where the runup depth is less than 3.0 feet, or as a VE Zone where the runup depth is greater than 3.0 feet. The area landward of the eroded dune face would be mapped as X Zone. However, given the PFD designation, the area between the shoreline and the landward heel of the dune will be mapped as a VE Zone; the BFE at the dune face (EL 13) will be continued landward to the PFD landward limit. Note that this is the only mapping scenario where the hazard zone (landward of the dune face) is based on coastal morphology, not on actual flood hazards during the 1-percent-annual-chance flood. Likewise, the BFE landward of the dune face is an extension of the BFE at the dune face, not representative of the actual flood profile.

If the dune in Figure D.2.11-5a were not high enough to prevent overtopping and the potential runup extended more than 3.0 feet above the crest, an overtopping splash VE Zone would be indicated on the landward side of the eroded crest. In all cases, the BFEs landward of the eroded dune crest would be mapped at the higher BFE (PFD or splash zone) at any given point along the transect.

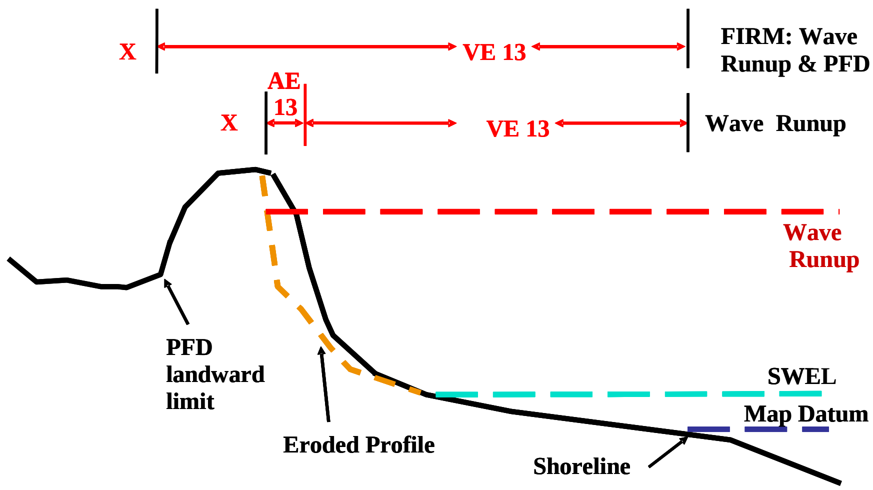

Figure D.2.11-5a. Example 3: Sandy Beach Backed by High Sand Dune with PFD Controlling the VE Zone

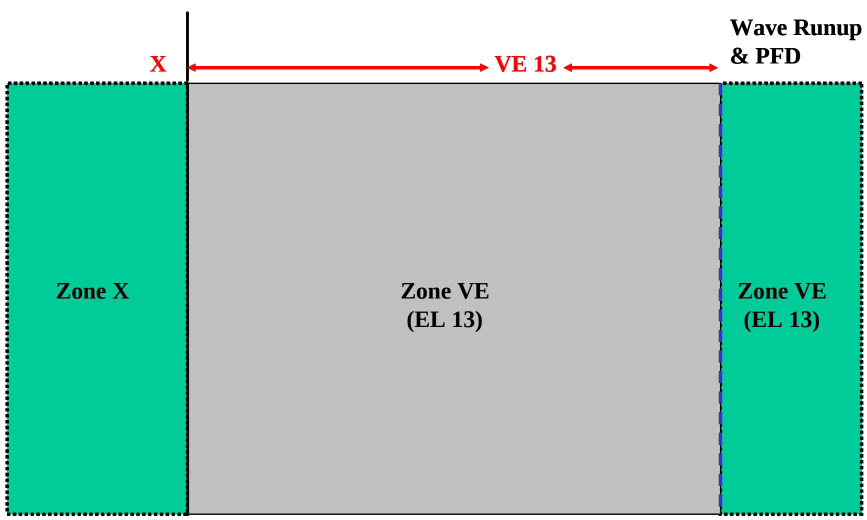

Figure D.2.11-5b. Example 3: Sandy Beach Backed by High Sand Dune with PFD Controlling the VE Zone

Example 4. Figures D.2.11-6a and D.2.11-6b illustrate flood hazard mapping for an overtopped coastal structure that remains intact during the 1-percent-annual-chance flood (see Subsection D.2.10.3 for a discussion of structure failure and local scour considerations). In this example, the potential runup reaches an elevation greater than 3.0 feet above the crest of the structure; therefore, an overtopping splash VE Zone is mapped landward of the structure crest. If the potential runup is less than 3.0 feet above the crest, no VE overtopping splash zone should be mapped; an AO sheet flow zone should be mapped instead. Guidance for determining AO zone depths based on the overtopping rate is provided in Subsection D.2.8.2.3. The same basic procedure is used for vertical and sloping structures, with the principal difference being the equations used to calculate wave runup and splash distances. Thus, if this particular structure was assumed to sustain total or partial failure during the 1-percent-annual-chance flood, a similar procedure would be applied, but with sloping structure equations rather than vertical structure equations.

For shore structures with steep slopes, runup elevations are relatively high and a wide range of wave hazards can occur, including erosion or scour near the structure. These circumstances may result in a variety of distinct and compact situations where appreciable engineering judgment can be required for the appropriate assessment of flood hazards.

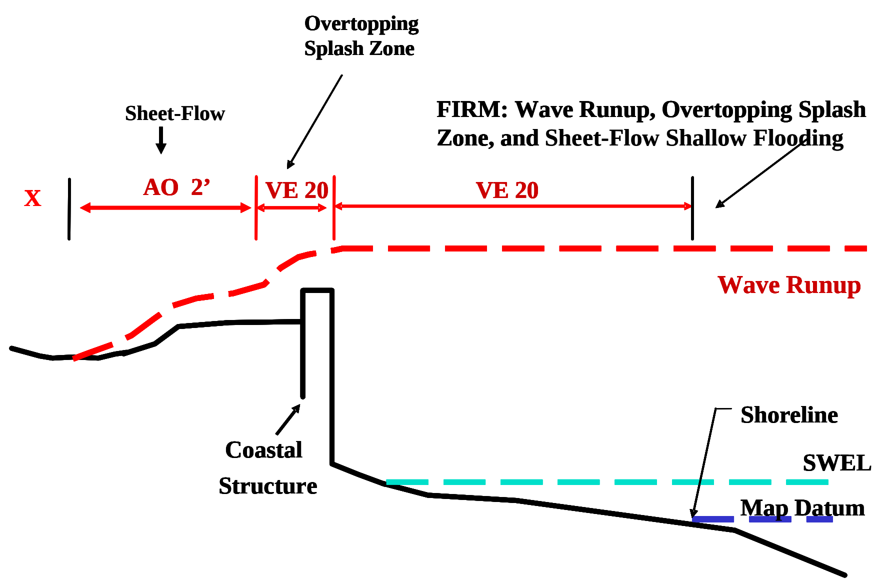

Figure D.2.11-6a. Example 4: Sandy Beach Backed by Shore Protection Structure with VE Zone Controlled by the Splash Zone from Wave Overtopping

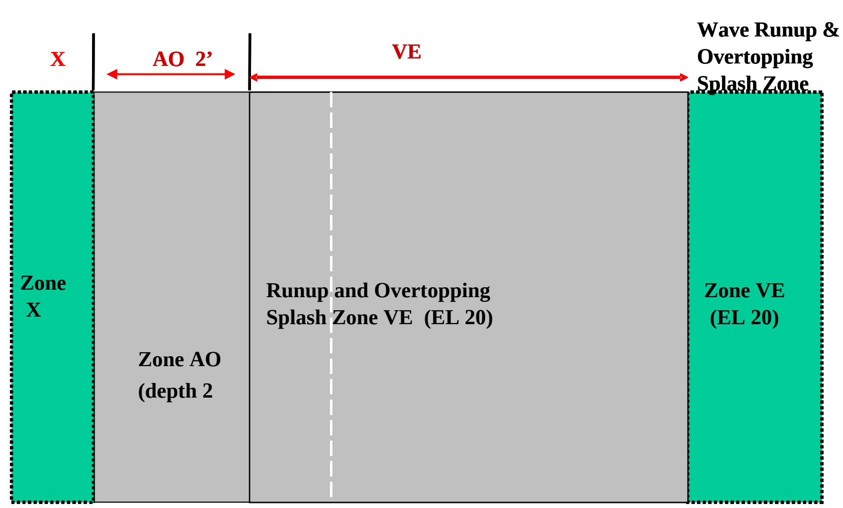

Figure D.2.11-6b. Example 4: Sandy Beach Backed by Shore Protection Structure with VE Zone Controlled by the Splash Zone from Wave Overtopping

Example 5. Figures D.2.11-7a and D.2.11-7b illustrate flood hazard mapping for a beach composed of gravel, cobble, or mixed-grain sizes. In this example, the profile configuration should be determined in accordance with Subsection D.2.9.3.2, and the wave hazards should be modeled using the eroded profile. There will be no PFD designation for a gravel, cobble, or mixed-grain-size profile, so the mapped hazard zones and BFEs will reflect calculated flood hazards only.

In this example, the potential runup is assumed to reach more than 3.0 feet above the crest, so an overtopping splash zone is mapped landward of the profile crest, with an AE Zone to the rear. The AE Zone is mapped instead of the AO Zones shown in Examples 3 and 4, because the overtopping ponds are in the area behind the crest in this case. The mean overtopping rate calculations (see Subsection D.2.8.2.2) should be used to determine the volume of water overtopping the barrier during the 1-percent-annual-chance flood conditions, and the BFE in the AE Zone should be determined by the overtopping volume and the local topography.

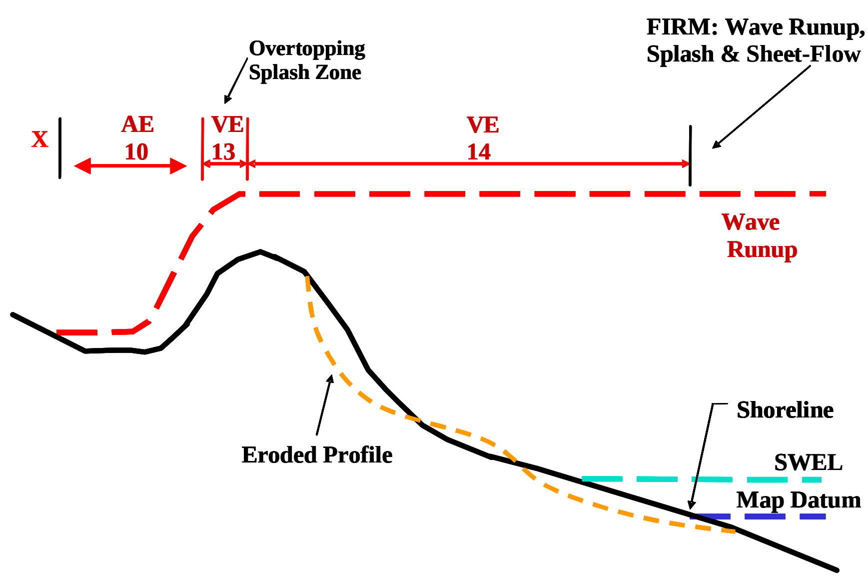

Figure D.2.11-7a. Example 5: Cobble, Gravel, Shingle, or Mixed-Grain-Sized Beach with VE Zone Controlled by Wave Runup and Overtopping Splash

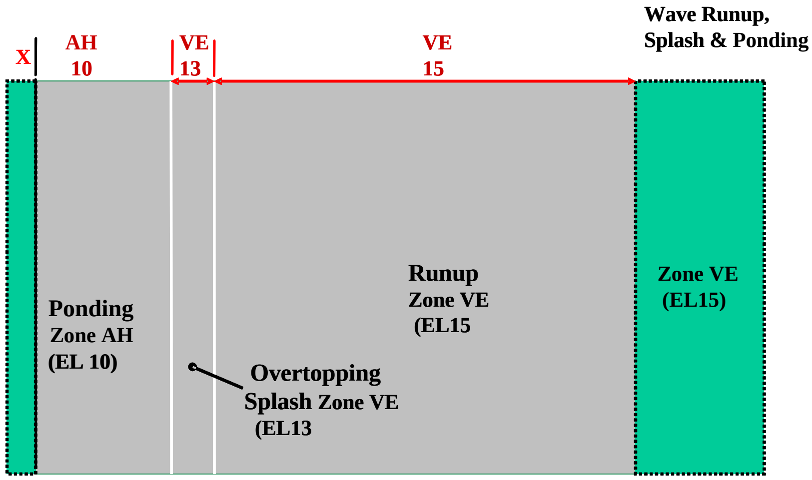

Figure D.2.11-7b. Example 5: Cobble, Gravel, Shingle or Mixed-Grain-Sized Beach with VE Zone Controlled by Wave Runup and Overtopping Splash

Example 6. Figures D.2.11-8a and D.2.11-8b illustrate flood hazard mapping for an erodible coastal bluff that is not high enough to prevent overtopping and where the potential runup reaches higher than 3.0 feet above the crest. In this example, the eroded profile is calculated first using procedures described in Subsection D.2.9, then wave runup and overtopping are mapped against the eroded profile. The area seaward of the bluff will be mapped as the VE Zone, with a BFE set at the potential runup elevation. The area immediately landward of the eroded bluff face will be mapped as a VE Zone based on the presence of an overtopping splash zone. BFEs in the VE splash zone will be based on the calculated water-surface profile decay (see Subsection D.2.5.3.3).

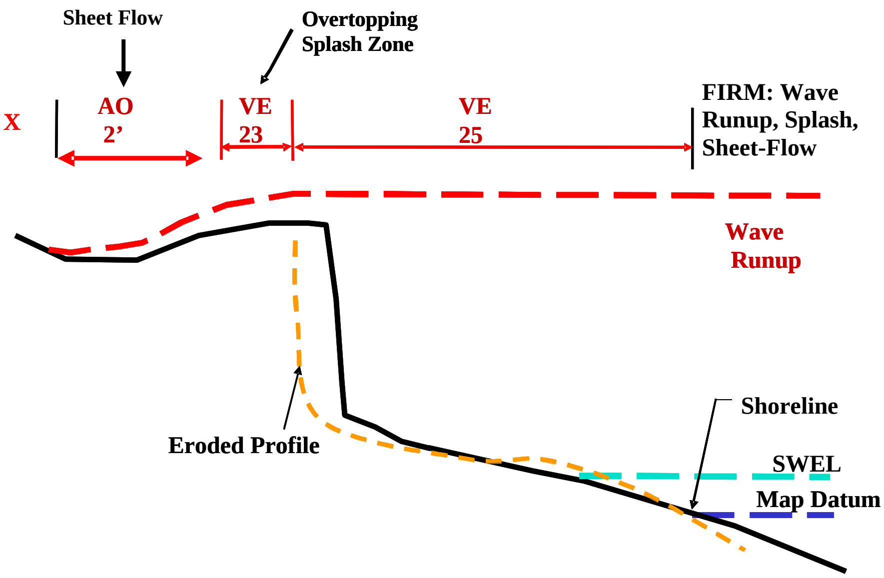

Figure D.2.11-8a. Example 6: Erodible Low Coastal Bluff with VE Zone Controlled by Wave Runup and Overtopping Splash

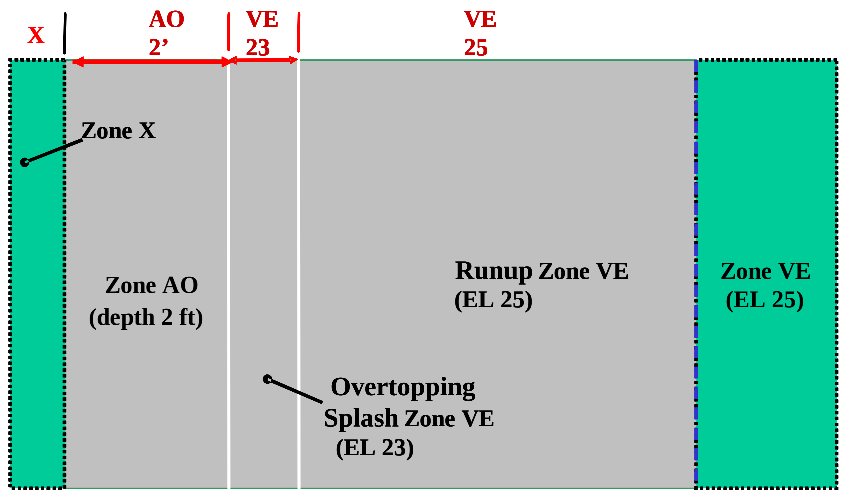

Figure D.2.11-8b. Example 6: Erodible Low Coastal Bluff with VE Zone Controlled by Wave Runup and Overtopping Splash

Example 7. Figures D.2.11-9a and D.2.11-9b illustrate flood hazard mapping for a non-erodible coastal bluff high enough to prevent overtopping during 1-percent-annual-flood conditions. The area seaward of the bluff will be mapped as the VE Zone, with a BFE set at the potential runup elevation. The area landward of the bluff face will be mapped as X Zone (unshaded).

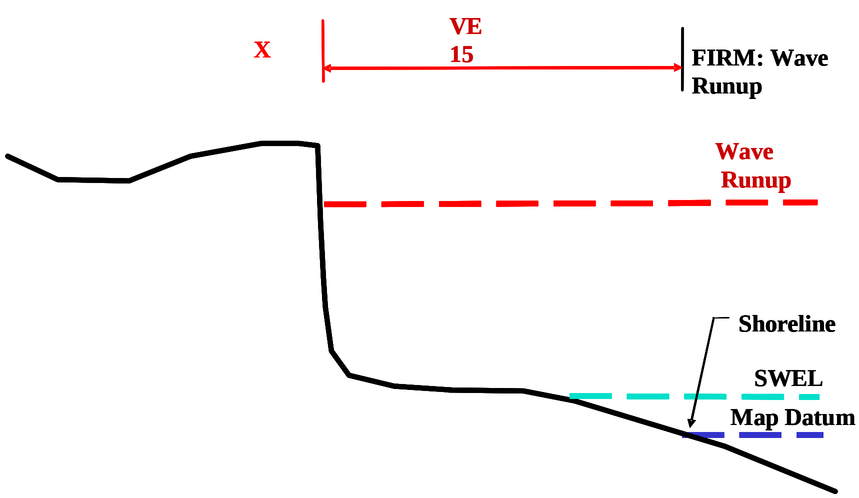

Figure D.2.11-9a. Example 7: Non-erodible High Coastal Bluff with VE Zone Controlled by Wave Runup (No Overtopping)

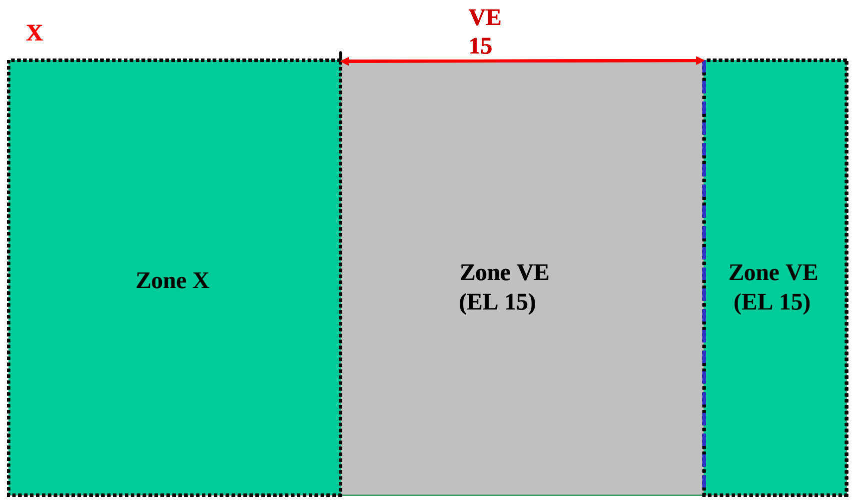

Figure D.2.11-9b. Example 7: Plan View of Flood insurance risk zones and BFEs, Non-erodible High Coastal Bluff with VE Zone Controlled by Wave Runup (No Overtopping)

Example 8 (no figure). For cases in which a profile is inundated by the static water level during the 1-percent-annual-chance flood – such as a tidal wetland, low sand beach, or other flooded low-lying area – wave runup and overtopping do not need to be calculated and mapped. Instead, the hazard zones and BFEs should be mapped with the results of the WHAFIS model (see Subsection D.2.4.3.3) or other similar analysis. The VE Zone should be mapped where the vertical difference between the wave crest elevation and the static water level is equal to or greater than 2.1 feet; the AE Zone should be mapped where the difference is less than 2.1 feet. BFEs should be mapped at even-foot increments, in a stairstep fashion, following the wave crest profile.

D.2.11.6 Mapping Procedures

This subsection presents guidance for mapping newly studied coastal zones and remapping or redelineating coastal flood insurance risk zones. In redelineation, effective SWELs and BFEs are remapped using new or more detailed topographic data and base maps, or to implement a vertical datum conversion. Included below are the requirements for reviewing the initial model results and identifying flood insurance risk zones, guidance and examples for determining transects, and guidance for depicting the analysis on the FIRM.

D.2.11.6.1 Newly Studied Coastal Zones

A properly integrated delineation of the results of flooding analyses involves judgment and skill in reading topographic and land-cover maps. The time and effort put forth to determine the flood elevations and flood zone extents will be negated if the results of these analyses are not properly delineated on the FIRM. Provided below is a description of the general process by which the coastal analyses are to be transformed from a series of flood zones and BFEs calculated along numerous transects to a mapped product consistent with these mapping guidelines and specifications.

The preliminary FIRM is usually produced from engineering work maps based on the coastal analyses. Therefore, the Mapping Partner should transfer the flood zones and elevations identified on each transect's wave profile to the work maps and interpolate boundaries between transects. To do so, the Mapping Partner should set up the work maps with contour lines, buildings, structures, vegetation, and transect lines clearly located. Because roads are often the only fixed physical features shown on the FIRM, the Mapping Partner should ensure that other features and the flood zone boundaries are properly located on the work maps in relation to the centerline of the roads as they will appear on the FIRM. The starting point (0 Station) for each transect should be clearly annotated on the work maps.

The Mapping Partner should transfer the identified elevation zones from the wave profile to the work maps, marking the location of the flood zone boundaries along the transect line so that boundary lines can be interpolated between transects. The Mapping Partner should ensure that boundaries are marked at the correct location. Because of erosion assumptions, the location of the 0.0-foot elevation at the shoreline can change on the transect, but the 0 Station, the point from which the flood zone changes from the wave profile are referenced, must remain fixed on the work map. As discussed in Subsection D.2.11.4, some flood zones on the wave envelope may be too narrow to map at the current map scale. Thus, some zones must be eliminated, and elevations must be averaged. The Mapping Partner should measure the widths of the resulting flood zones carefully; zones that narrow to less than 0.2 inch at map scale must be tapered to an end. Likewise, if the averaged flood zone becomes much wider, it may be possible to break the averaged zone back into two (or more) separate elevation zones.

With final elevations from the wave profile plotted on the work maps and any zone averaging completed, the Mapping Partner should determine the location of each flood zone change in relation to a physical feature (e.g., ground contour, back side of a row of houses, 50 feet into a vegetated area) and delineate the boundary for the area represented by that transect along this feature. For example, if the BFE for a VE Zone decreases from 14 feet to 13 feet coincident with change from a residential area to a forest, the Mapping Partner should examine the land use data and follow the boundary of the forest to the left and right of the transect line to extend the delineation of the flood zone change.

One of the more difficult steps in delineating coastal flood zones and elevations is the transition between transects. Good judgment and an understanding of typical flooding patterns are the best tools for this job. Initially, the Mapping Partner should locate the area of transition (an area not exactly represented by either transect) on the work maps. The Mapping Partner should then delineate the floodplain boundaries for each transect up to this transition area. The Mapping Partner should examine how a transition can be made across this area to connect matching zones and still have the boundaries follow logical physical features. Other transects similar to this area could give an indication of flooding. Sometimes the elevation zones for the two contiguous transects are not the same; in such cases, the Mapping Partner may have to taper the zones to an end or enlarge the zones and subdivide them in the transition area.

With the advent of computer applications that can quickly pre- and post-process terrain, land-use, and other data to support wave analyses, coastal transects can now be generated at narrow alongshore spacings that approximate 2-D modeling. While the selection of the transect spacing is left to the judgment of the Mapping Partner, there is a point of diminishing returns beyond which the addition of more transects will not appreciably improve the final product. Furthermore, increasing the transect density may not fully resolve flood zone transition problems that occur coincident with physical features that end abruptly (e.g., boundaries between densely developed parcels and open space/parks; at the ends of shore protection structures). The Mapping Partner must determine the transect spacing that will be adequate to accurately model the base flood conditions and interpolate the results. The Mapping Partner should also recognize that it may not be possible to show all transects on the work maps or FIRM, or include all results in the FIS text tables or other derivative products associated with the mapping project. Care must be taken to ensure that the final work map or FIRM is consistent with the modeling completed by the Mapping Partner, and that transects shown on the final maps are, in fact, representative of these results.

In some cases, fewer transects may be adequate to characterize flood hazards in geographically separate but physically similar shoreline reaches. Areas with significant flooding hazards from wave runup may have one transect representing multiple alongshore reaches because the areas have similar shore slopes. In this case, the Mapping Partner should identify the different areas and delineate the results of the typical transect in each area. Transition zones may be necessary between areas with high runup elevations to avoid large differences in BFEs, and to smooth the change in flood zone boundaries. These zones should be fairly short and cover the shore segment with a slope not exactly typical of either area. The Mapping Partner should determine the transition elevation using judgment in examining runup transects with similar slopes. The Mapping Partner should not use transition zones if there is a very abrupt change in topography, such as at the end of a coastal structure.

Lastly, after plotting flood zones and BFEs and interpolating results between transects, the Mapping Partner should map the X Zone areas. The Mapping Partner should show areas below the 0.2-percent-annual-chance SWEL that are not covered by any other flood zone as X Zone (shaded) on the FIRM. Often, the maximum runup elevation associated with the base flood is higher than the 0.2-percent-annual-chance SWEL. In such cases, the X Zone (shaded) designation will not be used in that area. All other areas are designated X Zone without shading.

Because flood elevations are rounded to the nearest whole foot, the Mapping Partner does not need to spend time resolving a minor elevation difference. Also, because coastal structures must be located on the FIRM, the Mapping Partner should attempt, whenever possible, to smooth the boundary lines and to follow a fixed feature such as a road. In preparing the FIRM, the Mapping Partner should ensure that the mapped results are technically correct and that the FIRM is easy for the community official, engineer, surveyor, and insurance agent to use.

D.2.11.6.2 Redelineation of Coastal Zones

During the project scoping phase, coastal reaches may be identified where new surge modeling and detailed wave analyses are not required. In these cases, the Mapping Partner will be responsible for remapping or redelineating the effective coastal flood hazard data onto the new FIRM. When determining how a coastal area should be redelineated, the Mapping Partner should consider the availability of new or more detailed topographic data, the base map being used for the revised FIRM (including any new shoreline position), and whether a vertical datum conversion is necessary.

Although these guidelines provide information on the most common redelineation aspects and a general approach for identifying issues, each effective coastal flood hazard dataset can pose unique problems that could, in some instances, require new modeling to resolve. For this reason, it is critical that the Mapping Partner fully investigate redelineation issues and identify the most appropriate methodology early in the scoping process (see Subsection D.2.1.2), coordinating closely with the FEMA Study Representative to resolve any issues that are discovered.

Several typical redelineation scenarios, and the methods available to map the effective flood data, are presented below. Of the known redelineation concerns, shoreline retreat and datum conversions have the most significant impacts on remapping flood zone boundaries. For organizational purposes, the guidance and illustrative examples have been subdivided based on the degree of shoreline retreat at the study site. The discussion is further subsequently subdivided to present the effects of new topographic data and/or datum conversions on the redelineation process. The Mapping Partner should review *all* scenarios for relevant guidance. As redelineation is a relatively new activity for Mapping Partners, these scenarios should not be considered all-inclusive; the guidelines will be revised and supplemented in the future, as warranted.

Scenario 1: Minimal to No Shoreline Retreat

In this setting, the new base map being used for the FIRM shows that the shoreline (typically the High Water Line for vector-based maps, or the wet-dry line at the time of the collection for aerial photographic base maps) has undergone minimal net landward retreat in the time elapsed because the effective FIRM was published. That is, the new shoreline still lies within the same outermost VE Zone shown on the effective FIRM (see Figure 2.11-10). (Seaward progradation of the shoreline would also fit this scenario.)

- **If no new topographic data are being utilized and no datum conversion from NGVD29 to NAVD88 is required**, the redelineation will consist of duplicating the effective flood zone boundary locations, including the VE/AE boundary associated with the PFD (where applicable) and the 1-percent and 0.2-percent-annual-chance floodplain boundaries, exactly as they are shown on the effective FIRM.

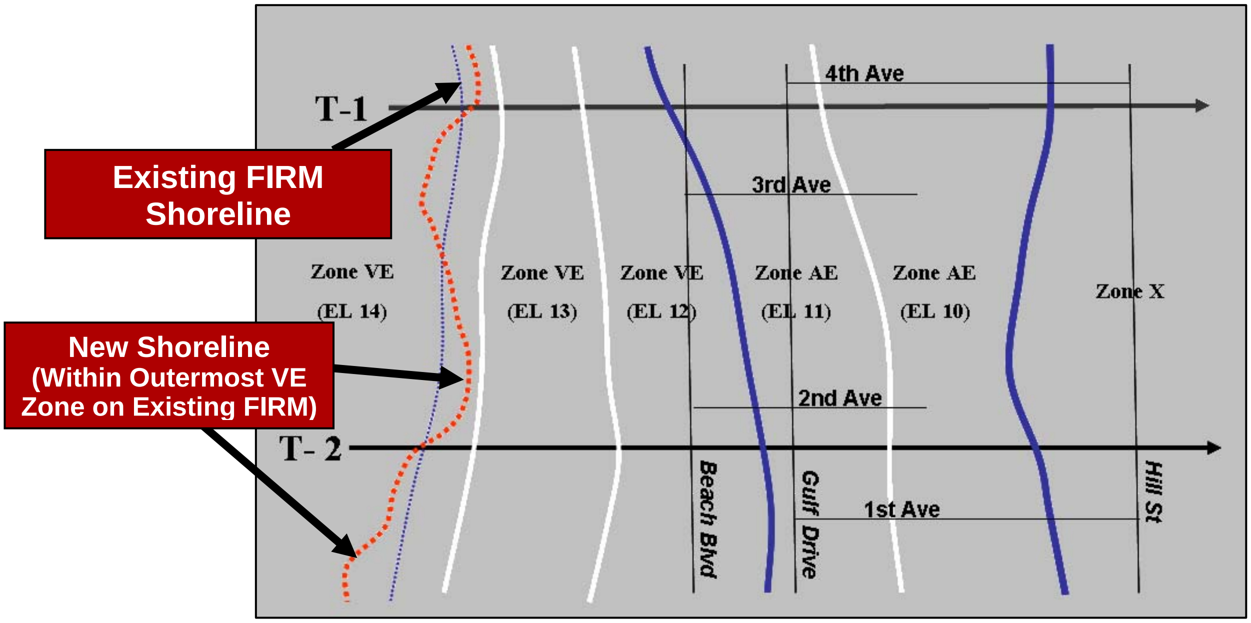

Figure D.2.11-10. Work map depicting the flood zones, BFEs, and shoreline from the effective FIRM and the new shoreline position (modified from DiCamillo et al., 2005). T-1 and T-2 represent transect locations. Because the shoreline retreat is restricted to the outermost VE Zone (EL 14), it has no impact on remapping of flood zones.

- **If new topographic data are being used** as the basis of the FIRM update, multiple flood zone boundaries can be redefined based on the new data, specifically the 1-percent and 0.2-percent-annual-chance floodplain limits and any PFD-based VE/AE boundary. Prior to redelineating the limit of the 1-percent and 0.2-percent-annual-chance floodplains, the Mapping Partner shall use the guidance below to review the effective FIS and FIRM and to determine the controlling factor for the limit of flooding in an area and determine the appropriate elevation(s) for redelineation:

  Identify the final flood insurance risk zone and BFE before the limit of the 1-percent-annual-chance floodplain. Because coastal flood insurance risk zones and BFEs are frequently averaged when the zones are too narrow to be mapped, and coastal BFEs may include a wave height component, the Mapping Partner should not assume that the final whole-foot BFE immediately seaward of the limit of the 1-percent-annual-chance floodplain is the appropriate elevation to use to redelineate the floodplain boundary. Where applicable, the Mapping Partner shall evaluate the effective modeling for areas where Zone AO is the final flood insurance risk zone to determine the appropriate elevation for redelineation of the 1-percent-annual-chance floodplain boundary. Also, in areas where Zone X is mapped immediately adjacent to the open coast, the Mapping Partner should consult the new topographic data and delineate the PFD landward heel.

  The Mapping Partner shall locate the effective transect nearest to the area being redelineated and determine the 1-percent and 0.2-percent-annual-chance SWELs from the “Transect Data Table” or “Transect Description Table” in the FIS. If the area being redelineated is along a tidally influenced stream, river, or other sheltered waters where there are no transects, the Mapping Partner shall obtain the 1-percent and 0.2-percent-annual-chance SWELs from the "Summary of Stillwater Elevations" table and/or Flood Profiles in the FIS. The Mapping Partner shall determine whether wave setup is included in the 1-percent-annual-chance SWELs reported in the FIS and ensure that the elevation used for redelineation of the 1-percent-annual-chance floodplain does not include wave setup.

When wave runup is the controlling factor for the limit of the 1-percent-annual-chance floodplain, the elevation being used to map the limit will be higher than the SWEL presented in the FIS. The Mapping Partner shall consult the FIS, FIRM, aerial photography, and/or topographic data to determine areas where wave runup is the dominant hazard. In these areas, the 0.2-percent-annual-chance runup elevation should be used to redelineate the limit of the 1-percent-annual-chance floodplain.

When redelineating the 1-percent and 0.2-percent-annual-chance floodplains between transects, there will be areas where the Mapping Partner must transition from one elevation to another, such as when there are flooding sources with varying SWELs or areas with varying runup elevations. For this reason, the Mapping Partner shall determine the appropriate elevation for mapping of the 1-percent and 0.2-percent-annual-chance floodplains at each transect prior to redelineation. In areas of transition between transects, the general shape of the effective boundaries should be maintained, but offset to follow the new topographic data (see Figure 2.11-11).

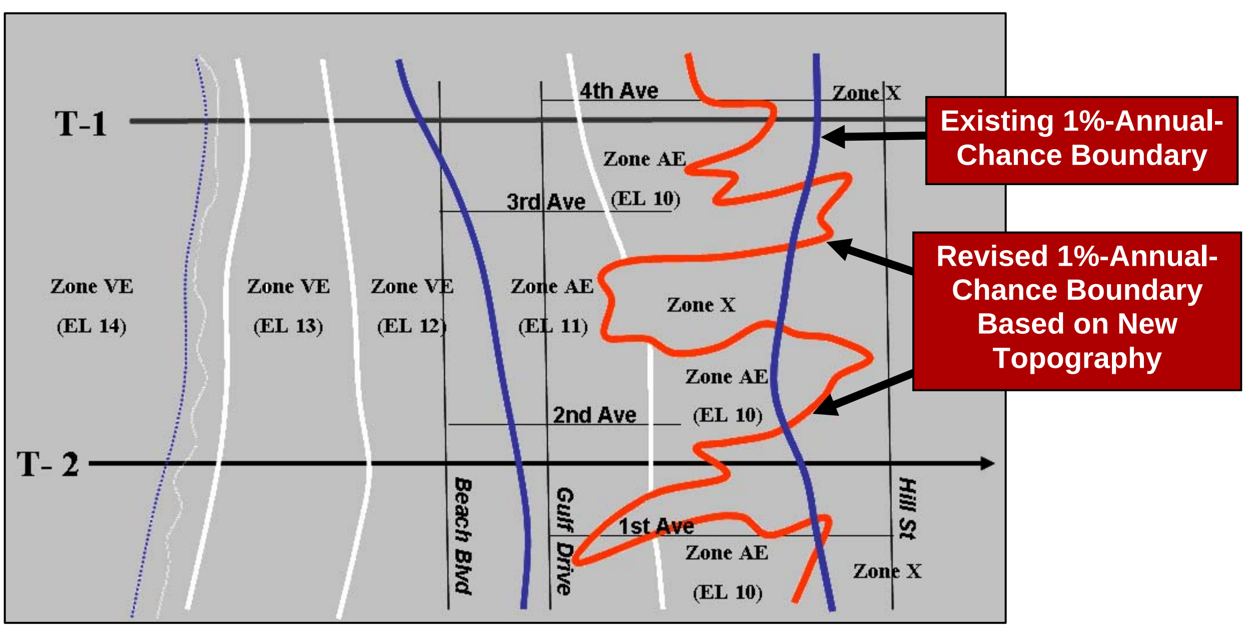

Figure D.2.11-11. Work map depicting the existing 1-percent-annual-chance floodplain boundary from the effective FIRM, and the new boundary redelineated based on the effective SWEL and new topographic data (modified from DiCamillo et al., 2005).

As shown in Figure 2.11-11, the redelineated limit of the 1-percent-annual-chance floodplain may impinge upon or cross flood zone boundaries located farther seaward. Similarly, a redelineated PFD limit may intersect flood zone boundaries located landward of the effective FIRM's PFD limit. The Mapping Partner shall not revise the location of gutter lines affected by the new 1-percent-annual-chance and PFD limits without first performing updated modeling; instead, these gutter lines should be clipped at the revised limit of flooding or PFD, as shown in Figure 2.11-11.

- **If no datum conversion is being performed**, the Mapping Partner shall ensure that all gutter lines separating flood insurance risk zones of differing BFEs (except for the PFD-based VE/AE boundary, if redelineated) will remain in the same location and orientation as on the effective FIRM. This is true even when new topographic data are utilized in the study. While topography is a key factor in establishing the wave profile from which the coastal gutter locations are derived, it is not the only factor.

- **If the study includes a datum conversion**, the complexity and level of effort required by the Mapping Partner to complete the redelineation may increase significantly. That is because datum conversions may require coastal gutters separating BFEs to be moved. Recall that each BFE is a whole-foot elevation that actually represents flood elevations from 0.5 feet below to 0.4 feet above the BFE. With the exception of the PFD-based VE/AE boundary, the coastal gutters are located at the half-foot elevations along the wave profile (see map and upper panel [A] on Figure 2.11.-12). When the *vertical* datum conversion is applied, the *horizontal* location (or station) of each half-foot elevation shifts either landward or seaward on each transect's wave profile (see lower panel [B] on Figure 2.11-12).

  Typically a datum conversion of more than 0.1 foot can have a significant impact to gutter locations, depending on the topography. If the land is relatively steep, the impact could be minimal. If the land has a gentle slope, the impact can be much greater because the distance between half-foot elevations along the wave elevation profile can be large. If a datum conversion is around 1.0 foot, then the gutters can remain in the same location with just a change in the BFEs by 1 foot. The Mapping Partner shall determine the conversion factor, review the topography, and propose a method for redelineating coastal flood hazards in the different datum to the FEMA Study Representative. Once the Mapping Partner has determined the location of the gutters along each transect, the flood insurance risk zones and BFEs shall then be mapped as discussed in previous subsections.

  Redelineation of coastal gutter locations can be accomplished efficiently if the effective wave transect modeling results are available. In cases where the modeling results are not available, the Mapping Partner shall propose an approach for the datum conversion and present it to the FEMA Study Representative for approval. One option may be to construct a simplified wave profile based on the effective gutter locations, interpolating the wave height between the half-foot elevations (e.g., Figure 2.11-12). Application of this approach must be limited to transects where wave heights were the dominant hazard in the effective study and no PFD was mapped.

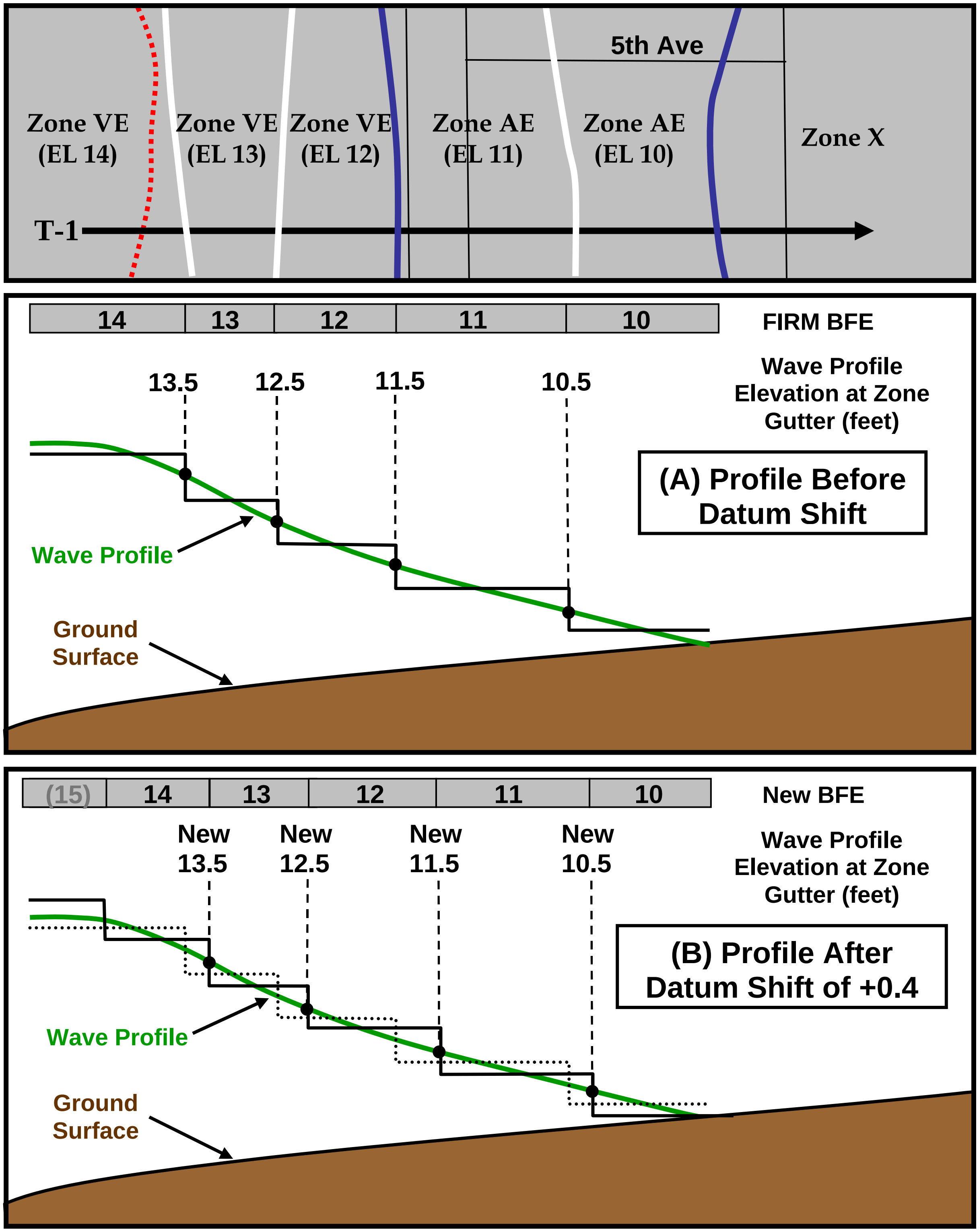

Figure D.2.11-12. Comparison of gutter locations prior to a datum conversion (A) and after (B). Although Zone VE (EL 15) can be identified on the new wave profile, it lies seaward of the mapped shoreline position and thus may not need to be included on the FIRM.

Scenario 2: Moderate Shoreline Retreat

In this setting, the new base map being used for the FIRM shows that the shoreline has retreated far enough landward that one or more effective VE Zones are now located in open water. If a Zone VE gutter falls seaward of the open-coast shoreline on the new base map, the Mapping Partner shall adjust the gutter to be coincident with or just landward of the shoreline. If multiple Zone VE gutters fall seaward of the open-coast shoreline on the new base map, the intermediate zones can be completely removed. The VE Zone with the highest BFE shall be adjusted so that the gutter is coincident with or just landward of the shoreline. The Mapping Partner shall use caution to not increase the flood insurance risk zone designation or BFE for any properties without modeling to justify such an increase. Incorporation of new or improved topographic data and/or a datum conversion by the Mapping Partner shall follow the guidelines provided earlier in this subsection.

Scenario 3: Significant Shoreline Retreat

This setting would apply in areas where the new base map indicates that the shoreline has retreated landward past the effective FIRMs VE/AE boundary (Figure 2.11-13). Such a scenario is possible (1) on coasts subject to chronic, long-term erosion; (2) where a severe storm (or series of storms) has eroded the shoreline and beach recovery has not yet occurred; (3) adjacent to dynamic tidal inlets; or (4) downdrift of shore protection structures that impede longshore transport of sediment.

While it is not advisable to redelineate coastal flood hazards in areas where significant changes to the open-coast shoreline have occurred since the effective coastal modeling was completed, the Mapping Partner shall utilize the following guidance to ensure that the effective flood hazards are transferred to the new base map in a logical, consistent manner:

> If the gutter separating the VE Zone and AE Zone flood hazard areas along the open coast falls seaward of the shoreline on the new base map, the Mapping Partner shall adjust the VE/AE gutter to be just landward of the shoreline and adjust the seaward VE Zone gutter with the highest BFE to be coincident with the shoreline and remove any intermediate gutters, taking care not to increase the flood insurance risk zone designation or BFE for any properties without modeling to justify such an increase. If this situation occurs with any frequency, the Mapping Partner should consider utilizing the effective shoreline rather than the shoreline from the new base map for the revised FIRM and discuss this with the FEMA Study Representative.
>
> In areas other than the open coast where shoreline changes result in gutters located in open water, the Mapping Partner shall use best judgment in evaluating the nature of the BFE change (wave regeneration over open fetches, wave damping due to vegetation, buildings, etc.) and shift the gutters as necessary to provide a logical identification of flood hazards on the new base map. Again, the Mapping Partner shall use caution to not increase the flood insurance risk zone designation or BFE for any properties without modeling to justify such an increase.

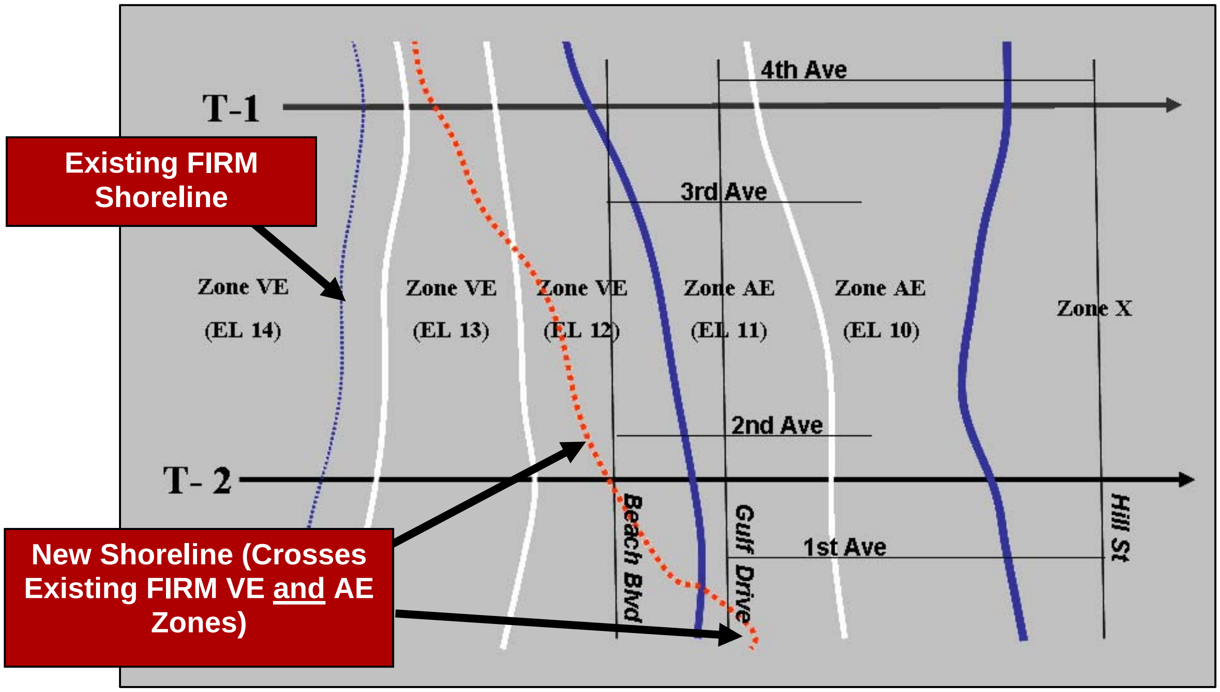

Figure D.2.11-13. Work map depicting existing shoreline position from the effective FIRM and the new shoreline location (modified from DiCamillo et al., 2005). Because the shoreline retreat extends landward of the effective VE/AE boundary, reanalysis of flood hazards may be warranted (in lieu of redelineation).

D.2.12 Study Documentation

This subsection summarizes the reporting requirements for coastal Flood Insurance Studies (FISs) on the Atlantic and Gulf of Mexico (herein referred to as Gulf) coasts, with emphasis on the intermediate data submissions that document the basis and results of coastal flooding analyses during the course of the FIS.

The Mapping Partner shall fully document the coastal flood hazard determination for each affected community. FIS reports and FIRMs form the basis of Federal, State, and local regulatory and statutory enforcement mechanisms and are subject to administrative appeal and litigation. Mapping Partners must ensure that all technical processes and decisions are recorded and documented. This report will provide detailed data needed by FEMA or the community to reconstruct or defend the study results on technical grounds. The minimum information required for the engineering report is summarized below.

Reporting requirements for coastal studies shall follow guidance provided in Appendix M: Guidance for Preparing and Maintaining Technical and Administrative Support Data for the preparation of a Technical Support Data Notebook (TSDN). The TSDN shall consist of the following four major sections, which are described in more detail in Appendix M:

- General Documentation;
- Engineering Analyses;
- Mapping Information; and
- Miscellaneous Reference Materials.

The material compiled for these sections of a coastal study TSDN will be similar to a riverine study, with the exception of the Engineering Analyses section. The Engineering Analyses section of a TSDN for a coastal study shall be formatted to reflect the intermediate data submissions required for such studies.

D.2.12.1 General Documentation

This portion of the TSDN incorporates background information compiled by the Mapping Partner related to changes in scope; special problem reports; minutes of meetings held with the FEMA, communities, and other Mapping Partners; and all correspondence for the study effort (email and hard copy). A complete list of TSDN reporting requirements for General Documentation is provided in Appendix M.

D.2.12.2 Engineering Analyses

Intermediate data submissions provide defined milestones in the coastal flood study process for review of the study approach and results. The Mapping Partner shall submit the data to FEMA in the sequence below.

- Intermediate Submission No. 1 – Scoping and Data Review
- Intermediate Submission No. 2 – Storm-surge Model Calibration and Storm Selection
- Intermediate Submission No. 3 – Storm-surge Runs and Flood-frequency Analysis
- Intermediate Submission No. 4 – Nearshore Hydraulics
- Intermediate Submission No. 5 – Draft Flood Hazard Mapping

The Mapping Partner shall receive review comments within 30 days of receipt of each data submission. The Mapping Partner performing the study shall establish a work plan, so the interim review does not cause any delay in the submission of the draft FIS report and FIRM.

Notes:

- Several different computer codes may be used in the wind, hydrodynamic, and statistical analysis, and several basic computer programs have been listed in numerous FEMA reports. In each section of the engineering report, the Mapping Partner performing the coastal analysis shall list and describe any modifications to these programs and special data inputs used in the study.
- In each section of the engineering report, the Mapping Partner shall provide a complete list of technical references, including computer program references indicating how to obtain copies of the exact program and the input data sources used in the analysis.

D.2.12.2.1 Intermediate Submission No. 1 – Scoping and Data Review

In this report phase, the Mapping Partner shall provide the background information on the study setting and available data relevant to the study area. Any new data needed for the detailed coastal analyses in subsequent phases shall be identified in this phase. Unless otherwise agreed upon with the FEMA Study Representative, the study shall not proceed until all of the information is available and incorporated into the scoping document, which is then submitted for approval by FEMA.

Topographic and Bathymetric Data: If available at this stage, this submission shall include survey control data, topographic data from aerial photography, LIDAR, and field and bathymetric surveys. If survey work is still in progress, the submission shall include available data at the time of submission and a detailed description of the planned survey data collection. Information shall be submitted on the extent of topographic and bathymetric mapping, key mapping parameters (e.g., contour intervals and accuracy standards), horizontal and vertical datum, location and extent of transects, and other pertinent information describing the extent and quality of survey information to be used in the study. If existing community mapping data will be used to supplement survey efforts for the study, the Mapping Partner shall submit information on the date, accuracy standards, datum, extent, and limitations of the mapping.

Tide, Wind, Wave, Current, and Flooding Data: This submission shall include a description of available tidal elevation, windspeed, and wave data that relate to study analysis requirements.

The submission shall include an evaluation of local and regional tide gage records while recognizing that these records include astronomical tide, surge, and possibly other influences (e.g., river flows and wave setup). Residuals based on astronomical tide predictions also shall be included where relevant to the study analysis. The submission shall include the review and selection of wind stations in the vicinity of the study area that can provide reasonable length of record, hourly values, and peak gusts to help estimate extreme wind statistics; the evaluation of available wave or wave hindcast data; the evaluation of available current data and the influence of currents on coastal flooding, if any; and the evaluation of available historical data (measured and anecdotal) on past coastal flood events.

Site Reconnaissance: The results of the site reconnaissance shall be documented to characterize exposure and coastal morphology, inventory existing coastal structures and levees (including buried coastal structures), identify shorelines where beach nourishment has occurred and could influence coastal flooding analyses and mapping, characterize coastal vegetation where it may influence coastal flooding analyses and mapping, locate analysis transects for subsequent field survey and ultimate use in wave calculations, and identify representative reaches with similar exposure, morphology, and features.

Technical Approach: The submission shall describe the technical approach for the analysis of coastal processes and the mapping of flood hazards in the various settings and shoreline morphologies present in the study area.

Hydrodynamic Storm-surge model: This section of the engineering report should address the hydrodynamic storm-surge model employed in performing the coastal study. The model used to calculate the surge elevation has been described in detail in various FEMA documents and only need be cited by reference. The Mapping Partner shall:

- Report the unique model characteristics used for the study, including a discussion of the specific grid system and sub-grid systems employed, the grid used for bottom topography (bathymetry) and the shoreline, small-scale features such as harbors and barrier islands, and the location and conditions applied for the open boundaries to the grid.

- Describe and document the adjustments made to land features to account for erosion.

- Describe and document the method used to determine average ground elevations and water depths within the cells of the grid system. (This discussion is to be augmented by diagrams that show the grid systems as computer listings of the grid data used in the actual model calculations.)

- Describe the method used to relate windspeed to the surface drag coefficient.

- Discuss the Manning's "n" values used in the calculation of bottom and overland friction and provide values in tabular form, including a discussion of any sensitivity tests used to estimate these values in nearshore water. (Nearshore, bottom, and overland friction are important parts of the overall analysis and shall be described with care and in sufficient detail.)

- Provide a graphical depiction of the model cells and grid system as an overlay to the bathymetric charts and topographic maps covering the study area, annotated with the individual cell inputs for the grid system.

- Discuss the treatment of barriers, inlets, and rivers.

- Explain the procedures used to determine inland flooding, including parameterization of local features and selection of the friction factors used for the terrain.

D.2.12.2.2 Intermediate Submission No. 2 – Storm-surge Model Calibration and Storm Selection

Documentation of this phase shall include a description of the calibration, validation and sensitivity analysis of the storm-surge model to be used in the generation of surge elevations for flood frequency-of-occurrence analysis. It shall also include a description of the selection and definition of storm events to be used in the statistical analysis.

Storm Climatology and Storm Windfield Methodology: The Mapping Partner shall describe the basic climatological storm data used and the windfield methodology. The Mapping Partner must map, tabulate, and discuss the methodology in terms of local surge impact and the storm paths used in the analysis. The Mapping Partner must also tabulate and describe in written form the storm parameters, including central pressure deficit, radius to maximum wind, forward speed, shoreline crossing point, and shoreline crossing angle, used in the analysis. It must also identify sources of the basic data used to develop the storm climatology and the method used to sort the data and compare them to the NWS Hurricane Climatology for the Atlantic and Gulf coasts of the United States (NWS 38, U.S. Department of Commerce, 1987). In addition, the Mapping Partner must describe the technique employed to determine the spatial/temporal distribution of storm occurrences (i.e., storms/nautical mile/year), derivation and discretization of storm intensity parameters, and exceedence probability distributions, and provide a graphical presentation of the results, including an overlay to show the orientation of the coast to storm path/direction. The Mapping Partner shall also provide a discussion of storm parameter independence and any unique storm model treatments.

The windfield used in the analysis is a key component in the determination of the storm surge elevation. The Mapping Partner shall give the exact equations used to parameterize the model windfield along with any unique values among the appropriate coefficients and constants used. The submission must include a diagram of the windfield model that shows the surface velocity structure as it changes radially outward from the storm center, provide a comparative graphic depiction of measured windfield(s) and the modeled windfield, if available, describe in detail the method by which winds are reduced as the storm approaches land and moves inland, and report the constants used in windspeed reduction.

Wave Data and Hindcasts: The submission shall describe data and analyses used to select and define storm events for use in response-based analysis of nearshore processes and subsequent statistical analysis of 1- and 0.2-percent-annual-chance flood conditions. Documentation shall include details of the sources of wave and wind data. It shall also include comparisons between alternate sources, in cases for which more than one is available and feasible for use in the study, and comparison with local measurements. Documentation of incident deepwater waves should include period, direction, and directional spreading parameters. The selection of coefficients for angular spreading and spectral peakedness parameters shall be clearly stated and justified.

Storm-surge model Calibration and Validation: The Mapping Partner shall document the calibration and validation of the hydrodynamic surge model. Once the hydrodynamic storm-surge model and grid have been constructed, calibration and validation are performed. Model calibration involves changing (fine tuning) values of model input parameters or coefficients in an attempt to replicate observed conditions within a given acceptable range. Validation confirms that the model has been calibrated to a set of conditions which are spatially and temporally representative of actual field conditions. Sensitivity runs are used to investigate the effect that small changes in the chosen grid and 'tuning parameters', will have on the computed flood and tide levels. Calibration and validation runs compare computed results with observed water levels. Sensitivity runs compare computed results with other computed results.

When observed (or model simulation) data are employed to calibrate (or compare) hydrodynamic storm-surge model results with other available studies, the Mapping Partner shall give a complete description of this calibration procedure (or model comparison), including a listing of measured and simulated tidal data. Calibration (and model comparison) is an important aspect of the model analysis; therefore, the Mapping Partner shall describe these activities with sufficient detail and care to allow an independent reviewer to understand the exact procedures and local historical records employed.

Sheltered Waters – Hindcast Waves: Documentation shall be provided on fetch length determination and corresponding windspeeds, directions, and durations for use in hindcast analyses. This shall include documentation of windspeed adjustments and windfield hindcast methods.

Sheltered Waters – Water Levels: The Mapping Partner shall document the characteristics of tide gages located within or near the study area that will potentially be used in study analyses or validation. Methods adopted to infer the variation of tidal datums between gages shall be documented, as shall procedures used to transpose data from one site to another. If a field effort is undertaken to determine the variation of tidal datum within ungaged regions, the Mapping Partner shall fully document that effort, including the locations of observations; a description of observation methods and instrumentation, dates and times of all observations, meteorological and oceanographic conditions during and preceding the period of observation, and other factors that may have influenced water levels or that may affect interpretation of the results. If surge variation was inferred from tide variation, the Mapping Partner shall document the basis for similarity assumptions and the manner in which the inferences were made. Inlet analyses shall be documented, including all procedures, methodological assumptions, field surveys (dates, times, procedures, instrumentation, and findings), and all inlet data adopted from other sources.

Proposed Transect Location Map: The Mapping Partner should submit one or more maps as appropriate depicting the location and orientation of transects to be used in the subsequent wave elevation determination analyses. The transect location map(s) should be at a suitable scale and should show transects of sufficient length to account for modeling of all coastal flooding conditions.

D.2.12.2.3 Intermediate Submission No. 3 – Storm-surge Modeling and Flood-Frequency Analysis

Documentation shall be provided on the methods used to estimate the 1- and 0.2-percent–annual-chance coastal flooding conditions. Documentation may include response-based and simulation methods (e.g., JPM, Monte Carlo, or EST), depending on study setting. Methods of extrapolation of hindcast and/or measured data to 1- and 0.2-percent–annual-chance values should be documented, including comparisons between alternate procedures, if appropriate. In cases for which extreme value analyses of wave, wind, water level, and residual tides are used, the submission shall include documentation of the analyses to develop frequency relationships, including a description of the data sets and analytical assumptions.

Joint Probability Methodology (JPM): If the JPM is used, the Mapping Partner shall summarize, map, and report the values and combinations used for storm parameters, annual storm density, spacing between storms, and the storm tracks used in the analysis in this section of the engineering report. The Mapping Partner shall compare the information above to the probabilities reported in the NWS Hurricane Climatology for the Atlantic and Gulf coasts of the United States (NWS 38, U.S. Department of Commerce, 1987). Specifically, the Mapping Partner shall:

- Note the total number of simulations.
- Summarize tidal elevation data, if used, in sufficient detail to remove any doubt as to the values used in the simulations.
- Describe the method used to determine the contribution of tide to the total water level.
- Describe storm occurrence rate or storm density, the definition of the storm region used to define storm density, and storm kinematics and intensity with respect to their use in the joint probability calculation.
- Report and discuss comparisons to long-term gage statistics.
- Describe and report adjustments to account for the combined probability of coastal and riverine flooding for each area where this approach was taken.

Monte Carlo Simulations: The requirements for Monte Carlo study documentation are similar to those described above for JPM studies, but should also include a tabulation of the cases randomly simulated, or specification of an algorithm by which those cases can be reconstructed. The Mapping Partner should also provide justification for the number of simulations, including appropriate evidence of convergence at the extreme levels.

Empirical Simulation Technique (EST): If the EST method is used, the Mapping Partner shall summarize all of the historical and hypothetical storms that were used and the manner of EST implementation. Specifically, the Mapping Partner shall:

- Document the storm occurrence rate for the study area, as used in the Poisson annual occurrence assumption;

- Document the historical storm selection process, listing all storms chosen for the analysis;
- Document the manner in which hypothetical storms were constructed, such as by track displacement of historical storms and/or by EST resampling and random walk procedures;
- Document the source of wind and pressure data for all simulated historical storms;
- Summarize tidal elevation data and describe the methods by which the tides and surge are combined;
- Discuss any special steps taken to reduce the impact of sample error while addressing local geographic variability of storm occurrence and implications of period-of-record limitations;
- Report and discuss comparisons to long-term gage statistics; and
- Describe and report adjustments to account for the combined probability of coastal and riverine flooding for each area where this approach was taken.

D.2.12.2.4 Intermediate Submission No. 4 – Nearshore Hydraulics

The nearshore hydraulics phase shall provide documentation of methods applied and detailed analyses conducted before the hazard zone mapping phase.

Wave Information: The Mapping Partner shall document all assumptions used to define waves. In sheltered waters, the documentation shall include a summary of fetch determination, winds (speed, direction, and duration), and bathymetry used in hindcasts. The documentation shall include the approximations or assumptions used in the analysis. When observational data, such as wave buoy data, are available, the wave height, period, and spectral parameters should be compared to the predicted waves.

Wave Transformation: The Mapping Partner shall document the assumptions, methods, and results of all analyses of wave transformations conducted for the study. This documentation shall include selection of offshore and nearshore points, source of transformation coefficients, and any special assumptions regarding local transformation processes, such as sheltering and reflection. If a spectral wave model is applied for nearshore transformation, all modeling factors shall be sufficiently documented so the modeling effort can be reproduced if necessary. If a field effort is undertaken to validate transformation models, the field work shall be summarized in detail, including times and locations of all observations, general conditions at the time the work was performed, a full description of all equipment and procedures, and a summary of all data in archival form. A description of the bathymetric data used in the transformation calculations shall also be provided.

Runup, Setup, and Overtopping Analyses: The Mapping Partner shall document the runup, setup, and overtopping analysis assumptions, methods, input data, and results. This shall include a determination of runup heights and stillwater elevations (SWELs) and determination of flood insurance risk zone parameters (1- and 0.2-percent-annual-chance flood depths, overtopping splash penetration and overtopping rate, and overland flow velocity) at each transect. This shall include a description of profiles used, runup reduction factors, and the basis for splash zones to be used in hazard mapping. The documentation shall include a description of any observations or measurements used to validate or adjust analysis results, any deviations from recommended procedures in Subsection D.2.8, any difficulties encountered in the analyses, and the technical decisions or approaches taken in their resolution. The Mapping Partner should include one or more transect location maps as appropriate and include computer printout listings for the input and output data, keyed to the transect location map(s), as an appendix to the report.

Wave Dissipation and Overland Propagation: The Mapping Partner shall describe the areas where wave attenuation was investigated, and document the analysis assumptions, methods, input data, and results. This shall include documentation of any field observations or measurements, as well as available historical or anecdotal information regarding wave attenuation during flooding events. The Mapping Partner should include one or more transect location maps as appropriate and include computer printout listings for the input and output data, keyed to the transect location map(s), as an appendix to the report.

Coastal Armoring Structures: The Mapping Partner shall describe assumptions and investigations of the various coastal armoring structures (e.g., seawalls, revetments, bulkheads, levees, etc.) in the study area relevant to stability and capability to withstand 1-percent-annual-chance water-level and wave conditions. This documentation shall include any assumptions or approximations used in the analyses. The same documentation shall be required in the event that coastal structures are apparently buried and not visible, but are indicated by information collected during the study. In cases where the Mapping Partner could not determine whether a given structure would survive the 1-percent-annual-chance flood intact, and where multiple analyses were conducted for the structure (i.e., intact condition, failed condition/removed from the analysis transect), the Mapping Partner shall document each analysis and record the structure condition used to map flood insurance risk zones and BFEs. This information will be useful in the event a map revision is requested based upon a structure condition different from that used as the basis for the FIRM. The Mapping Partner shall consult with the FEMA Study Representative regarding the treatment of levees (single levees or multiple-levee systems) during the study.

Beach Stabilization Structures: The Mapping Partner shall document the treatment of beach stabilization structures (e.g., groins, offshore breakwaters, sills, etc.) during the study. If the Mapping Partner proposes removal or modification of beach stabilization structures (or their shoreline effects) during the 1-percent-annual-chance flood, the Mapping Partner shall document the existence, history of, and shoreline response to beach stabilization structures and consult with the FEMA Study Representative.

Miscellaneous Structures: If miscellaneous structures (e.g., piers, port and navigation structures, bridges, culverts, tide gates, etc.) are present in the study area and could exert a significant influence on nearshore waves, currents, sediment transport, or backshore ponding, the Mapping Partner shall document the data, methods, and procedures used to evaluate the stability of these structures during the 1-percent-annual-chance flood and their effects on coastal flooding. This documentation shall include assumptions or approximations used in the analyses.

Erosion Analyses: The Mapping Partner shall document the erosion analysis assumptions, methods, input data, and results. If the erosion analysis is not preformed with the established erosion assessment methods (such as the 540-square-foot erosion criteria), the Mapping Partner shall provide historical documentation or other justification to provide that the method utilized will yield a feasible and technically sound eroded profile. The Mapping Partner shall document any unusual conditions in the study area and the methods proposed to map hazard zones based on these conditions. These may include the effects of beach nourishment and/or floodborne debris; special hydrodynamic considerations in tidal inlets and passages; the effects of riverine inflows, unusual erosion or other sedimentation characteristics; unusual structure effects and/or the effects of multiple levees, and any other factors that the Mapping Partner considers relevant to mapping flood hazards accurately.

D.2.12.2.5 Intermediate Submission No. 5 – Draft Flood Hazard Mapping

The draft flood hazard mapping phase shall provide documentation of the methods used to convert the results of the detailed hydraulic analyses into flood insurance risk zones.

Flood insurance risk zone Limit Identification: The Mapping Partner shall document the analysis results used in the determination of hazard zone limits and BFEs. This shall include a summary table, by transect, of results for 1-percent wave envelope, 1-percent SWEL, and determination of flood insurance risk zone parameters (1-percent and 0.2-percent flood depths, overtopping splash penetration and overtopping rate, overland flow velocity, overland wave propagation, and PFD location), as appropriate. In addition, the summary shall include a description of the basis for erosion and coastal structure conditions (e.g., overtopping cases, method of profile determination, failed and buried coastal structure cases, etc.) used in the determination of the hazard zones.

Flood insurance risk zone Map Boundary Delineation: The Mapping Partner shall provide draft work maps for the study area showing all flood insurance risk zone limits identified along the transects resulting from the detailed analyses and transferred to the topographic work maps. This submission shall describe the engineering judgment used to interpolate and delineate hazard zones between transects, including land features that might affect flood hazards, changes in contours, and the lateral extent of coastal structures. It shall also provide detailed documentation and technical justification of adjustments in the hazard zone mapping due to observed historical flood data and/or damages in the study area.

The Mapping Partner shall also incorporate all intermediate submissions and modifications based on review comments in each phase into the Engineering Analyses section of the TSDN.

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D.2.14 Notation

Symbol	Description	Units	Typical Units English	Typical Units SI
A	Equilibrium beach profile coefficient	$L^{1/3}$	$ft^{1/3}$	$m^{1/3}$
B	Berm height	L	ft	m
C	Wave phase velocity or celerity	L/T	ft/s	m/s
C_G	Wave group velocity	L/T	ft/s	m/s
C_k	Sample kurtosis	--	--	--
C_p	Plant drag coefficient	--	--	--
C_s	Sample skewness	--	--	--
C_0	Deepwater wave celerity, $gT/2\pi$	L/T	ft/s	m/s
D	Quarrystone diameter	L	ft	m
	Dune height	L	ft	m
D_{50}	Size of 50th percentile of sediment	L	mm	mm
d_h	Depth over berm	L	ft	m
E	Wave energy	LF/L	ft-lb/ft	N-m/m
	Crest elevation of structure	L	ft	m
$E_{HotSpot}$	Extra profile lowering at a hot spot	L	ft	m
E_j	Beach-dune juncture elevation	L	ft	m
E_{jMLWP}	Beach-dune juncture elevation for the MLWP	L	ft	m
E_{jStorm}	Beach dune juncture elevation during a storm	L	ft	m
E_T	Total still water elevation	L	ft	m
e	Base of natural logarithms (=2.718)	--	--	--
F	Cumulative probability function	--	--	--
F_c	Freeboard	L	ft	m
F'	Dimensionless freeboard	--	--	--
F_H, F_T, F_{gamma}, F_{slope}	Static setup coefficients in DIM model	--	--	--
F_n	Discrete spectral frequency	1/T	hz	hz
F_R	Wind wave runup coefficient	--	--	--
f	Wave frequency	1/T	hz	hz
	Darcy-Weisbach resistance coefficient	--	--	--
	Probability density function	--	--	--
f_e	Coriolis coefficient	1/T	1/S	1/S
f_p	Spectral peak frequency, $1/T_p$	1/T	hz	hz
G_H, G_T, G_{gamma}, G_{slope}	Dynamic setup coefficients in DIM model	--	--	--
G_0	Normalizing function for directional spectrum spreading function	--	--	--

Symbol	Description	Units	Typical Units English	Typical Units SI
$G(\theta)$	Directional spectrum spreading function	--	--	--
$G(f,\theta)$	Directional spreading function	--	--	--
g	Gravitational acceleration	L/T^2	ft/s^2	m/s^2
H	Wave height	L	ft	m
H'_o	Unrefracted deep water wave height	L	ft	m
H_b	Breaking wave height	L	ft	m
H_{m0}	Spectral significant wave height	L	ft	m
H_o	Deep water wave height	L	ft	m
H_s	Significant wave height	L	Ft	m
H_x	Wave height at x location in surf zone	L	ft	m
h_*	Wave structure parameter	--	--	--
h	Water depth	L	ft	m
h_b	Breaker depth	L	ft	m
h_c	Depth over crest	L	ft	m
h_m	Height of the land barrier	L	ft	m
h_o	Depth over crest	L	ft	m
K_s	Shoaling coefficient	--	--	--
$K_s(f_n)$	Spectral shoaling coefficient	--	--	--
$K_r(f_n, \theta_{o,n,m})$	Spectral refraction coefficient	--	--	--
k	Wave number, $2\pi/L$	rad/T	rad/ft	rad/m
	Bluff erosion parameter	--	--	--
L	Likelihood	--	--	--
LL	Log-likelihood	--	--	--
L_{berm}	Berm width	L	ft	m
L_{om}	Spectral deep water wave length	L	ft	m
L_0	Deep water wave length, $gT^2/2\pi$	L	ft	m
$M(n)$	Number of direction components in spectrum at f_n	--	--	--
m	Beach slope (rise/run)	L/L	--	--
m_n	n^{th} moment of spectral density, $\int_{f_1}^{f_2} f^n S(f)df$	L^2/T^n	ft^2/s^n	m^2/s^n
N	Degrees of freedom of a chi-squared distribution	--	--	--
	Number of waves	--	--	--

Symbol	Description	Units	Typical Units English	Typical Units SI
P	Average porosity of rubble structure cover layer	--	--	--
	Precipitation rate	L/T	in./hr	mm/hr
	Probability	--	--	--
Q	Dimensionless overtopping	--	--	--
q	Mean overtopping rate per unit length	L^2/T	ft^2/s	m^2/s
R	Total wave runup	L	ft	m
	Spearman statistic	--	--	--
R_{iuc}	Incident wind wave runup	L	ft	m
R_m	Reduced recession due to storm duration	L	ft	m
R_{Total}	Total runup (static setup plus dynamic setup plus incident wave runup.)	--	--	--
$R_{2\%}$	Runup exceeded by 2% of the runup crest	L	ft	m
R_∞	Maximum potential profile recession	L	ft	m
$R_{\infty HotSpot}$	Potential recession at a hot spot	L	ft	m
$R_{\infty storm}$	Potential recession for storm	L	ft	m
r	Linear correlation coefficient			
S	Water level change	L	ft	m
S_c	Compressive strength of bluff material	F/L^2	lb/ft^2	N/m^2
$S(f)$	Spectral density	L^2-T^2	ft^2/hz	m^2/hz
$S(f,\theta)$	Directional spectral density	L^2T/deg	$(ft^2/hz)/deg$	$(m^2/hz)/deg$
$S_0\ (fn,\ \theta_{o,n,m})$	Discrete directional spectrum in deep water	L^2-T^2	ft^2/hz	m^2/hz
$S_{ns}\ (fn,\ \theta_{o,n,m})$	Discrete directional spectrum in nearshore	L^2-T^2	ft^2/hz	m^2/hz
s	Sample standard deviation	--	--	--
$S(f)$	Continuous spectrum	L^2T	ft^2/hz	m^2/hz
T	Wave period	T	s	s
T_D	Storm duration	T	hr	hr
T_p	Spectral peak period, $1/f_p$	T	s	s
T_s	Significant wave period	T	S	S
	Time scale for beach profile response	--	--	--
t	Time	T	s	s
V_c	Velocity at crest	L/T	ft/s	m/s
V_f	Fall velocity	L/T	ft/s	m/s

Symbol	Description	Units	Typical Units English	Typical Units SI
V_{max}	Maximum overtopping volume per wave per unit length	L^2/wave	ft^2/wave	m^2/wave
v	Horizontal (y) component of local fluid velocity (water particle	L/T	ft/s	m/s
	velocity)	L/T	ft/s	m/s
W	Windspeed	L/T	mi/hr	m/s
W_b	Surf zone width to breaker line	L	ft	m
W_c	Wind stress coefficient term	L/T	mph	kph
W_x	x component of windspeed	L/T	mi/hr	m/s
W_y	y component of windspeed	L/T	mi/hr	m/s
X	Accumulated bluff to erosion	L	ft	m
$\bar{x}$	Sample mean	--	--	--
x,y,z	Right-handed Cartesian coordinates	L	ft	m
$y_{G,inner}$	Seaward extent of overtopping	L	ft	m
$y_{G,outer}$	Landward extent of overtopping	L	ft	m
y_0	Cross-shore location of structure crest	L	ft	m
z_c	Structure crest elevation	L	ft	m
z_G	Elevation behind crest	L	ft	m
$(\ )_b$	Term evaluated at the breaker line	--	--	--
$(\ )o$	Term evaluated in deep water	--	--	--
tan α	Structure slope	--	--	--
α	Storm duration recession reduction factor	--	--	--
	JONSWAP Spectrum term	L^2T	ft^2/hz	m^2/hz
α_c	Structure crest slope	--	--	--
β	Storm profile response coefficient	--	--	--
	Wave angle at structure	deg	deg	deg
γ	Specific gravity of a fluid	F/L^3	lb/ft^3	N/m^3
	Peak enhancement factor used in the JONSWAP spectrum	--	--	--
γ_b	Breaker depth index	--	--	--
	Runup berm coefficient	--	--	--
$\gamma_r\ \gamma_p\ \gamma_b\ \gamma_f,\ \gamma_\beta$	Runup reduction coefficients	--	--	--
Δf	Frequency increment	1/T	hz	hz
ΔR	Potential excess runup	L	ft	m
ε	Energy dissipation rate	F/LT	lb/s-ft	n/m-s
$\hat{\eta}$	Dynamic or oscillating setup	L	ft	m
$\bar{\eta}$	Mean or static wave setup	L	ft	m
$\bar{\eta}_b$	Static setdown at the breaker point	L	ft	m

Symbol	Description	Units	Typical Units English	Typical Units SI
$\overline{\eta}_{max}$	Maximum static wave setup	L	ft	m
$\overline{\eta}_{min}$	Minimum static wave setup	L	ft	m
$\overline{\eta}_{o}$	Static setup at the shoreline	L	ft	m
$\eta(x,t)$	Displacement of water surface relative to SWL	L	ft	m
$\overline{\eta^2}$	Mean square of water surface fluctuations	L^2	ft^2	m^2
η_3	Coefficient of skewness	--	--	--
η_4	Coefficient of kurtosis	--	--	--
η_i	Water surface displacement by incident wave	L	ft	m
η_{rms}	rms value of free surface elevation	L	ft	m
$\overline{\theta}$	Overall mean wave direction	deg	deg	deg
θ	Direction of wave propagation	deg	deg	deg
θ_{main}	Main wave direction in a directional spectrum	deg	deg	deg
θ_m	Discrete wave direction	deg	deg	deg
$\theta_m(f)$	Mean wave direction as a function of frequency.	deg	deg	deg
κ	Breaker index	--	--	--
	Wind stress factor	--	--	--
μ	Population Mean	--	--	--
ν	Spectral narrowness parameter	--	--	--
ξ	Surf similarity parameter or Iribarren number	--	--	--
ξ_{om}	Spectral deep water ξ	--	--	--
ξ_0	Deep water ξ	--	--	--
π	Constant = 3.14159	--	--	--
ρ	Mass density of water	M/L^3	$slug/ft^3$	kg/m^3
ρ_a	Mass density of air	M/L^3	$slug/ft^3$	kg/m^3
ρ_{fw}	Mass density of fresh water	M/L^3	$slug/ft^3$	kg/m^3
ρ_s	Mass density of sediment	M/L^3	$slug/ft^3$	kg/m^3
Ω	Rotational speed of the earth	rad/T	rad/S	rad/S
φ	Latitude	deg	deg	deg
τ_x, τ_y	Wind stress	F/L^2	lb/ft2	N/m^2
σ	Population standard deviation	L	ft	m

D.2.15 Acronyms

The Federal Emergency Management Agency (FEMA) has an extensive list of acronyms posted on the FEMA website at <http://www.fema.gov/fhm/dl_cgs.shtm>, *Acronyms and Abbreviations*. The acronyms below are specific to this document and include some of the acronyms given in the FEMA list.

1-D	One-Dimensional
2-D	Two-Dimensional
BFE	Base Flood Elevation
BST	Bathystrophic Storm Tide
CDF	Cumulative Distribution Function
CDIP	Coastal Data Information Program
CEM	Coastal Engineering Manual
CERC	Coastal Engineering Research Center
CFR	Code of Federal Regulations
CHL	Coastal and Hydraulics Laboratory
DFIRM	Digital Flood Insurance Rate Map
DIM	Direct Integration Method
DWLX%	Dynamic water level X%
ENSO	El Niño, Southern Oscillation
EST	Empirical Simulation Technique
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
FNMOC	Fleet Numerical Meteorology and Oceanography Center
G&S	FEMA *Guidelines and Specifications*
GEV	Generalized extreme value
GIS	Geographic Information Systems
GROW	Global Reanalysis of Ocean Waves
JONSWAP	Joint North Sea Wave Project
JPM	Joint Probability Method
LIDAR	Light Detection and Ranging (System)
MHHW	Mean higher high water
MHLW	Mean higher low water
MHW	Mean high water
MII	Meteorology International Inc.
MLHW	Mean lower high water
MLLW	Mean lower low water
MLW	Mean low water
MLWP	Most Likely Winter Profile
MSL	Mean Sea Level
MTL	Mean tide level
MWD	Main wave direction
MWL	Mean water level
NAVD88	North American Vertical Datum of 1988
NDBC	National Data Buoy Center

NFIP	National Flood Insurance Program
NGDC	National Geophysical Data Center
NGVD29	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Survey
NWS	National Weather Service
PDF	Probability Density Function
PFD	Primary Frontal Dune
POT	Peak-over-threshold
rms	Root mean square
RWL	Reference water level
SOEN	Southern Oscillation El Niño
SPM	Shore Protection Manual
SPR	Special problem report
STWL	Static water level
SWEL	Still water elevation
SWL	Still water level
TAW	Technical Advisory Committee for Water Retaining Structures
TSDN	Technical Support Data Notebook
TWG	Technical Working Group
TWL	Total water level
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WHAFIS	Wave Height Analysis for Flood Insurance Studies
WIS	Wave Information Studies
WISE®	Watershed Information SystEm, a registered trademark of Watershed Solutions, Inc.

D.2.16 Glossary

Most of the coastal engineering terms in this glossary are from the Shore Protection Manual (USACE, 1984) and Coastal Engineering Manual (USACE, 2002) and are supplemented with additional terms relevant to hazard mapping. FEMA has an extensive glossary posted on the FEMA website at <http://www.fema.gov/fhm/dl_cgs.shtm> Glossary.

---------- A ----------

ACCRETION May be either natural or artificial. Natural accretion is the buildup of land, solely by the action of the forces of nature, on a beach by deposition of water- or airborne material. Artificial accretion is a similar buildup of land by reason of an act of man, such as the accretion formed by a GROIN, BREAKWATER, or beach fill deposited by mechanical means. Also AGGRADATION.

ADJUSTABLE GROIN A GROIN whose permeability can be changed, usually with gates or removable sections.

ADVANCE (of a beach) (1) A continuing seaward movement of the shoreline. (2) A net seaward movement of the shoreline over a specified time. Also PROGRESSION.

AEOLIAN See EOLIAN.

ALIGNMENT The course along which the center line of a channel, canal or drain is located.

ALLUVIAL DEPOSITS Detrital material which is transported by a river and deposited—usually temporarily—at points along the flood plain of a river. Commonly composed of sands and gravels.

ALLUVIAL PLAIN A plain bordering a river, formed by the deposition of material eroded from areas of higher elevation.

ALLUVIUM Soil (sand, mud, or similar detrital material) deposited by streams, or the deposits formed.

ALONGSHORE Parallel to and near the shoreline; LONGSHORE.

AMPLITUDE, WAVE (1) The magnitude of the displacement of a wave from a mean value. An ocean wave has an amplitude equal to the vertical distance from still-water level to wave crest. For a sinusoidal wave, the amplitude is one-half the wave height. (2) The semirange of a constituent tide.

ANGLE OF REPOSE The maximum slope (measured from the horizontal) at which soils and loose materials on the banks of canals, rivers or embankments will stay stable.

ANISOTROPIC Having properties that change with changing directions.

ANOXIC Refers to an environment that contains little or no dissolved oxygen and hence little or no benthic marine life. These conditions arise in some basins or fjords where physical circulation of seawater is limited.

ANTIDUNES BED FORMS that occur in trains and are in phase with, and strongly interact with, gravity water-surface waves.

APRON Layer of stone, concrete or other material to protect the toe of a structure.

AQUIFER A geologic formation that is water-bearing, and which transmits water from one point in the formation to another.

ARCHIPELAGO A sea that contains numerous islands; also the island group itself.

ARMOR LAYER Protective layer on a BREAKWATER or SEAWALL composed of armor units.

ARMOR UNIT A relatively large quarrystone or concrete shape that is selected to fit specified geometric characteristics and density. It is usually of nearly uniform size and usually large enough to require individual placement. In normal cases it is used as primary wave protection and is placed in thicknesses of at least two units.

ARTIFICIAL NOURISHMENT The process of replenishing a beach with material (usually sand) obtained from another location.

ASTRONOMICAL TIDE The tidal levels and character which would result from gravitational effects, e.g. of the Earth, Sun, and Moon, without any atmospheric influences.

ATTENUATION (1) A lessening of the amplitude of a wave with distance from the origin. (2) The decrease of water-particle motion with increasing depth. Particle motion resulting from surface oscillatory waves attenuates rapidly with depth, and practically disappears at a depth equal to a surface wavelength.

---------- B ----------

BACKRUSH The seaward return of the water following the uprush of the waves. For any given tide stage the point of farthest return seaward of the backrush is known as the limit of backrush or limit backwash.

BACKSHORE That zone of the shore or beach lying behind the upper swash zone.

BACKWASH RIPPLES Low amplitude ripple marks formed on fine sand beaches by the backwash of the waves.

BANK (1) The rising ground bordering a lake, river, or sea or of a river or channel, for which it is designated as right or left as the observer is facing downstream. (2) An elevation of the sea floor or large area, located on a continental (or island) shelf and over which the depth is relatively shallow but sufficient for safe surface navigation (e.g., Georges Bank); a group of shoals. (3) In its secondary sense, used only with a qualifying word such as "sand bank" or "gravel bank," a shallow area consisting of shifting forms of silt, sand, mud, and gravel.

BAR A submerged or emerged embankment of sand, gravel, or other unconsolidated material built on the sea floor in shallow water by waves and currents.

BARRIER SPIT Similar to a barrier island, but connected to the mainland.

BASIN A depressed area with no surface outlet, such as a lake basin or an enclosed sea.

BASIN, BOAT A naturally or artificially enclosed or nearly enclosed harbor area for small craft.

BATHYMETRIC CHART A topographic map of the bed of the ocean, with depths indicated by contours (isobaths) drawn at regular intervals.

BATHYMETRY The measurement of depths of water in oceans, seas, and lakes; also information derived from such measurements.

BAY A recess in the shore or an inlet of a sea between two capes or headlands, not as large as a gulf but larger than a cove.

BAYMOUTH BAR A bar extending partly or entirely across the mouth of a bay.

BEACH The zone of unconsolidated material that extends landward from the low water line to the place where there is marked change in material or physiographic form, or to the line of permanent vegetation (usually the effective limit of storm waves). The seaward limit of a beach—unless otherwise specified—is the mean low water line. A beach includes foreshore and backshore. See also SHORE, SUSTAINABLE BEACH, AND SELF-SUSTAINING BEACH.

BEACH BERM A nearly horizontal part of the BEACH or BACKSHORE formed by the deposit of material by wave action. Some beaches have no BERMS, others have one or several.

BEACH CREST The point representing the limit of normal high tide wave run-up (see BERM CREST).

BEACH EROSION The carrying away of beach materials by wave action, tidal currents, littoral currents, or wind.

BEACH FACE The section of the beach normally exposed to the action of the wave uprush. The FORESHORE of a BEACH. (Not synonymous with SHOREFACE.)

BEACH FILL Material placed on a beach to renourish eroding shores.

BEACH HEAD The cliff, dune or seawall looming above the landward limit of the active beach.

BEACH MATERIAL Granular sediments, usually sand or shingle moved by the sea.

BEACH PLAN SHAPE The shape of the beach in plan, usually shown as a contour line, combination of contour lines, or recognizable features such as beach crest and/or the still water line.

BEACH PROFILE A cross-section taken perpendicular to a given beach contour; the profile may include the face of a dune or seawall, extend over the backshore, across the foreshore, and seaward underwater into the nearshore zone.

BEACH RIDGE See RIDGE, BEACH.

BEACH SCARP See SCARP, BEACH.

BEACH WIDTH The horizontal dimension of the beach measured normal to the shoreline and landward of the higher-high tide line (on oceanic coasts) or from the still water level (on lake coasts).

BED The bottom of a watercourse or any body of water.

BED FORMS Any deviation from a flat bed that is readily detectable by eye and higher than the largest sediment size present in the parent bed material; generated on the bed of an alluvial channel by the flow.

BED LOAD Sediment transport mode in which individual particles either roll or slide along the bed as a shallow, mobile layer a few particle diameters deep, the part of the load that is not continuously in suspension.

BED PROTECTION A (rock) structure on the bed that protects the underlying bed against erosion due to current and/or wave action.

BED SHEAR STRESS The way in which waves (or currents) transfer energy to the sea bed.

BEDDING PLANE A surface parallel to the surface of deposition, which may or may not have a physical expression. The original attitude of a bedding plane should not be assumed to have been horizontal.

BEDROCK The solid rock that underlies gravel, soil, and other superficial material. Bedrock may be exposed at the surface (an outcrop) or it may be buried under a few centimeters to thousands of meters of unconsolidated material.

BENCH (1) A level or gently sloping erosion plane inclined seaward. (2) A nearly horizontal area at about the level of maximum high water on the sea side of a dike.

BENCHMARK A permanently fixed point of known elevation. A primary bench mark is one close to a tide station to which the tide staff and tidal datum originally are referenced.

BENCHMARK, TIDAL A bench mark whose elevation has been determined with respect to mean sea level at a nearby tide gage; the tidal bench mark is used as reference for that tide gage.

BENEFITS The asset value of a scheme, usually measured in terms of the cost of damages avoided by the scheme, or the valuation of perceived amenity or environmental improvements

BENTHIC Pertaining to the sub-aquatic bottom.

BENTHOS Those animals who live on the sediments of the sea floor, including both mobile and non-mobile forms.

BERM (1) On a beach, a nearly horizontal plateau on the beach face or backshore, formed by the deposition of beach material by wave action or by means of a mechanical plant as part of a beach renourishment scheme. Some natural beaches have no berm, others have several. (2) On a structure: a nearly horizontal area, often built to support or key-in an armor layer.

BERM, BEACH See BEACH BERM.

BERM BREAKWATER Rubble mound structure with horizontal berm of armor stones at about sea level, which is allowed to be (re)shaped by the waves.

BERM CREST The seaward limit of a BERM. Also called BERM EDGE.

BIFURCATION Location where a river separates in two or more reaches or branches (the opposite of a confluence).

BIGHT A bend in a coastline forming an open bay. A bay formed by such a bend.

BIOTURBATION The disturbance of sediment bedding by the activities of burrowing organisms.

BIRDFOOT DELTA A river delta formed by many levee-bordered distributaries extending seaward and resembling in plan the outstretched claws of a bird. Example: Mississippi River delta.

BLANKET (FOUNDATION or BEDDING) A layer or layers of graded fine stones underlying a BREAKWATER, GROIN or rock embankment to prevent the natural bed material from being washed away.

BLOWOUT A depression on the land surface caused by wind erosion.

BLUFF A high, steep bank or cliff.

BOG A wet, spongy, poorly drained area which is usually rich in very specialized plants, contains a high percentage of organic remnants and residues and frequently is associated with a spring, seepage area, or other subsurface water source. A bog sometimes represents the final stage of the natural processes of eutrophication by which lakes and other bodies of water are very slowly transformed into land areas.

BOIL An upward flow of water in a sandy formation due to an unbalanced hydrostatic pressure resulting from a rise in a nearby stream, or from removing the overburden in making excavations.

BOLD COAST A prominent landmass that rises steeply from the sea.

BORE A very rapid rise of the tide in which the advancing water presents an abrupt front of considerable height. In shallow estuaries where the range of tide is large, the high water is propagated inward faster than the low water because of the greater depth at high water. If the high water overtakes the low water, an abrupt front is presented, with the high-water crest finally falling forward as the tide continues to advance. Also EAGER.

BOTTOM (nature of) The composition or character of the bed of an ocean or other body of water (e.g., clay, coral, gravel, mud, ooze, pebbles, rock, shell, shingle, hard, or soft).

BOTTOM BOUNDARY LAYER The lower portion of the water flow that experiences frictional retardation based on its proximity to the bed.

BOTTOMSET One of the horizontal or gently inclined sediment layers deposited in front of the advancing forest beds of a delta.

BOULDER A rounded rock more than 256 mm (10 inch) in diameter; larger than a cobblestone. See SOIL CLASSIFICATION.

BOUNDARY CONDITIONS Environmental conditions, e.g. waves, currents, drifts, etc. used as boundary input to physical or numerical models.

BRAIDED RIVER A river type with multiple channels separated by shoals, bars and islands.

BREACHING Failure of the beach head or a dike allowing flooding by tidal action.

BREAKER A wave breaking on a shore, over a REEF, etc. Breakers may be classified into four types: COLLAPSING--breaking occurs over lower half of wave, with minimal air pocket and usually no splash-up. Bubbles and foam present. PLUNGING--crest curls over air pocket; breaking is usually with a crash. Smooth splash-up usually follows. SPILLING--bubbles and turbulent water spill down front face of wave. The upper 25 percent of the front face may become vertical before breaking. Breaking generally occurs over quite a distance. SURGING--wave peaks up, but bottom rushes forward from under wave, and wave slides up beach face with little or no bubble production. Water surface remains almost plane except where ripples may be produced on the beachface during runback.

BREAKER DEPTH The still-water depth at the point where a wave breaks. Also called BREAKING DEPTH.

BREAKER INDEX Ratio of breaking wave height to deepwater wave height.

BREAKER ZONE The zone within which waves approaching the coastline commence breaking, typically in water depths of between 5 and 10 meters.

BREAKING Reduction in wave energy and height in the surf zone due to limited water depth.

BREAKWATER A structure protecting a shore area, harbor, anchorage, or basin from waves.

BUFFER AREA A parcel or strip of land that is designed and designated to permanently remain vegetated in an undisturbed and natural condition to protect an adjacent aquatic or wetland site from upland impacts, to provide habitat for wildlife and to afford limited public access.

BULKHEAD A structure or partition to retain or prevent sliding of the land. A secondary purpose is to protect the upland against damage from wave action.

BUOY A float; especially a floating object moored to the bottom, to mark a channel, anchor, shoal rock, etc. Some common types include: a nun or nut buoy is conical in shape; a can buoy is squat and cylindrical above water and conical below water; a spar buoy is a vertical, slender spar anchored at one end; a bell buoy, bearing a bell, runs mechanically or by the action of waves, usually marks shoals or rocks; a whistling buoy, similarly operated, marks shoals or channel entrances; a dan buoy carries a pole with a flag or light on it.

BUOYANCY The resultant of upward forces, exerted by the water on a submerged or floating body, equal to the weight of the water displaced by this body.

BYPASSING, SAND Hydraulic or mechanical movement of sand from the accreting updrift side to the eroding downdrift side of an inlet or harbor entrance. The hydraulic movement may include natural movement as well as movement caused by man.

---------- C ----------

CAISSON Concrete box-type structure.

CALIFORNIA CURRENT A deep-ocean boundary current that flows south-southeasterly along the U.S. west coast. The current is shallow, broad and slow moving carrying cold, nutrient poor waters toward the equator.

CALCAREOUS Containing calcium carbonate (C_aCO_3), chiefly as the minerals calcite and aragonite. When applied to rock, it implies that as much as 50 percent of the rock is carbonate (e.g., calcareous sand).

CALM The condition of the water surface when there is no wind waves or swell.

CANAL An artificial watercourse cut through a land area for such uses as navigation and irrigation.

CANYON A relatively narrow, deep depression with steep slopes, the bottom of which grades continuously downward. May be underwater (submarine) or on land (SUBAERIAL).

CAPE A land area jutting seaward from a continent or large island which prominently marks a change in, or interrupts notably, the coastal trend; a prominent feature. Examples: Cape Cod, Massachusetts; Cape Hatteras, North Carolina.

CAPILLARY WAVE A wave whose velocity of propagation is controlled primarily by the surface tension of the liquid in which the wave is traveling. Water waves of length less than about 1 inch are considered capillary waves. Waves longer than 1 inch and shorter than 2 inches are in an indeterminate zone between capillary and gravity waves.

CARTOGRAPHY The science and art of making maps.

CATCHMENT AREA The area which drains naturally to a particular point on a river, thus contributing to its natural discharge.

CAUSEWAY A raised road across wet or marshy ground, or across water.

CAUSTIC In refraction of waves, the name given to the curve to which adjacent orthogonals of waves refracted by a bottom whose contour lines are curved, are tangents. The occurrence of a caustic always marks a region of crossed orthogonals and high wave convergence.

CELERITY Wave speed.

CHANNEL (1) A natural or artificial waterway of perceptible extent which either periodically or continuously contains moving water, or which forms a connecting link between two bodies of water. (2) The part of a body of water deep enough to be used for navigation through an area otherwise too shallow for navigation. (3) A large strait, as the English Channel. (4) The deepest part of a stream, bay, or strait through which the main volume or current of water flows.

CHANNEL CAPACITY The maximum flow which a channel is capable of transmitting without its banks being overtopped.

CHANNEL-MOUTH BAR A bar built where a stream enters a body of standing water, resulting from decreased flow velocity.

CHART A special-purpose map, esp. one designed for navigation such as a bathymetric chart.

CHART DATUM The plane or level to which soundings (or elevations) or tide heights are referenced (usually LOW WATER DATUM). The surface is called a tidal datum when referred to a certain phase of tide. To provide a safety factor for navigation, some level lower than MEAN SEA LEVEL is generally selected for hydrographic charts, such as MEAN LOW WATER or MEAN LOWER LOW WATER. See DATUM PLANE.

CHEMICAL WEATHERING Disintegration of rocks and sediments by chemical alteration of the constituent minerals or of the cementing matrix. It is caused by exposure, oxidation, temperature changes, and biological processes.

CHOP The short-crested waves that may spring up quickly in a moderate breeze, and which break easily at the crest. Also WIND CHOP.

CHOPPY SEA Short, rough waves tumbling with a short and quick motion. Short-crested waves that may spring up quickly in a moderate breeze, and break easily at the crest.

CLAPOTIS The French equivalent for a type of STANDING WAVE. In American usage it is usually associated with the standing wave phenomenon caused by the reflection of a nonbreaking wave train from a structure with a face that is vertical or nearly vertical. Full clapotis is one with 100 percent reflection of the incident wave; partial clapotis is one with less than 100 percent reflection.

CLASTIC ROCKS Rocks built up of fragments which have been produced by weathering and erosion of pre-existing rocks and minerals and, typically, transported mechanically to their point of deposition.

CLAY A fine grained, plastic, sediment with a typical grain size less than 0.004 mm. Possesses electromagnetic properties which bind the grains together to give a bulk strength or cohesion. See SOIL CLASSIFICATION.

CLIFF A high, steep face of rock; a precipice. See also SEA CLIFF.

CLIMATE The characteristic weather of a region, particularly regarding temperature and precipitation, averaged over some significant internal of time (years).

CLOSURE DEPTH The water depth beyond which repetitive profile surveys (collected over several years) do not detect vertical sea bed changes, generally considered the seaward limit of littoral transport. The depth can be determined from repeated cross-shore profile surveys or estimated using formulas based on wave statistics. Note that this does not imply the lack of sediment motion beyond this depth.

CNOIDAL WAVE A type of wave in shallow water (i.e., where the depth of water is less than 1/8 to 1/10 the wavelength). The surface profile is expressed in terms of the Jacobian elliptic function cn u; hence the term cnoidal.

CO-TIDAL LINES Lines which link all the points where the tide is at the same stage (or phase) of its cycle.

COAST (1) A strip of land of indefinite width (may be several kilometers) that extends from the shoreline inland to the first major change in terrain features. (2) The part of a country regarded as near the coast.

COASTAL AREA The land and sea area bordering the shoreline.

COASTAL CURRENTS (1) Those currents which flow roughly parallel to the shore and constitute a relatively uniform drift in the deeper water adjacent to the surf zone. These currents may be tidal currents, transient, wind-driven currents, or currents associated with the distribution of mass in local waters. (2) For navigational purposes, the term is used to designate a current in coastwise shipping lanes where the tidal current is frequently rotary.

COASTAL DEFENSE General term used to encompass both coast protection against erosion and sea defense against flooding.

COASTAL FORCING The natural processes which drive coastal hydro- and morphodynamics (e.g. winds, waves, tides, etc).

COASTAL PLAIN The plain composed of horizontal or gently sloping strata of clastic materials, generally representing a strip of sea bottom that has emerged from the sea in recent geologic time.

COASTAL PROCESSES Collective term covering the action of natural forces on the shoreline, and near shore seabed.

COASTAL STRIP A zone directly adjacent to the waterline, where only coast related activities take place. Usually this is a strip of some 100 m wide. In this strip the coastal defense activities take place. In this strip often there are restrictions to land use.

COASTAL ZONE The transition zone where the land meets water, the region that is directly influenced by marine and lacustrine hydrodynamic processes. Extends offshore to the continental shelf break and onshore to the first major change in topography above the reach of major storm waves. On barrier coasts, includes the bays and lagoons between the barrier and the mainland.

COASTAL ZONE MANAGEMENT The integrated and general development of the coastal zone. Coastal Zone Management is not restricted to coastal defense works, but includes also a development in economical, ecological and social terms. Coastline Management is a part of Coastal Zone Management.

COASTLINE (1) Technically, the line that forms the boundary between the coast and the shore. (2) Commonly, the line that forms the boundary between the land and the water, esp. the water of a sea or ocean.

COBBLE A rock fragment between 64 and 256 mm in diameter, usually rounded. See SOIL CLASSIFICATION.

COFFERDAM A temporary watertight structure enclosing all or part of the construction area so that construction can proceed in the dry.

COHESIVE SEDIMENT Sediment containing significant proportion of clays, the electromagnetic properties of which cause the sediment to bind together.

COLLOID As a size term refers to particles smaller than 0.00024 mm, smaller than clay size.

COMBER (1) A deepwater wave whose crest is pushed forward by a strong wind; much larger than a whitecap. (2) A long-period breaker.

COMPETENCE The ability of a wind or water current to transport detritus, in terms of particle size rather than amount, measured as the diameter of the largest particles.

COMPLEX SPIT A large RECURVED SPIT with secondary spits developed at its end. Example: Sandy Hook, New Jersey.

CONFLUENCE The junction of two or more river reaches or branches (the opposite of a bifurcation).

CONSOLIDATION The gradual, slow compression of a cohesive soil due to weight acting on it, which occurs as water is driven out of the voids in the soil. Consolidation only occurs in clays or other soils of low permeability.

CONTINENTAL SHELF (1) The zone bordering a continent extending from the line of permanent immersion to the depth, usually about 100 m to 200 m, where there is a marked or rather steep descent toward the great depths of the ocean. (2) The area under active littoral processes during the HOLOCENE period. (3) The region of the oceanic bottom that extends outward from the shoreline with an average slope of less than 1:100, to a line where the gradient begins to exceed 1:40 (the CONTINENTAL SLOPE).

CONTINENTAL SLOPE The declivity from the offshore border of the CONTINENTAL SHELF to oceanic depths. It is characterized by a marked increase in slope.

CONTOUR A line on a map or chart representing points of equal elevation with relation to a DATUM. It is called an ISOBATH when connecting points of equal depth below a datum. Also called DEPTH CONTOUR.

CONTROLLING DEPTH The least depth in the navigable parts of a waterway, governing the maximum draft of vessels that can enter.

CONVERGENCE (1) In refraction phenomena, the decreasing of the distance between orthogonals in the direction of wave travel. Denotes an area of increasing wave height and energy concentration. (2) In wind-setup phenomena, the increase in setup observed over that which would occur in an equivalent rectangular basin of uniform depth, caused by changes in planform or depth; also the decrease in basin width or depth causing such increase in setup.

CORE (1) A cylindrical sample extracted from a beach or seabed to investigate the types and depths of sediment layers. (2) An inner, often much less permeable portion of a BREAKWATER or BARRIER BEACH.

CORIOLIS EFFECT Force due to the Earth's rotation, capable of generating currents. It causes moving bodies to be deflected to the right in the Northern Hemisphere and to the left in the Southern Hemisphere. The "force" is proportional to the speed and latitude of the moving object. It is zero at the equator and maximum at the poles.

COVE A small, sheltered recess in a coast, often inside a larger embayment.

COVER LAYER The outer layer used in a rubble system as protection against external hydraulic loads.

CREEK (1) A stream, less predominant than a river, and generally tributary to a river. (2) A small tidal Channel through a coastal MARSH.

CREEP Very slow, continuous downslope movement of soil or debris.

CRENULATE An indented or wavy shoreline beach form, with the regular seaward- pointing parts rounded rather than sharp, as in the cuspate type.

CREST Highest point on a beach face, BREAKWATER, or seawall.

CREST LENGTH, WAVE The length of a wave along its crest. Sometimes called CREST WIDTH.

CREST OF WAVE (1) the highest part of a wave. (2) That part of the wave above still-water level.

CREST OF BERM The seaward limit of a berm. Also called BERM EDGE.

CROSS-BEDDING An arrangement of relatively thin layers of rock inclined at an angle to the more nearly horizontal BEDDING PLANES of the larger rock unit. Also referred to as cross-stratification.

CROSS-SHORE Perpendicular to the shoreline.

CROWN WALL Concrete superstructure on a rubble mound.

CURRENT (1) The flowing of water, or other liquid or gas. (2) That portion of a stream of water which is moving with a velocity much greater than the average or in which the progress of the water is principally concentrated. (3) Ocean currents can be classified in a number of different ways. Some important types include the following: (1) Periodic - due to the effect of the tides; such Currents may be rotating rather than having a simple back and forth motion. The currents accompanying tides are known as tidal currents; (2) Temporary - due to seasonal winds; (3) Permanent or ocean - constitute a part of the general ocean circulation. The term DRIFT CURRENT is often applied to a slow broad movement of the oceanic water; (4) Nearshore - caused principally by waves breaking along a shore.

CURRENT, COASTAL One of the offshore currents flowing generally parallel to the shoreline in the deeper water beyond and near the surf zone; these are not related genetically to waves and resulting surf, but may be related to tides, winds, or distribution of mass.

CURRENT, DRIFT A broad, shallow, slow-moving ocean or lake current. Opposite of CURRENT, STREAM.

CURRENT, EBB The tidal current away from shore or down a tidal stream. Usually associated with the decrease in the height of the tide.

CURRENT, EDDY See EDDY.

CURRENT, FEEDER Any of the parts of the nearshore current system that flow parallel to shore before converging and forming the neck of the RIP CURRENT.

CURRENT, FLOOD The tidal current toward shore or up a tidal stream. Usually associated with the increase in the height of the tide.

CURRENT, INSHORE See INSHORE CURRENT.

CURRENT, LITTORAL Any current in the littoral zone caused primarily by wave action; e.g., LONGSHORE CURRENT, RIP CURRENT. See also CURRENT, NEARSHORE.

CURRENT, LONGSHORE The littoral current in the breaker zone moving essentially parallel to the shore, usually generated by waves breaking at an angle to the shoreline.

CURRENT METER An instrument for measuring the velocity of a current.

CURRENT, NEARSHORE A current in the NEARSHORE ZONE.

CURRENT, OFFSHORE See OFFSHORE CURRENT.

CURRENT, PERIODIC See CURRENT, TIDAL.

CURRENT, PERMANENT See PERMANENT CURRENT.

CURRENT, RIP See RIP CURRENT.

CURRENT, STREAM A narrow, deep, and swift ocean current, as the Gulf Stream. CURRENT, DRIFT.

CURRENT SYSTEM, NEARSHORE See NEARSHORE CURRENT SYSTEM.

CURRENT, TIDAL The alternating horizontal movement of water associated with the rise and fall of the tide caused by the astronomical tide-producing forces. Also CURRENT, PERIODIC. See also CURRENT, FLOOD and CURRENT, EBB.

CURRENT-REFRACTION Process by which wave velocity, height, and direction are affected by a current.

CUSP One of a series of short ridges on the FORESHORE separated by crescent-shaped troughs spaced at more or less regular intervals. Between these cusps are hollows. The cusps are spaced at somewhat uniform distances along beaches. They represent a combination of constructive and destructive processes. Also BEACH CUSP.

CUSPATE BAR A crescent-shaped bar uniting with the shore at each end. It may be formed by a single spit growing from shore and then turning back to again meet the shore, or by two spits growing from the shore and uniting to form a bar of sharply cuspate form.

CUSPATE SPIT The spit that forms in the lee of a shoal or offshore feature (BREAKWATER, island, rock outcrop) by waves that are refracted and/or diffracted around the offshore feature. It may eventually grow into a TOMBOLO linking the feature to the mainland.

CYCLOIDAL WAVE A steep, symmetrical wave whose crest forms an angle of 120 degrees and whose form is that of a cycloid. A trochoidal wave of maximum steepness. See also TROCHOIDAL WAVE.

CYCLONE A system of winds that rotates about a center of low atmospheric pressure. Rotation is clockwise in the Southern Hemisphere and anti-clockwise in the Northern Hemisphere. In the Indian Ocean, the term refers to the powerful storms called HURRICANES in the Atlantic.

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DAM Structure built in rivers or estuaries, basically to separate water at both sides and/or to retain water at one side.

DATUM Any permanent line, plane or surface used as a reference datum to which elevations are referred.

DATUM, CHART See CHART DATUM.

DATUM, PLANE The horizontal plane to which soundings, ground elevations, or water surface elevations are referred. Also REFERENCE PLANE. The plane is called a TIDAL DATUM when defined by a certain phase of the tide. The following datums are ordinarily used on hydrographic charts: MEAN LOW WATER--Atlantic coast (U. S.), Argentina, Sweden, and Norway. MEAN LOWER LOW WATER--Pacific coast (U. S.). LOW WATER DATUM--Great Lakes (U. S. and Canada). A common datum used on United States topographic maps is MEAN SEA LEVEL. See also BENCH MARK.

DAVIDSON CURRENT Deep-ocean boundary current off the west coast of the U.S. which brings warmer, saltier, low oxygen, high phosphate equatorial type water from low to high latitudes.

DEBRIS LINE A line near the limit of storm wave uprush marking the landward limit of debris deposits.

DECAY AREA Area of relative CALM through which waves travel after emerging from the generating area.

DECAY DISTANCE The distance waves travel after leaving the generating area (FETCH).

DECAY OF WAVES The change waves undergo after they leave a generating area (FETCH) and pass through a calm, or region of lighter winds. In the process of decay, the significant wave height decreases and the significant wavelength increases.

DEEP WATER Water so deep that surface waves are little affected by the ocean bottom. Generally, water deeper than one-half the surface wavelength is considered deep water. Compare SHALLOW WATER.

DEEP WATER WAVES A wave in water the depth of which is greater than one-half the WAVE LENGTH.

DEFLATION The removal of loose material from a beach or other land surface by wind action.

DEGRADATION The geologic process by means of which various parts of the surface of the earth are worn away and their general level lowered, by the action of wind and water.

DELTA (1) An ALLUVIAL DEPOSIT, usually triangular or semicircular, at the mouth of a river or stream. The delta is normally built up only where there is no tidal or current action capable of removing the sediment at the same rate as it is deposited, and hence the delta builds forward from the coastline. (2) A TIDAL DELTA is a similar deposit at the mouth of a tidal INLET, put there by TIDAL CURRENTS.

DELTA PLAIN The nearly-level surface composing the landward portion of a large DELTA.

DENSITY Mass (in kg) per unit of volume of a substance; kg/m3. For pure water, the density is 1,000 kg/m^3, for seawater the density is usually more. Density increases with increasing salinity, and decreases with increasing temperature. More information can be found in "properties of seawater". For stone and sand, usually a density of 2,600 kg/m^3 is assumed. Concrete is less dense, in the order of 2,400 kg/m^3. Some types of basalt may reach 2,800 kg/m^3. For sand, including the voids, one may use 1,600 kg/m^3, while mud often has a density of 1,100 – 1,200 kg/m^3.

DENSITY CURRENT Phenomenon of relative flow within water due to difference in density. For example, the salt-water wedge is a density current, as is a volcanic nuée ardente.

DENSITY STRATIFICATION The lateral expansion of a sediment plume as it moves out of the distributary mouth, where salt and fresh water mix. This is most likely to occur where the speed of the river flow is moderate to low and the distributary mouth is relatively deep.

DENSITY-DRIVEN CIRCULATION Variations in salinity create variations in density in estuaries. These variations in density create horizontal pressure gradients, which drive estuarine circulation.

DEPRESSION A general term signifying any depressed or lower area in the ocean floor.

DEPTH The vertical distance from a specified datum to the sea floor.

DEPTH CONTOUR See CONTOUR., also isobath.

DEPTH, CONTROLLING See CONTROLLING DEPTH.

DEPTH FACTOR See SHOALING COEFFICIENT.

DEPTH OF BREAKING The still-water depth at the point where the wave breaks. Also BREAKER DEPTH.

DERRICK STONE See STONE, DERRICK.

DESIGN HURRICANE See HYPOTHETICAL HURRICANE.

DESIGN STORM A hypothetical extreme storm whose waves coastal protection structures will often be designed to withstand. The severity of the storm (i.e. return period) is chosen in view of the acceptable level of risk of damage or failure. A DESIGN STORM consists of a DESIGN WAVE condition, a design water level and a duration.

DESIGN WAVE In the design of HARBORS, harbor works, etc., the type or types of waves selected as having the characteristics against which protection is desired.

DESIGN WAVE CONDITION Usually an extreme wave condition with a specified return period used in the design of coastal works.

DETACHED BREAKWATER A BREAKWATER without any SUBAERIAL connection to the shore.

DETRITUS Small fragments of rock which have been worn or broken away from a mass by the action of water or waves.

DIFFERENTIAL EROSION / WEATHERING These features develop in rocks which have varying resistance to the agencies of erosion and/or weathering so that parts of the rock are removed at greater rates than others. A typical example is the removal of soft beds from between harder beds in a series of sedimentary rocks. The term may be applied to any size of feature, from small-scale etching to the regional development of hills and valleys controlled by hard and soft rocks.

DIFFRACTION (of water waves) The phenomenon by which energy is transmitted laterally along a wave crest. When a part of a train of waves is interrupted by a barrier, such as a BREAKWATER, the effect of diffraction is manifested by propagation of waves into the sheltered region within the barrier's geometric shadow.

DIFFRACTION COEFFICIENT Ratio of diffracted wave height to deep water wave height.

DIKE Earth structure along sea or river in order to protect low lands from flooding by high water; dikes along rivers are sometimes called levees. Sometimes written as DYKE.

DISCHARGE The volume of water per unit of time flowing along a pipe or channel.

DITCH A channel to convey water for irrigation or drainage.

DIURNAL Having a period or cycle of approximately one TIDAL DAY.

DIURNAL CURRENT The type of tidal current having only one flood and one ebb period in the tidal day. A ROTARY CURRENT is diurnal if it changes its direction through all points of the compass once each tidal day.

DIURNAL INEQUALITY The difference in height of the two high waters or of the two low waters of each day. Also, the difference in velocity between the two daily flood or EBB CURRENTS of each day.

DIURNAL TIDE A tide with one high water and one low water in a tidal day.

DIVERGENCE (1) In refraction phenomena, the increasing of distance between orthogonals in the direction of wave travel. Denotes an area of decreasing wave height and energy concentration. (2) In wind-setup phenomena, the decrease in setup observed under that which would occur in an equivalent rectangular basin of uniform depth, caused by changes in planform or depth. Also the increase in basin width or depth causing such decrease in setup.

DIVERGING WAVE Waves that move obliquely out from a vessel's sailing line.

DIVERSION CHANNEL A waterway used to divert water from its natural course. The term is generally applied to a temporary arrangement e.g. to by-pass water around a dam site during construction.

DOCK The slip or waterway between two piers, or cut into the land, for the reception of ships.

DOLPHIN A cluster of piles.

DOWNDRIFT The direction of predominant movement of littoral materials.

DOWNSTREAM Along coasts with obliquely approaching waves there is a longshore (wave-driven) current. For this current, one can define an upstream and a DOWNSTREAM direction. For example, on a beach with an orientation west-east, the sea is to the north. Suppose the waves come from NW, then the current flows from West to East. Here, UPSTREAM is west of the observer, and east is downstream of the observer.

DOWNWELLING A downward movement (sinking) of surface water caused by onshore Ekman transport, converging CURRENTS, or when a water mass becomes more dense than the surrounding water.

DRAINAGE BASIN Total area drained by a stream and its tributaries.

DREDGING Excavation or displacement of the bottom or shoreline of a water body. Dredging can be accomplished with mechanical or hydraulic machines. Most is done to maintain channel depths or berths for navigational purposes; other dredging is for shellfish harvesting, for cleanup of polluted sediments, and for placement of sand on beaches.

DRIFT (noun) (1) Sometimes used as a short form for LITTORAL DRIFT. (2) The speed at which a current runs. (3) Floating material deposited on a beach (driftwood). (4) A deposit of a continental ice sheet; e.g., a DRUMLIN.

DRIFT CURRENT A broad, shallow, slow-moving ocean or lake current.

DRIFT SECTOR A particular reach of marine shore in which LITTORAL DRIFT may occur without significant interruption, and which contain any and all natural sources of such drift, and also any accretion shore forms accreted by such drift.

DROMOND A large medieval fast-sailing galley or cutter.

DROWNED COAST A shore with long, narrow channels, implying that subsidence of the coast has transformed the lower portions of river valleys into tidal estuaries.

DRUMLIN A low, smoothly-rounded, elongate hill of compact glacial till built under the margin of the ice and shaped by its flow.

DRYING BEACH That part of the beach which is uncovered by water (e.g. at low tide). Sometimes referred to as 'SUBAERIAL' beach.

DUNES (1) Ridges or mounds of loose, wind-blown material, usually sand. (2) Bed forms smaller than bars but larger than ripples that are out of phase with any water-surface gravity waves associated with them.

DURABILITY The ability of a rock to retain its physical and mechanical properties (i.e. resist degradation) in engineering service.

DURATION In wave forecasting, the length of time the wind blows in nearly the same direction over the FETCH (generating area).

DURATION, MINIMUM The time necessary for steady-state wave conditions to develop for a given wind velocity over a given fetch length.

DURATION OF EBB The interval of time in which a tidal current is ebbing, determined from the middle of the slack waters.

DURATION OF FALL The interval from high water to low water.

DURATION OF FLOOD The interval of time in which a tidal current is flooding, determined from the middle of slack waters.

DURATION OF RISE The interval from low water to high water.

DYNAMIC EQUILIBRIUM Short term morphological changes that do not affect the morphology over a long period.

DYNAMIC VISCOSITY In fluid dynamics, the ratio between the shear stress acting along any plane between neighboring fluid elements and the rate of deformation of the velocity gradient perpendicular to this plane.

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EAGER See BORE.

EBB Period when tide level is falling; often taken to mean the ebb current which occurs during this period.

EBB CURRENT The movement of a tidal current away from shore or down a tidal stream. In the semidiurnal type of reversing current, the terms greater ebb and lesser ebb are applied respectively to the ebb currents of greater and lesser velocity of each day. The terms of maximum ebb and minimum ebb are applied to the maximum and minimum velocities of a continuously running ebb current, the velocity alternately increasing and decreasing without coming to a slack or reversing. The expression maximum ebb is also applicable to any ebb current at the time of greatest velocity.

EBB INTERVAL The interval between the transit of the moon over the meridian of a place and the time of the following strength of ebb.

EBB SHIELD High, landward margin of a flood-tidal shoal that helps divert ebb-tide currents around the shoal.

EBB STRENGTH The EBB CURRENT at the time of maximum velocity.

EBB TIDAL DELTA The bulge of sand formed at the seaward mouth of TIDAL INLETS as a result of interaction between tidal currents and waves. Also called inlet-associated bars and estuary entrance shoals.

EBB TIDE The period of tide between high water and the succeeding low water; a falling tide.

ECHO SOUNDER An electronic instrument used to determine the depth of water by measuring the time interval between the emission of a sonic or ultrasonic signal and the return of its echo from the bottom.

ECOSYSTEM The living organisms and the nonliving environment interacting in a given area, encompassing the relationships between biological, geochemical, and geophysical systems.

EDDY A circular movement of water formed on the side of a main current. Eddies may be created at points where the main stream passes projecting obstructions or where two adjacent currents flow counter to each other.

EDDY CURRENT See EDDY.

EDGE WAVE An ocean wave parallel to a coast, with crests normal to the shoreline. An edge wave may be STANDING or PROGRESSIVE. Its height diminishes rapidly seaward and is negligible at a distance of one wavelength offshore.

EKMAN TRANSPORT Resultant flow at right angles to and to the right of the wind direction (in the northern hemisphere) referred to as UPWELLING and DOWNWELLING.

ELEVATION The vertical distance from mean sea level or other established datum plane to a point on the earth's surface; height above sea level. Although sea floor elevation below msl should be marked as a negative value, many charts show positive numerals for water depth.

EL NIÑO Warm equatorial water which flows southward along the coast of Peru and Ecuador during February and March of certain years. It is caused by poleward motions of air and unusual water temperature patterns in the Pacific Ocean, which cause coastal downwelling, leading to the reversal in the normal north-flowing cold coastal currents. During many El Niño years, storms, rainfall, and other meteorological phenomena in the Western Hemisphere are measurably different than during non-El Niño years. (See La Niña).

ELUTRIATION The process by which a granular material can be sorted into its constituent particle sizes by means of a moving stream of fluid (usually air or water). Elutriators are extensively used in studies of sediments for determining Particle size distribution. Under certain circumstances wind, rivers and streams may act as elutriating agents.

EMBANKMENT Fill material, usually earth or rock, placed with sloping sides and with a length greater than its height. Usually an embankment is wider than a dike.

EMBAYMENT An indentation in the shoreline forming an open bay.

EMERGENT COAST A coast in which land formerly under water has recently been exposed above sea level, either by uplift of the land or by a drop in sea level.

ENDEMIC Native or confined to a specific geographic area.

ENERGY COEFFICIENT The ratio of the energy in a wave per unit crest length transmitted forward with the wave at a point in shallow water to the energy in a wave per unit crest length transmitted forward with the wave in deep water. On refraction diagrams this is equal to the ratio of the distance between a pair of orthogonals at a selected shallow-water point to the distance between the same pair of orthogonals in deep water. Also the square of the REFRACTION COEFFICIENT.

ENTRANCE The avenue of access or opening to a navigable channel or inlet.

EOLIAN (also AEOLIAN) Pertaining to the wind, esp. used with deposits such as loess and dune sand, and sedimentary structures like wind-formed ripple marks.

EOLIAN SANDS Sediments of sand size or smaller which have been transported by winds. They may be recognized in marine deposits off desert coasts by the greater angularity of the grains compared with waterborne particles.

EQUATORIAL CURRENTS (1) Ocean currents flowing westerly near the equator. There are two such currents in both the Atlantic and Pacific Oceans. The one to the north of the equator is called the North Equatorial Current and the one to the south is called the South Equatorial Current. Between these two currents there is an easterly flowing stream known as the Equatorial Countercurrent. (2) Tidal currents occurring semimonthly as a result of the moon being over the equator. At these times the tendency of the moon to produce DIURNAL INEQUALITY in the current is at a minimum.

EQUATORIAL TIDES Tides occurring semimonthly as the result of the moon being over the equator. At these times the tendency of the moon to produce a DIURNAL INEQUALITY in the tide is at a minimum.

EROSION The wearing away of land by the action of natural forces. On a beach, the carrying away of beach material by wave action, tidal currents, littoral currents, or by deflation.

ESCARPMENT A more or less continuous line of cliffs or steep slopes facing in one general direction which are caused by erosion or faulting. Also SCARP.

ESTUARY (1) The part of a river that is affected by tides. (2) The region near a river mouth in which the fresh water of the river mixes with the salt water of the sea and which received both fluvial and littoral sediment influx.

EUSTATIC SEA LEVEL CHANGE Change in the relative volume of the world=s ocean basins and the total amount of ocean water.

EYE In meteorology, usually the "eye of the storm" (hurricane): the roughly circular area of comparatively light winds and fair weather found at the center of a severe tropical cyclone.

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FAIRWAY The parts of a waterway that are open and unobstructed for navigation. The main traveled part of a waterway; a marine thoroughfare.

FAR-INFRAGRAVITY The frequency band (nominally 0.001 - 0.02 Hz) occupied by SHEAR INSTABILITIES of the longshore current. This band falls both below and in the lower part of the Infragravity band occupied by Infragravity waves.

FATHOM A unit of measurement used for soundings equal to 1.83 meters (6 feet).

FATHOMETER The copyrighted trademark for a type of ECHO SOUNDER.

FAULT A fracture in rock along which there has been an observable amount of displacement. Faults are rarely single planar units; normally they occur as parallel to sub-parallel sets of planes along which movement has taken place to a greater or lesser extent. Such sets are called fault or fracture-zones.

FAUNA The entire group of animals found in an area.

FEEDER BEACH An artificially widened beach serving to nourish downdrift beaches by natural littoral currents or forces.

FEEDER CURRENT The currents which flow parallel to shore before converging and forming the neck of a RIP CURRENT.

FEEDER CURRENT See CURRENT, FEEDER.

FEELING BOTTOM The initial action of a deepwater wave, in response to the bottom, upon running into shoal water.

FETCH The area in which SEAS are generated by a wind having a fairly constant direction and speed. Sometimes used synonymously with FETCH LENGTH. Also GENERATING AREA.

FETCH LENGTH The horizontal distance (in the direction of the wind) over which a wind generates seas or creates a WIND SETUP.

FETCH-LIMITED Situation in which wave energy (or wave height) is limited by the size of the wave generation area (fetch).

FILTER Intermediate layer, preventing fine materials of an underlayer from being washed through the voids of an upper layer.

FIORD (FJORD) A narrow, deep, steep-walled inlet of the sea, usually formed by entrance of the sea into a deep glacial trough.

FIRTH A narrow arm of the sea; also, the opening of a river into the sea.

FLOOD (1) Period when tide level is rising; often taken to mean the flood current which occurs during this period (2) A flow beyond the carrying capacity of a channel.

FLOOD CHANNEL Channel located on ebb-tidal shoal that carries the flood tide over the tidal flat into the back bay or lagoon.

FLOOD CURRENT The movement of a tidal current toward the shore or up a tidal stream. In the semidiurnal type of reversing current, the terms greater flood and lesser flood are applied respectively to the flood currents of greater and lesser velocity each day. The terms maximum flood and minimum flood are applied to the maximum and minimum velocities of a flood current the velocity of which alternately increases and decreases without coming to slack or reversing. The expression maximum flood is also applicable to any flood current at the time of greatest velocity.

FLOOD GATE A gravity outlet fitted with vertically-hinged doors, opening if the inner water level is higher than the outer water level, so that drainage takes place during low water.

FLOOD INTERVAL The interval between the transit of the moon over the meridian of a place and the time of the following flood.

FLOOD MARK Proof of any kind on the shoreline, or on structures like bridge abutments, used to determine the highest level attained by the water surface during the flood (note: the height of the flood mark usually includes the wave run-up).

FLOOD PLAIN (1) A flat tract of land bordering a river, mainly in its lower reaches, and consisting of ALLUVIUM deposited by the river. It is formed by the sweeping of the meander belts downstream, thus widening the valley, the sides of which may become some kilometers apart. In time of flood, when the river overflows its banks, sediment is deposited along the valley banks and plains. (2) Synonymous with 100-year floodplain. The land area susceptible to being inundated by stream derived waters with a 1 percent chance of being equaled or exceeded in any given year.

FLOOD RAMP Seaward-dipping sand platform dominated by flood-tidal currents, located on ebb-tidal shoal near the opening to the inlet.

FLOOD ROUTING The determination of the attenuating effect of storage on a river-flood passing through a valley by reason of a feature acting as control (e.g. a reservoir with a spillway capacity less than the flood inflow, or the widening or narrowing of a valley).

FLOOD TIDAL DELTA The bulge of sand formed at the landward mouth of TIDAL INLETS as a result of flow expansion.

FLOOD TIDE The period of tide between low water and the succeeding high water; a rising tide.

FLOODWALL, SPLASH WALL Wall, retired from the seaward edge of the seawall crest, to prevent water from flowing onto the land behind.

FLORA The entire group of plants found in an area.

FLUVIAL Of or pertaining to rivers; produced by the action of a river or stream (e.g., fluvial sediment).

FLUSHING TIME The time required to replace all the water in an ESTUARY, HARBOR, etc., by action of current and tide.

FOAM LINE (1) The front of a wave as it advances shoreward, after it has broken. (2) Lines of foam such as those which move around the head of a RIP.

FOLLOWING WIND Generally, the same as a tailwind; in wave forecasting, wind blowing in the direction of ocean-wave advance.

FOREDUNE The front DUNE immediately behind the backshore.

FORERUNNER Low, long-period ocean SWELL which commonly precedes the main swell from a distant storm, especially a tropical cyclone.

FORESHORE The part of the shore, lying between the crest of the seaward berm (or upper limit of wave wash at high tide) and the ordinary low-water mark, that is ordinarily traversed by the uprush and backrush of the waves as the tides rise and fall. See BEACH FACE.

FORE REEF The seaward side of a REEF (usually coral); in places a steep slope covered with reef talus.

FORWARD SPEED (hurricane)Rate of movement (propagation) of the hurricane eye in meters per second, knots, or miles per hour.

FREEBOARD At a given time, the vertical distance between the water level and the top of the structure. On a ship, the distance from the waterline to main deck or gunwale.

FRINGING REEF A coral REEF attached directly to an insular or continental shore. There may be a shallow channel or lagoon between the reef and the adjacent mainland.

FRONT OF THE FETCH In wave forecasting, the end of the generating area toward which the wind is blowing.

FROUDE NUMBER The dimensionless ratio of the inertial force to the force of gravity for a given fluid flow. It may be given as $Fr = V/Lg$ where V is a characteristic velocity, is a characteristic length, and g the acceleration of gravity - or as the square root of this number.

FULLY-DEVELOPED SEA The waves that form when wind blows for a sufficient period of time across the open ocean. The waves of a fully developed sea have the maximum height possible for a given windspeed, FETCH and duration of wind.

---------- G ----------

GABION (1) Steel wire-mesh basket to hold stones or crushed rock to protect a bank or bottom from erosion. (2) Structures composed of masses of rocks, rubble or masonry held tightly together usually by wire mesh so as to form blocks or walls. Sometimes used on heavy erosion areas to retard wave action or as a foundation for BREAKWATERS or JETTIES.

GALE A wind between a strong breeze and a storm. A continuous wind blowing in degrees of moderate, fresh, strong, or whole gale and varying in velocity from 28 to 47 nautical miles per hour (see BEAUFORT SCALE).

GAGE (GAGE) Instrument for measuring the water level relative to a datum.

GENERATING AREA In wave forecasting, the continuous area of water surface over which the wind blows in nearly a constant direction. Sometimes used synonymously with FETCH LENGTH. Also FETCH.

GEOGRAPHICAL INFORMATION SYSTEM (GIS) Database of information which is geographically referenced, usually with an associated visualization system.

GEOMETRIC MEAN DIAMETER The diameter equivalent of the arithmetic mean of the logarithmic frequency distribution. In the analysis of beach sands, it is taken as that grain diameter determined graphically by the intersection of a straight line through selected boundary sizes, (generally points on the distribution curve where 16 and 84 percent of the sample is coarser by weight) and a vertical line through the median diameter of the sample.

GEOMETRIC SHADOW In wave diffraction theory, the area outlined by drawing straight lines paralleling the direction of wave approach through the extremities of a protective structure. It differs from the actual protected area to the extent that the diffraction and refraction effects modify the wave pattern.

GEOMORPHOLOGY (1) That branch of physical geography which deals with the form of the Earth, the general configuration of its surface, the distribution of the land, water, etc. (2) The investigation of the history of geologic changes through the interpretation of topographic forms.

GEOPHYSICS The study of the physical characteristics and properties of the earth, usually employing quantitative physical methods.

GEOTEXTILE A synthetic fabric which may be woven or non-woven used as a filter.

GLACIER A large body of ice moving slowly down a slope of valley or spreading outward on a land surface (e.g., Greenland, Antarctica) and surviving from year to year.

GLACIO-ISOSTACY The state of hydrostatic equilibrium of the earth=s crust as influenced by the weight of glacier ice.

GLOBAL POSITIONING SYSTEM (GPS) A navigational and positioning system developed by the U.S. Department of Defense, by which the location of a position on or above the Earth can be determined by a special receiver at that point interpreting signals received simultaneously from several of a constellation of special satellites.

GORGE (1) The deepest portion of an inlet, the THROAT. (2) A narrow, deep valley with nearly vertical rock walls.

GRADED BEDDING An arrangement of particle sizes within a single bed, with coarse grains at the bottom of the bed and progressively finer grains toward the top of the bed.

GRADIENT (1) A measure of slope (soil- or water-surface) in meters of rise or fall per meter of horizontal distance. (2) More general, a change of a value per unit of distance, e.g. the gradient in longshore transport causes erosion or accretion. (3) With reference to winds or currents, the rate of increase or decrease in speed, usually in the vertical; or the curve that represents this rate.

GRADING Distribution with regard to size or weight, of individual stones within a bulk volume; heavy, light and fine grading are distinguished.

GRADUAL CLOSURE METHOD Method in which the final closure gap in a dam is closed gradually either by the vertical or horizontal closure method; this in contradiction with a sudden closure.

GRANULAR FILTER A layer of granular material which is incorporated in an embankment, dam, dike, or bottom protection and is graded so as to allow seepage to flow across or down the filter zone without causing the migration of the material adjacent to the filter.

GRAVEL Unconsolidated natural accumulation of rounded rock fragments coarser than sand but finer than pebbles (2-4 mm diameter).

GRAVITY WAVE A wave whose velocity of propagation is controlled primarily by gravity. Water waves more than 5 cm long are considered gravity waves. Waves longer than 2.5 cm and shorter than 5 cm are in an indeterminate zone between CAPILLARY and GRAVITY WAVES. See RIPPLE.

GROIN (British, GROYNE) Narrow, roughly shore-normal structure built to reduce longshore currents, and/or to trap and retain littoral material. Most groins are of timber or rock and extend from a SEAWALL, or the backshore, well onto the foreshore and rarely even further offshore. See T-GROIN, PERMEABLE GROIN, IMPERMEABLE GROIN.

GROIN BAY The beach compartment between two groins.

GROIN SYSTEM A series of groins acting together to protect a section of beach. Commonly called a GROIN field.

GULF A relatively large portion of the ocean or sea extending far into land; the largest of various forms of inlets of the sea (e.g., Gulf of Mexico, Gulf of Aqaba).

GUT A tidal stream connecting two larger waterways.

---------- H ----------

HALCOCLINE A zone in which salinity changes rapidly.

HALF-TIDE LEVEL A plane midway between MEAN HIGH WATER and MEAN LOW WATER, also called MEAN TIDE LEVEL.

HARBOR (British, HARBOUR) Any protected water area affording a place of safety for vessels. See also PORT.

HARBOR OSCILLATION (HARBOR SURGING) The nontidal vertical water movement in a harbor or bay. Usually the vertical motions are low; but when oscillations are excited by a tsunami or storm surge, they may be quite large. Variable winds, air oscillations, or surf beat also may cause oscillations. See SEICHE.

HARD DEFENSES General term applied to impermeable coastal defense structures of concrete, timber, steel, masonry, etc, which reflect a high proportion of incident wave energy.

HEAD OF RIP The part of a rip current that has widened out seaward of the breakers. See also CURRENT, RIP; CURRENT, FEEDER; and NECK (RIP).

HEADLAND (HEAD) (1) A comparatively high promontory with either a CLIFF or steep face extending out into a body of water, such as a sea or lake. An unnamed HEAD is usually called a headland. (2) The section of RIP CURRENT which has widened out seaward of the BREAKERS, also called HEAD OF RIP. (3) Seaward end of BREAKWATER or dam.

HEADWATER LEVEL The level of water in the reservoir.

HEAVE (1) The vertical rise or fall of the waves or the sea. (2) The translational movement of a craft parallel to its vertical axis. (3) The net transport of a floating body resulting from wave action.

HIGH SEAS This term, in municipal and international law, denotes the continuous body of salt water in the world that is navigable in its character and that lies outside territorial waters and maritime belts of the various countries.

HIGH TIDE, HIGH WATER (HW) The maximum elevation reached by each rising tide. See TIDE.

HIGH WATER (HW) Maximum height reached by a rising tide. The height may be solely due to the periodic tidal forces or it may have superimposed upon it the effects of prevailing meteorological conditions. Nontechnically, also called the HIGH TIDE.

HIGH WATER LINE In strictness, the intersection of the plane of mean high water with the shore. The shoreline delineated on the nautical charts of the National Ocean Service is an approximation of the high water line. For specific occurrences, the highest elevation on the shore reached during a storm or rising tide, including meteorological effects.

HIGH-WATER MARK A reference mark on a structure or natural object, indicating the maximum stage of tide or flood.

HIGH WATER OF ORDINARY SPRING TIDES (HWOST) A tidal datum appearing in some British publications, based on high water of ordinary spring tides.

HIGHER HIGH WATER (HHW) The higher of the two high waters of any tidal day. The single high water occurring daily during periods when the tide is diurnal is considered to be a higher high water.

HIGHER LOW WATER (HLW) The higher of two low waters of any tidal day.

HINDCASTING In wave prediction, the retrospective forecasting of waves using measured wind information.

HINTERLAND The region lying inland from the coast. Also the inland area served by a port.

HISTORIC EVENT ANALYSIS Extreme analysis based on hindcasting typically ten events over a period of 100 years.

HOLOCENE An epoch of the QUATERNARY period, from the end of the PLEISTOCENE, about 8,000 years ago, to the present time.

HOMOPYCNAL FLOW A condition in which the outflow jet from a river or coastal inlet and the water in the receiving basin are of the same density or are vertically mixed.

HOOK A spit or narrow cape of sand or gravel which turns landward at the outer end; a RECURVED SPIT.

HORIZONTAL CLOSURE METHOD Construction of a dam by dumping the materials from one or both banks, thus constricting the channel progressively laterally until the dam is closed. This method is also known as end dumping and point tipping.

HURRICANE An intense tropical cyclone in which winds tend to spiral inward toward a core of low pressure, with maximum surface wind velocities that equal or exceed 33.5 m/sec (75 mph or 65 knots) for several minutes or longer at some points. TROPICAL STORM is the term applied if maximum winds are less than 33.5 m/sec but greater than a whole gale (63 mph or 55 knots). Term is used in the Atlantic, Gulf, and eastern Pacific.

HURRICANE PATH or TRACK Line of movement (propagation) of the eye through an area.

HURRICANE STAGE HYDROGRAPH A continuous graph representing water level stages that would be recorded in a gage well located at a specified point of interest during the passage of a particular hurricane, assuming that effects of relatively short-period waves are eliminated from the record by damping features of the gage well. Unless specifically excluded and separately accounted for, hurricane surge hydrographs are assumed to include effects of astronomical tides, barometric pressure differences, and all other factors that influence water level stages within a properly designed gage well located at a specified point.

HURRICANE WIND PATTERN or ISOVEL PATTERN An actual or graphical representation of near-surface wind velocities covering the entire area of a hurricane at a particular instant. Isovels are lines connecting points of simultaneous equal wind velocities, usually referenced 9 meters (30 feet) above the surface, in meters per second, knots, or meters per hour; wind directions at various points are indicated by arrows or deflection angles on the isovel charts. Isovel charts are usually prepared at each hour during a hurricane, but for each half hour during critical periods.

HYDRAULIC RADIUS Quotient of the wetted cross-sectional area and the wetted perimeter.

HYDRAULICALLY EQUIVALENT GRAINS Sedimentary particles that settle at the same rate under the same conditions.

HYDROGRAPHY (1) The description and study of seas, lakes, rivers and other waters. (2) The science of locating aids and dangers to navigation. (3) The description of physical properties of the waters of a region.

HYDROGRAPHIC PRESSURE The pressure exerted by water at any given point in a body of water at rest.

HYPOPYCNAL FLOW Outflow from a river or coastal inlet in which a wedge of less dense water flows over the denser sea water.

HYPOTHETICAL HURRICANE ("HYPOHURRICANE") A representation of a hurricane, with specified characteristics, that is assumed to occur in a particular study area, following a specified path and timing sequence. TRANSPOSED--A hypohurricane based on the storm transposition principle, assumed to have wind patterns and other characteristics basically comparable to a specified hurricane of record, but transposed to follow a new path to serve as a basis for computing a hurricane surge hydrograph that would be expected at a selected point. Moderate adjustments in timing or rate of forward movement may also be made, if these are compatible with meteorological considerations and study objectives. HYPOHURRICANE BASED ON GENERALIZED PARAMETERS--Hypohurricane estimates based on various logical combinations of hurricane characteristics used in estimating hurricane surge magnitudes corresponding to a range of probabilities and potentialities. The STANDARD PROJECT HURRICANE is most commonly used for this purpose, but estimates corresponding to more severe or less severe assumptions are important in some project investigations. STANDARD PROJECT HURRICANE (SPH)--A hypothetical hurricane intended to represent the most severe combination of hurricane parameters that is reasonably characteristic of a specified region, excluding extremely rare combinations. It is further assumed that the SPH would approach a given project site from such direction, and at such rate of movement, to produce the highest HURRICANE SURGE HYDROGRAPH, considering pertinent hydraulic characteristics of the area. Based on this concept, and on extensive meteorological studies and probability analyses, a tabulation of "Standard Project Hurricane Index Characteristics" mutually agreed upon by representatives of the U. S. Weather Service and the Corps of Engineers, is available. PROBABLE MAXIMUM HURRICANE--A hypohurricane that might result from the most severe combination of hurricane parameters that is considered reasonably possible in the region involved, if the hurricane should approach the point under study along a critical path and at optimum rate of movement. This estimate is substantially more severe than the SPH criteria. DESIGN HURRICANE--A representation of a hurricane with specified characteristics that would produce HURRICANE SURGE HYDROGRAPHS and coincident wave effects at various key locations along a proposed project alignment. It governs the project design after economics and other factors have been duly considered. The design hurricane may be more or less severe than the SPH, depending on economics, risk, and local considerations.

---------- I ----------

ICE AGE A loosely-used synonym of glacial epoch, or time of extensive glacial activity; specifically of the latest period of widespread continental glaciers, the PLEISTOCENE Epoch.

ICE FRONT The floating vertical cliff forming the seaward edge of an ICE SHELF or other glacier that enters the sea.

ICE SHELF A extensive sheet of ice which is attached to the land along one side but most of which is afloat and bounded on the seaward side by a steep cliff (ICE FRONT) rising 2 tp 50+ m above sea level. Common along polar coasts (Antarctica, Greenland), and generally of great breadth and sometimes extending tens or hundreds of km seaward from the continental coastline.

IMPERMEABLE GROIN A GROIN constructed such that sand cannot pass through the structure (but sand may still move over or around it).

INCIDENT WAVE Wave moving landward.

INFRAGRAVITY WAVE Long waves with periods of 30 seconds to several minutes.

INLET (1) A short, narrow waterway connecting a bay, lagoon, or similar body of water with a large parent body of water. (2) An arm of the sea (or other body of water) that is long compared to its width and may extend a considerable distance inland. See also TIDAL INLET.

INLET GORGE Generally, the deepest region of an inlet channel.

INSHORE (ZONE) In beach terminology, the zone of variable width extending from the low water line through the breaker zone. Also SHOREFACE.

INSHORE CURRENT Any current in or landward of the breaker zone.

INSULAR SHELF The zone surrounding an island extending from the low water line to the depth (usually about 183 m; 100 fathoms) where there is a marked or rather steep descent toward the great depths.

INTERNAL WAVES Waves that occur within a fluid whose density changes with depth, either abruptly at a sharp surface of discontinuity (an interface), or gradually. Their amplitude is greatest at the density discontinuity or, in the case of a gradual density change, somewhere in the interior of the fluid and not at the free upper surface where the surface waves have their maximum amplitude.

INTERTIDAL The zone between the high and low water tides.

IRREGULAR WAVES Waves with random wave periods (and in practice, also heights), which are typical for natural wind-induced waves.

IRROTATIONAL WAVE A wave with fluid particles that do not revolve around an axis through their centers, although the particles themselves may travel in circular or nearly circular orbits. Irrotational waves may be PROGRESSIVE, STANDING, OSCILLATORY, or TRANSLATORY. For example, the Airy, Stokes, cnoidal, and solitary wave theories describe irrotational waves. Compare TROCHOIDAL WAVE.

ISOBATH A contour line connecting points of equal water depths on a chart.

ISOPACHYTE Line connecting points on the seabed with an equal depth of sediment.

ISOVEL PATTERN See HURRICANE WIND PATTERN.

ISTHMUS A narrow strip of land, bordered on both sides by water, that connects two larger bodies of land.

--------- J ----------

JET To place (a pile, slab, or pipe) in the ground by means of a jet of water acting at the lower end.

JETTY On open seacoasts, a structure extending into a body of water, which is designed to prevent shoaling of a channel by littoral materials and to direct and confine the stream or tidal flow. Jetties are built at the mouths of rivers or tidal inlets to help deepen and stabilize a channel.

JOINT PROBABILITY The probability of two (or more) things occurring together.

JOINT PROBABILITY DENSITY Function specifying the joint distribution of two (or more) variables.

JOINT RETURN PERIOD Average period of time between occurrences of a given joint probability event.

JONSWAP SPECTRUM Wave spectrum typical of growing deep water waves developed from field experiments and measurements of waves and wave spectra in the Joint North Sea Wave Project

---------- K ----------

KATABATIC WIND Wind caused by cold air flowing down slopes due to gravitational acceleration.

KEY A cay, esp. one of the low, insular banks of sand, coral, and limestone off the southern coast of Florida.

KINEMATIC VISCOSITY The dynamic viscosity divided by the fluid density.

KINETIC ENERGY (OF WAVES) In a progressive oscillatory wave, a summation of the energy of motion of the particles within the wave.

KNOLL A submerged elevation of rounded shape rising less than 1000 meters from the ocean floor and of limited extent across the summit. Compare SEAMOUNT.

KNOT The unit of speed used in navigation equal to 1 nautical mile (6,076.115 ft or 1,852 m) per hour.

---------- L ----------

LAGGING OF TIDE The periodic retardation in the time of occurrence of high and low water due to changes in the relative positions of the moon and sun.

LAGOON A shallow body of water, like a pond or sound, partly or completely separated from the sea by a barrier island or REEF. Sometimes connected to the sea via an INLET.

LAMINAR FLOW Slow, smooth flow, with each drop of water traveling a smooth path parallel to its neighboring drops. Laminar flow is characteristic of low velocities, and particles of sediment in the flow zones are moved by rolling or SALTATION.

LAND BREEZE A light wind blowing from the land to the sea, caused by unequal cooling of land and water masses.

LAND-SEA BREEZE The combination of a land breeze and a sea breeze as a diurnal phenomenon.

LANDLOCKED Enclosed, or nearly enclosed, by land--thus protected from the sea, as a bay or a harbor.

LANDMARK A conspicuous object, natural or artificial, located near or on land, which aids in fixing the position of an observer.

LEAD LINE A line, wire, or cord used in sounding (to obtain water depth). It is weighted at one end with a plummet (sounding lead). Also SOUNDING LINE.

LEDGE A rocky formation forming a ridge or REEF, especially one underwater or near shore.

LEE (1) Shelter, or the part or side sheltered or turned away from the wind or waves. (2) (Chiefly nautical) the quarter or region toward which the wind blows.

LEEWARD The direction toward which the wind is blowing; the direction toward which waves are traveling.

LENGTH OF WAVE The horizontal distance between similar points on two successive waves measured perpendicularly to the crest.

LEVEE (1) A ridge or EMBANKMENT of sand and silt, built up by a stream on its flood plain along both banks of its channel. (2) A large DIKE or artificial EMBANKMENT, often having an access road along the top, which is designed as part of a system to protect land from floods.

LIGHT BREEZE A wind with velocity from 4 to 6 KNOTS.

LIMIT OF BACKRUSH (LIMIT OF BACKWASH) See BACKRUSH, BACKWASH.

LITTORAL Of or pertaining to a shore, especially of the sea.

LITTORAL CELL A reach of the coast that is isolated sedimentologically from adjacent coastal reaches and that features its own sources and sinks. Isolation is typically caused by protruding headlands, submarine canyons, inlets, and some river mouths that prevent littoral sediment from one cell to pass into the next. Cells may range in size from a multi-hundred meter POCKET BEACH in a rocky coast to a BARRIER ISLAND many tens of kilometers long.

LITTORAL CURRENT See CURRENT, LITTORAL.

LITTORAL DEPOSITS Deposits of littoral drift.

LITTORAL DRIFT, LITTORAL TRANSPORT The movement of beach material in the littoral zone by waves and currents. Includes movement parallel (long shore drift) and sometimes also perpendicular (cross-shore transport) to the shore.

LITTORAL TRANSPORT RATE Rate of transport of sedimentary material parallel or perpendicular to the shore in the littoral zone. Usually expressed in cubic meters (cubic yards) per year. Commonly synonymous with LONGSHORE TRANSPORT RATE.

LITTORAL ZONE In beach terminology, an indefinite zone extending seaward from the shoreline to just beyond the breaker zone.

LOAD The quantity of sediment transported by a current. It includes the suspended load of small particles and the bedload of large particles that move along the bottom.

LONG WAVES Waves with periods above about 30 seconds; can be generated by wave groups breaking in the surf zone. See also INFRAGRAVITY WAVES.

LONGSHORE Parallel to and near the shoreline; ALONGSHORE.

LONGSHORE BAR A sand ridge or ridges, running roughly parallel to the shoreline and extending along the shore outside the trough, that may be exposed at low tide or may occur below the water level in the offshore.

LONGSHORE CURRENT See CURRENT, LONGSHORE.

LONGSHORE DRIFT Movement of (beach) sediments approximately parallel to the coastline.

LONGSHORE TRANSPORT RATE See LITTORAL TRANSPORT RATE.

LONGSHORE TROUGH An elongate DEPRESSION or series of depressions extending along the lower BEACH or in the offshore zone inside the BREAKERS.

LOOP That part of a STANDING WAVE where the vertical motion is greatest and the horizontal velocities are least. Loops (sometimes called ANTINODES) are associated with CLAPOTIS and with SEICHE action resulting from wave reflections. Compare NODE.

LOW TIDE (LOW WATER, LW) The minimum elevation reached by each falling tide. See TIDE.

LOW TIDE TERRACE A flat zone of the beach near the low water level.

LOW WATER (LW) The minimum height reached by each falling tide. Nontechnically, also called LOW TIDE.

LOW WATER DATUM An approximation to the plane of mean low water that has been adopted as a standard reference plane. See also DATUM, PLANE and CHART DATUM.

LOW WATER LINE The line where the established LOW WATER DATUM intersects the shore. The plane of reference that constitutes the LOW WATER DATUM differs in different regions.

LOW WATER OF ORDINARY SPRING TIDES (LWOST) A tidal datum appearing in some British publications, based on low water of ordinary spring tides.

LOWER HIGH WATER (LHW) The lower of the two high waters of any tidal day.

LOWER LOW WATER DATUM An approximation to the plane of MEAN LOWER LOW WATER that has been adopted as a standard reference plane for a limited area and is retained for an indefinite period regardless of the fact that it may differ slightly from a better determination of MEAN LOWER LOW WATER from a subsequent series of observations.

LOWER LOW WATER (LLW) The lower of the two low waters of any tidal day. The single low water occurring daily during periods when the tide is diurnal is considered to be a lower low water.

LUNAR DAY The time of rotation of the Earth with respect to the moon, or the interval between two successive upper transits of the moon over the meridian of a place. The mean lunar day is approximately 24.84 solar hours in length, or 1.035 times as great as the mean solar day. Also called TIDAL DAY.

LUNAR TIDE The portion of the tide that can be attributed directly to attraction to the moon.

---------- M ----------

MACH-STEM WAVE Higher-than-normal wave generated when waves strike a structure at an oblique angle.

MACRO-TIDAL Tidal range greater than 4 m.

MANAGED RETREAT The deliberate setting back (moving landward) of the existing line of sea defense in order to obtain engineering or environmental advantages - also referred to as managed landward realignment. Sometimes refers to moving roads and utilities landward in the face of shore retreat.

MANGROVE A tropical tree with interlacing prop roots, confined to low-lying brackish areas.

MARGIN, CONTINENTAL A zone separating a continent from the deep-sea bottom.

MARGINAL PROBABILITY The probability of a single variable in the context of a joint probability analysis.

MARGINAL RETURN PERIOD The return period of a single variable in the context of a joint probability analysis.

MARIGRAM A graphic record of the rise and fall of the tide. The record is in the form of a curve in which time is represented by abscissas and the height of the tide by ordinates.

MARKER, REFERENCE A mark of permanent character close to a survey station, to which it is related by an accurately measured distance and azimuth (or bearing).

MARKER, SURVEY An object placed at the site of a station to identify the surveyed location of that station.

MARSH (1) A tract of soft, wet land, usually vegetated by reeds, grasses and occasionally small shrubs. (2) Soft, wet area periodically or continuously flooded to a shallow depth, usually characterized by a particular subclass of grasses, cattails and other low plants.

MARSH, DIKED A former salt marsh which has been protected by a DIKE.

MARSH, SALT A marsh periodically flooded by salt water.

MASS TRANSPORT, SHOREWARD The movement of water due to wave motion, which carries water through the BREAKER ZONE in the direction of wave propagation. Part of the NEARSHORE CURRENT SYSTEM.

MATTRESS A blanket of brushwood or bamboo, poles, geotextile and reed lashed together to protect a shoreline, embankment or river/sea bed against erosion. Sometimes placed on the sea bed during JETTY construction to prevent stone from settling into soft bottom.

MEAN DEPTH The average DEPTH of the water area between the still water level and the SHOREFACE profile from the waterline to any chosen distance seaward.

MEAN DIAMETER, GEOMETRIC See GEOMETRIC MEAN DIAMETER.

MEAN HIGH WATER SPRINGS (MHWS) The average height of the high water occurring at the time of spring tides.

MEAN HIGH WATER (MHW) The average height of the high waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All high water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the higher high water heights are included in the average where the type of tide is diurnal. So determined, mean high water in the latter case is the same as mean higher high water.

MEAN HIGHER HIGH WATER (MHHW) The average height of the higher high waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

MEAN LOW WATER (MLW) The average height of the low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All low water heights are included in the average where the type of tide is either semidiurnal or mixed. Only lower low water heights are included in the average where the type of tide is diurnal. So determined, mean low water in the latter case is the same as mean lower low water.

MEAN LOW WATER SPRINGS The average height of low waters occurring at the time of the spring tides. It is usually derived by taking a plane depressed below the half-tide level by an amount equal to one-half the spring range of tide, necessary corrections being applied to reduce the result to a mean value. This plane is used to a considerable extent for hydrographic work outside of the United States and is the plane of reference for the Pacific approaches to the Panama Canal. Frequently abbreviated to LOW WATER SPRINGS.

MEAN LOWER LOW WATER (MLLW) The average height of the lower low waters over a 19-year period. For shorter periods of observations, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. Frequently abbreviated to LOWER LOW WATER.

MEAN RANGE OF TIDE The difference in height between MEAN HIGH WATER and MEAN LOW WATER.

MEAN RISE OF THE TIDE The height of MEAN HIGH WATER above the plane of reference or DATUM of chart.

MEAN SEA LEVEL The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings. Not necessarily equal to MEAN TIDE LEVEL.

MEAN STEEPNESS The ratio of the MEAN DEPTH to the horizontal distance over which the MEAN DEPTH was determined.

MEAN TIDE LEVEL A plane midway between MEAN HIGH WATER and MEAN LOW WATER. Not necessarily equal to MEAN SEA LEVEL. Also HALF-TIDE LEVEL.

MEAN WATER LEVEL The mean water surface level as determined by averaging the heights of the water at equal intervals of time, usually at hourly intervals. The mean water level includes all components contributing to the stillwater level, including astronomical tides, storm surge, wave setup and freshwater input. Also called the "total stillwater level".

MEAN WAVE HEIGHT The mean of all individual waves in an observation interval of approximately half an hour. In case of a Rayleigh distribution 63 percent of the significant wave height.

MEANDERING A single channel having a pattern of successive deviations in alignment which results in a more or less sinusoidal course.

MEDIAN DIAMETER The diameter which marks the division of a given sand sample into two equal parts by weight, one part containing all grains larger than that diameter and the other part containing all grains smaller.

MEGARIPPLE See SAND WAVE.

MESO-TIDAL Tidal range between 2 m and 4 m.

METEOROLOGICAL TIDES Tidal constituents having their origin in the daily or seasonal variation in weather conditions which may occur with some degree of periodicity.

MICRO-TIDAL Tidal range less than 2 m.

MID-EXTREME TIDE A plane midway between the extreme high water and the extreme LOW WATER occurring in any locality.

MIDDLE-GROUND SHOAL A shoal formed by ebb and flood tides in the middle of the channel of the LAGOON or estuary end of an inlet.

MINERAL A naturally occurring, inorganic, crystalline solid that has a definite chemical composition and possesses characteristic physical properties.

MINIMUM DURATION See DURATION, MINIMUM.

MINIMUM FETCH The least distance in which steady-state wave conditions will develop for a wind of given speed blowing a given duration of time.

MIST Water vapor suspended in the air in very small drops finer than rain, larger than fog.

MIXED CURRENT Type of tidal current characterized by a conspicuous velocity difference between the two floods or two ebbs usually occurring each tidal day.

MIXED TIDE A type of tide in which the presence of a diurnal wave is conspicuous by a large inequality in either the high or low water heights, with two high waters and two low waters usually occurring each tidal day. In strictness, all tides are mixed, but the name is usually applied without definite limits to the tide intermediate to those predominantly semidiurnal and those predominantly diurnal.

MOLE In coastal terminology, a massive land-connected, solid-fill structure of earth (generally revetted), masonry, or large stone, which may serve as a breakwater or pier.

MONOCHROMATIC WAVES A series of waves generated in a laboratory, each of which has the same length and period.

MONOLITHIC Like a single stone or block. In coastal structures, the type of construction in which the structure's component parts are bound together to act as one.

MORAINE An accumulation of earth, stones, etc., deposited by a glacier, usually in the form of a mound, ridge or other prominence on the terrain.

MORPHODYNAMICS (1) The mutual interaction and adjustment of the seafloor topography and fluid dynamics involving the motion of sediment. (2) The coupled suite of mutually interdependent hydrodynamic processes, seafloor morphologies, and sequences of change.

MORPHOLOGICALLY AVERAGED Single wave condition producing the same net longshore drift as a given proportion of the annual wave climate.

MORPHOLOGY River/estuary/lake/seabed form and its change with time.

MUD A fluid-to-plastic mixture of finely divided particles of solid material and water.

MUD FLAT A level area of fine silt and clay along a shore alternately covered or uncovered by the tide or covered by shallow water.

---------- N ----------

NATIONAL TIDAL DATUM EPOCH (NTDE) A period of 19 years adopted by the National Ocean Service as the period over which observations of tides are to be taken and reduced to average values for tidal datums.

NATURAL TRACER A component of a sediment deposit that is unique to a particular source and can be used to identify the source and transport routes to a place of deposition.

NAUTICAL MILE The length of a minute of arc, 1/21,600 of an average great circle of the Earth. Generally one minute of latitude is considered equal to one nautical mile. The accepted United States value as of 1 July 1959 is 1,852 meters (6,076.115 feet), approximately 1.15 times as long as the U.S. statute mile of 5,280 feet. Also geographical mile.

NEAP HIGH WATER See NEAP TIDE.

NEAP LOW WATER See NEAP TIDE.

NEAP RANGE See NEAP TIDE.

NEAP TIDAL CURRENT Tidal current of decreased velocity occurring semimonthly as the result of the moon being in quadrature.

NEAP TIDE Tide of decreased range occurring semimonthly as the result of the moon being in quadrature. The NEAP RANGE of the tide is the average semidiurnal range occurring at the time of neap tides and is most conveniently computed from the harmonic constants. The NEAP RANGE is typically 10 to 30 percent smaller than the mean range where the type of tide is either semidiurnal or mixed and is of no practical significance where the type of tide is DIURNAL. The average height of the high waters of the neap tide is called NEAP HIGH WATER or HIGH WATER NEAPS (MHWN), and the average height of the corresponding LOW WATER is called NEAP LOW WATER or LOW WATER NEAPS (MLWN).

NEARSHORE (1) In beach terminology an indefinite zone extending seaward from the SHORELINE well beyond the BREAKER ZONE. (2) The zone which extends from the swash zone to the position marking the start of the offshore zone, typically at water depths of the order of 20 m.

NEARSHORE CIRCULATION The ocean circulation pattern composed of the NEARSHORE CURRENTS and the COASTAL CURRENTS.

NEARSHORE CURRENT SYSTEM The current system caused primarily by wave action in and near the breaker zone, and which consists of four parts: the shoreward mass transport of water; longshore currents; seaward return flow, including rip currents; and the longshore movement of the expanding heads of rip currents. See also NEARSHORE CIRCULATION.

NECK (1) The narrow strip of land which connects a peninsula with the mainland, or connects two ridges. (2) The narrow band (rip) of water flowing seaward through the surf. See also RIP CURRENT.

NESS Roughly triangular promontory of land jutting into the sea, often consisting of mobile material, i.e. a beach form.

NETWORK A set consisting of: (a) stations for which geometric relationships have been determined and which are so related that removal of one station from the set will affect the relationships (distances, directions, coordinates, etc.) between the other stations; and (b) lines connecting the stations to show this interdependence.

NIP The cut made by waves in a shoreline of emergence.

NODAL ZONE An area in which the predominant direction of the LONGSHORE TRANSPORT changes.

NODE That part of a STANDING WAVE where the vertical motion is least and the horizontal velocities are greatest. Nodes are associated with CLAPOTIS and with SEICHE action resulting from wave reflections. Compare LOOP.

NOURISHMENTT he process of replenishing a beach. It may occur naturally by longshore transport, or be brought about artificially by the deposition of dredged materials or of materials trucked in from upland sites.

NUMERICAL MODELING Refers to analysis of coastal processes using computational models.

---------- O ----------

OCEANOGRAPHY The study of the sea, embracing and indicating all knowledge pertaining to the sea's physical boundaries, the chemistry and physics of seawater, marine biology, and marine geology.

OFFSHORE (1) In beach terminology, the comparatively flat zone of variable width, extending from the SHOREFACE to the edge of the CONTINENTAL SHELF. It is continually submerged. (2) The direction seaward from the shore. (3) The zone beyond the nearshore zone where sediment motion induced by waves alone effectively ceases and where the influence of the sea bed on wave action is small in comparison with the effect of wind. (4) The breaker zone directly seaward of the low tide line.

OFFSHORE BARRIER See BARRIER BEACH.

OFFSHORE BREAKWATER A BREAKWATER built towards the seaward limit of the littoral zone, parallel (or nearly parallel) to the shore.

OFFSHORE CURRENT (1) Any current in the offshore zone. (2) Any current flowing away from shore.

OFFSHORE WIND A wind blowing seaward from the land in the coastal area.

ONSHORE A direction landward from the sea.

ONSHORE WIND A wind blowing landward from the sea in the coastal area.

OPPOSING WIND In wave forecasting, a wind blowing in a direction opposite to the ocean-wave advance; generally, a headwind.

ORBIT In water waves, the path of a water particle affected by the wave motion. In deepwater waves the orbit is nearly circular, and in shallow-water waves the orbit is nearly elliptical. In general, the orbits are slightly open in the direction of wave motion, giving rise to MASS TRANSPORT.

ORBITAL CURRENT The flow of water accompanying the orbital movement of the water particles in a wave. Not to be confused with wave-generated LITTORAL CURRENTS.

ORDINARY HIGH-WATER MARK (OHWM) That mark that will be found by examining the bed and banks and ascertaining where the presence and action of waters are so common and usual, and so long continued in all ordinary years, as to mark upon the soil a character distinct from that of the abutting upland, in respect to vegetation as that condition exists on June 1, 1971, as it may naturally change thereafter, or as it may change thereafter in accordance with permits issued by a local government. Also defined as MEAN HIGH WATER LINE.

ORDINARY TIDE This expression is not used in a technical sense by the U.S. Coast and Geodetic Survey, but the word "ordinary" when applied to tides, may be taken as the equivalent of the word "mean". Thus "ordinary HIGH WATER LINE" may be assumed to be the same as "mean high water line".

ORTHOGONAL On a wave-refraction diagram, a line drawn perpendicularly to the wave crests. Also called WAVE RAY.

OSCILLATION (1) A periodic motion backward and forward. (2) Vibration or variance above and below a mean value.

OSCILLATORY WAVE A wave in which each individual particle oscillates about a point with little or no permanent change in mean position. The term is commonly applied to progressive oscillatory waves in which only the form advances, the individual particles moving in closed or nearly closed orbits. Compare WAVE OF TRANSLATION. See also ORBIT.

OUTCROP A surface exposure of bare rock, not covered by soil or vegetation.

OUTFALL A structure extending into a body of water for the purpose of discharging sewage, storm runoff, or cooling water.

OUTFLANKING EROSION behind or around the land-based end of a GROIN, JETTY, or BREAKWATER or the terminus of a BULKHEAD, REVETMENT, or SEAWALL, usually causing failure of the structure or its function

OVERSPLASH The water that splashes over the top of a BREAKWATER, SEAWALL, etc.

OVERTOPPING Passing of water over the top of a structure as a result of wave runup or surge action.

OVERWASH (1) The part of the UPRUSH that runs over the crest of a BERM or structure and does not flow directly back to the ocean or lake. (2) The effect of waves overtopping a COASTAL DEFENSE, often carrying sediment landwards which is then lost to the beach system.

---------- P ----------

PARAPET A low wall built along the edge of a structure such as a SEAWALL or QUAY.

PARTICLE VELOCITY The velocity induced by wave motion with which a specific water particle moves within a wave.

PATCH REEF A moundlike or flat-topped organic REEF, generally less than 1 km across, frequently forming part of a larger reef complex.

PASS In hydrographic usage, a navigable channel through a bar, REEF, or shoal, or between closely adjacent islands. On the Gulf coast, inlets are often known as passes (e.g., Sabine Pass).

PEAK PERIOD The wave period determined by the inverse of the frequency at which the wave energy spectrum reaches its maximum.

PEBBLES Beach material usually well-rounded and between about 4 mm to 64 mm diameter. See SOIL CLASSIFICATION.

PENINSULA An elongated body of land nearly surrounded by water and connected to a larger body of land by a neck or isthmus.

PERCHED BEACH A beach or fillet of sand retained above the otherwise normal profile level by a submerged dike.

PERCOLATION The process by which water flows through the interstices of a sediment. Specifically, in wave phenomena, the process by which wave action forces water through the interstices of the bottom sediment and which tends to reduce wave heights.

PERIGEAN RANGE The average semidiurnal range occurring at the time of the PERIGEAN TIDES and most conveniently computed from the harmonic constants. It is larger than the mean range where the type of tide is either semidiurnal or mixed and is of no practical significance where the type of tide is diurnal.

PERIGEAN TIDAL CURRENTS Tidal currents of increased velocity occurring monthly as the result of the moon being in perigee (i.e., at the point in its orbit nearest the Earth).

PERIGEAN TIDES Tides of increased range occurring monthly as the result of the moon being in perigee.

PERIODIC CURRENT A current caused by the tide-producing forces of the moon and the sun; a part of the same general movement of the sea that is manifested in the vertical rise and fall of the tides. See also CURRENT, FLOOD and CURRENT, EBB.

PERMANENT CURRENT A current that runs continuously, independent of the tides and temporary causes. Permanent currents include the freshwater discharge of a river and the currents that form the general circulatory systems of the oceans.

PERMEABILITY The property of bulk material (sand, crushed rock, soft rock in situ) which permit movement of water through its pores.

PERMEABLE GROIN A GROIN with openings or voids large enough to permit passage of appreciable quantities of LITTORAL DRIFT through the structure.

PETROGRAPHY The systematic description and classification of rocks.

PETROLOGY That branch of geology which treats the scientific study of rocks.

PHASE In surface wave motion, a point in the period to which the wave motion has advanced with respect to a given initial reference point.

PHASE INEQUALITY Variations in the tides or tidal currents associated with changes in the phase of the Moon in relation to the Sun.

PHASE VELOCITY Propagation velocity of an individual wave as opposed to the velocity of a wave group.

PHI GRADE SCALE A logarithmic transformation of the Wentworth grade scale for size classifications of sediment grains based on the negative logarithm to the base 2 of the particle diameter: $= -\log_2 d$. See SOIL CLASSIFICATION.

PHOTIC ZONE The zone extending downward from the ocean surface within which the light is sufficient to sustain photosynthesis. The depth of this layer varies with water clarity, time of year and cloud cover, but is about 100 m in the open ocean. It may be considered the Depth to which all light is filtered out except for about one percent and may be calculated as about two and one-half times the depth of a SECCHI DISK reading.

PHOTOGRAMMETRY The science of deducing the physical dimensions of objects from measurements on images (usually photographs) of the objects.

PHOTOMOSAIC An assemblage of photographs, each of which shows part of a region, put together in such a way that each point in the region appears once and only once in the assemblage, and scale variation is minimized.

PHREATIC LEVEL Upper surface of an unconfined aquifer (e.g. the top sand layer in a dike) at which the pressure in the groundwater is equal to atmospheric pressure.

PHYSICAL GEOLOGY A large division of Geology concerned with earth materials, changes of the surface and interior of the earth, and the forces that cause those changes.

PHYSICAL MODELING Refers to the investigation of coastal or riverine processes using a scaled model.

PIER A structure, usually of open construction, extending out into the water from the shore, to serve as a landing place, recreational facility, etc., rather than to afford coastal protection. In the Great Lakes, a term sometimes improperly applied to jetties.

PIERSON-MOSKOWITZ SPECTRUM Wave spectrum typical of fully-developed deep water waves.

PIEZOMETRIC SURFACE The level at which the hydrostatic water pressure in an aquifer will stand if it is free to seek equilibrium with the atmosphere. For artesian wells, this is above the ground surface.

PILE A long, heavy timber or section of concrete or metal that is driven or jetted into the earth or seabed to serve as a support or protection.

PILING A group of piles.

PIPING Erosion of closed flow channels (tunnels) by the passage of water through soil; flow underneath structures, carrying away particles, may endanger the stability of the structure.

PLACER DEPOSITS Mineral deposits consisting of dense, resistant and often economically valuable minerals which have been weathered from TERRIGENOUS rocks, transported to the sea and concentrated in marine sediments by wave or current action.

PLACER MINE Surface mines in which valuable mineral grains are extracted from stream bar or beach deposits.

PLAIN, COASTAL See COASTAL PLAIN.

PLANFORM The outline or shape of a body of water as determined by the still-water line.

PLATEAU A land area (usually extensive) having a relatively level surface raised sharply above adjacent land on at least one side; table land. A similar undersea feature.

PLEISTOCENE An epoch of the Quaternary Period characterized by several glacial ages.

PLUNGE POINT (1) For a plunging wave, the point at which the wave curls over and falls. (2) The final breaking point of the waves just before they rush up on the beach.

PLUNGING BREAKER See BREAKER.

POCKET BEACH A beach, usually small, in a coastal reentrant or between two littoral barriers.

POINT (1) The extreme end of a CAPE, or the outer end of any land area protruding into the water, usually less prominent than a CAPE. (2) A low profile shoreline promontory of more or less triangular shape, the top of which extends seaward.

POORLY-SORTED (POORLY-GRADED) Said of a clastic sediment or rock that consists of particles of many sizes mixed together in an unsystematic manner so that no one size class predominates.

PORE PRESSURE The interstitial pressure of water within a mass of soil or rock.

POROSITY Percentage of the total volume of a soil not occupied by solid particles but by air and water.

PORT A place where vessels may discharge or receive cargo; it may be the entire harbor including its approaches and anchorages, or only the commercial part of a harbor where the QUAYS, WHARVES, facilities for transfer of cargo, docks, and repair shops are situated.

POTENTIAL ENERGY OF WAVES In a progressive oscillatory wave, the energy resulting from the elevation or depression of the water surface from the undisturbed level.

PRISM See TIDAL PRISM.

PROBABILITY The chance that a prescribed event will occur, represented by a number (p) in the range 0 - 1. It can be estimated empirically from the relative frequency (i.e. the number of times the particular event occurs, divided by the total count of all events in the class considered).

PROBABILITY DENSITY Function specifying the distribution of a variable.

PROBABLE MAXIMUM WATER LEVEL A hypothetical water level (exclusive of wave runup from normal wind-generated waves) that might result from the most severe combination of hydrometeorological, geoseismic, and other geophysical factors and that is considered reasonably possible in the region involved, with each of these factors considered as affecting the locality in a maximum manner. This level represents the physical response of a body of water to maximum applied phenomena such as hurricanes, moving squall lines, other cyclonic meteorological events, tsunamis, and astronomical tide combined with maximum probable ambient hydrological conditions such as wave setup, rainfall, runoff, and river flow. It is a water level with virtually no risk of being exceeded.

PRODELTA The part of a DELTA that is below the effective depth of wave erosion, lying beyond the delta front and sloping down into the basin into which the delta is advancing.

PROFILE, BEACH The intersection of the ground surface with a vertical plane; may extend from the behind the DUNE line or the top of a bluff to well seaward of the breaker zone.

PROGRESSION (of a beach) See ADVANCE.

PROGRESSIVE WAVE A wave that moves relative to a fixed coordinate system in a fluid. The direction in which it moves is termed the direction of wave propagation.

PROMONTORY A high point of land projecting into a body of water; a HEADLAND.

PROPAGATION OF WAVES The transmission of waves through water.

PROTOTYPE In laboratory usage, the full-scale structure, concept, or phenomenon used as a basis for constructing a scale model or copy.

---------- Q ----------

QUARRY RUN Waste of generally small material, in a quarry, left after selection of larger grading.

QUARRYSTONE Any stone processed from a quarry.

QUATERNARY (1) The youngest geologic period; includes the present time. (2) The latest period of time in the stratigraphic column, 0 B 2 million years, represented by local accumulations of glacial (PLEISTOCENE) and post-glacial (HOLOCENE) deposits which continue, without change of fauna, from the top of the Pliocene (Tertiary). The quaternary appears to be an artificial division of time to separate pre-human from post-human sedimentation. As thus defined, the quaternary is increasing in duration as man’s ancestry becomes better understood.

QUAY (pronounced KEY) A stretch of paved bank, or a solid artificial landing place parallel to the navigable waterway, for use in loading and unloading vessels.

QUICKSAND Loose, yielding, wet sand which offers no support to heavy objects. The upward flow of the water has a velocity that eliminates contact pressures between the sand grains and causes the sand-water mass to behave like a fluid that yields easily to pressure and tends to suck down heavy objects.

---------- R ----------

RADAR An instrument for determining the distance and direction to an object by measuring the time needed for radio signals to travel from the instrument to the object and back, and by measuring the angle through which the instrument's antenna has traveled.

RADIOACTIVE DATING (RADIOMETRIC DATING) Calculating an age in years for geologic materials by measuring the presence of a short-life radioactive element (e.g., carbon-14) or a long-life element (e.g., potassium-40/argon-40). The term applies to all methods of age determination based on nuclear decay of naturally-occurring radioactive isotopes. Carbon-14 methods are often used to determine the age of peat or wood found in BARRIER ISLANDS.

RADIUS OF MAXIMUM WINDS Distance from the eye of a hurricane, where surface and wind velocities are zero, to the place where surface windspeeds are maximum.

RAISED BEACH A wave-cut platform, with or without a covering of beach materials, which is now raised above the present sea-level.

RANDOM WAVES The laboratory simulation of irregular sea states that occur in nature.

RANGE OF TIDE The difference in height between consecutive high and low waters. The MEAN RANGE is the difference between MEAN HIGH WATER and MEAN LOW WATER. The GREAT DIURNAL RANGE or DIURNAL RANGE is the difference in height between MEAN HIGHER HIGH WATER (MHHW) and MEAN LOWER LOW WATER (MLLW). Where the type of tide is diurnal, the mean range is the same as the diurnal range.

RAY, WAVE See ORTHOGONAL.

RAYLEIGH DISTRIBUTION A model probability distribution, commonly used in analysis of waves.

REACH (1) An arm of the ocean extending into the land, e.g., an ESTUARY. (2) A straight section of restricted waterway that is uniform with respect to discharge, slope, and cross-section.

RECENT(Geological) A synonym of HOLOCENE. See also QUATERNARY.

RECESSION (1) A continuing landward movement of the shoreline. (2) A net landward movement of the shoreline over a specified time.

RECHARGE The addition of new water to an AQUIFER or to the zone of saturation.

RECTIFICATION The process of producing, from a tilted or oblique photograph, a photograph from which displacement caused by tilt has been removed.

RECURVED SPIT A spit whose outer end in turned landward by current deflection, by the opposing action of two or more currents, or by WAVE REFRACTION; a HOOK.

RED TIDE Discoloration of surface waters, most frequently in COASTAL ZONES, caused by large concentrations of microorganisms.

REEF An offshore consolidated rock hazard to navigation, with a least depth of about 20 meters (10 fathoms) or less. Often refers to coral FRINGING REEFS in tropical waters.

REEF, ATOLL See ATOLL.

REEF, BARRIER See BARRIER REEF.

REEF BREAKWATER Rubble mound of single-sized stones with a crest at or below sea level which is allowed to be (re)shaped by the waves.

REEF, FRINGING See FRINGING REEF.

REFERENCE PLANE The plane to which sounding and tidal data are referred. See DATUM PLANE.

REFERENCE POINT (1) A specified location (in plan elevation) to which measurements are referred. (2) In beach material studies, a specified point within the REFERENCE ZONE.

REFERENCE STATION A place for which tidal constants have previously been determined and which is used as a standard for the comparison of simultaneous observations at a second station. Also, a station for which independent daily predictions are given in the tide or current tables from which corresponding predictions are obtained for other stations by means of differences or factors.

REFERENCE ZONE In regard to beach measuring procedure, the part of the FORESHORE subject to wave action (between the Limit of UPRUSH and the Limit of BACKWASH) at mid-tide stage. In areas of great tidal range a more complex definition is needed.

REFLECTED WAVE That part of an incident wave that is returned seaward when a wave impinges on a steep beach, barrier, or other reflecting surface.

REFLECTION The process by which the energy of the wave is returned seaward.

REFRACTION (of water waves) (1) The process by which the direction of a wave moving in shallow water at an angle to the contours is changed: the part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours. (2) The bending of wave crests by currents.

REFRACTION COEFFICIENT The square root of the ratio of the distance between adjacent orthogonals in deep water to their distance apart in shallow water at a selected point. When multiplied by the SHOALING FACTOR and a factor for friction and percolation, this becomes the WAVE HEIGHT COEFFICIENT or the ratio of the refracted wave height at any point to the deepwater wave height. Also, the square root of the ENERGY COEFFICIENT.

REFRACTION DIAGRAM A drawing showing positions of wave crests and/or orthogonals in a given area for a specific deepwater wave period and direction.

REGULAR WAVES Waves with a single height, period, and direction.

RESERVOIR An artificial lake, basin or tank in which a large quantity of water can be stored.

RESIDUAL (WATER LEVEL) The components of water level not attributable to astronomical effects.

RESONANCE The phenomenon of amplification of a free wave or oscillation of a system by a forced wave or oscillation of exactly equal period. The forced wave may arise from an impressed force upon the system or from a boundary condition.

RETARDATION The amount of time by which corresponding tidal phases grow later day by day (about 50 minutes).

RETROGRESSION (of a beach) See RECESSION.

RETURN PERIOD Average period of time between occurrences of a given event.

REVERSING TIDAL CURRENT A tidal current that flows alternately in approximately opposite directions with a SLACK WATER at each reversal of direction. Currents of this type usually occur in rivers and straits where the direction of flow is more or less restricted to certain channels. When the movement is towards the shore, the current is said to be flooding, and when in the opposite direction it is said to be ebbing.

REVETMENT (1) A facing of stone, concrete, etc., to protect an EMBANKMENT, or shore structure, against erosion by wave action or currents. (2) A retaining wall. (3) Facing of stone, concrete, etc., built to protect a SCARP, EMBANKMENT or shore structure against erosion by waves of currents.

REYNOLDS NUMBER The dimensionless ratio of the inertial force to the viscous force in fluid motion, Re = VL/v where L is a characteristic length, v the kinematic viscosity, and V a characteristic velocity. The Reynolds number is of importance in the theory of hydrodynamic stability and the origin of turbulence.

RIA A long, narrow inlet, with depth gradually diminishing inward. Shorter and shallower than a FJORD.

RIDGE AND RUNNEL Beach topography consisting of sand bars that have welded to the shore during the recovery stage after a storm. At low tide, water ponds in the runnels and flows seaward through gaps in the ridge.

RIDGE, BEACH A nearly continuous mound of beach material that has been shaped by wave or other action. Ridges may occur singly or as a series of approximately parallel deposits.

RILL MARKS Tiny drainage channels in a beach caused by the flow seaward of water left in the sands of the upper part of the beach after the retreat of the tide or after the dying down of storm waves.

RIP A body of water made rough by waves meeting an opposing current, particularly a tidal current; often found where tidal currents are converging and sinking.

RIP CHANNEL A channel cut by seaward flow of RIP CURRENT, usually crosses a LONGSHORE BAR.

RIP CURRENT A strong surface current flowing seaward from the shore. It usually appears as a visible band of agitated water and is the return movement of water piled up on the shore by incoming waves and wind. With the seaward movement concentrated in a limited band its velocity is somewhat accentuated. A rip consists of three parts: the FEEDER CURRENTS flowing parallel to the shore inside the breakers; the NECK, where the feeder currents converge and flow through the breakers in a narrow band or "rip"; and the HEAD OF RIP, where the current widens and slackens outside the breaker line. A rip current is often miscalled a rip tide. Also called RIP SURF.

RIP SURF See RIP CURRENT.

RIP TIDE Incorrect term for RIP CURRENT.

RIPARIAN (1) Pertaining to the banks of a body of water. (2) Of, on or pertaining to the banks of a river.

RIPPLE (1) The ruffling of the surface of water; hence, a little curling wave or undulation. (2) A wave less than 0.05 meter (2 inches) long controlled to a significant degree by both surface tension and gravity. See CAPILLARY WAVE and GRAVITY WAVE.

RIPPLE MARKS Undulations produced by fluid movement over sediments. Oscillatory currents produce symmetric ripples whereas a well-defined current direction produces asymmetrical ripples. The crest line of ripples may be straight or sinuous. The characteristic features of ripples depend upon current velocity, particle size, persistence of current direction and whether the fluid is air or water. Sand DUNES may be regarded as a special kind of super-ripple.

RIPPLES (bed forms) Small bed forms with wavelengths less than 0.3 m (1 foot) and heights less than 0.03 m (0.1 foot).

RIPRAP A protective layer or facing of quarrystone, usually well graded within wide size limit, randomly placed to prevent erosion, scour, or sloughing of an embankment or bluff; also the stone so used. The quarrystone is placed in a layer at least twice the thickness of the 50 percent size, or 1.25 times the thickness of the largest size stone in the gradation.

RISK ANALYSIS Assessment of the total risk due to all possible environmental inputs and all possible mechanisms.

ROCK WEATHERING Physical and mineralogical decay processes in rock brought about by exposure to climatic conditions either at the present time or in the geological past.

ROCK (1) An aggregate of one or more minerals; or a body of undifferentiated mineral matter (e.g., obsidian). The three classes of rocks are: (a) Igneous B crystalline rocks formed from molten material. Examples are granite and basalt. (b) Sedimentary B resulting from the consolidation of loose sediment that has accumulated in layers. Examples are sandstone, shale and limestone. (c) Metamorphic B formed from preexisting rock as a result of burial, heat, and pressure. (2) A rocky mass lying at or near the surface of the water or along a jagged coastline, especially where dangerous to shipping.

ROLLER An indefinite term, sometimes considered to denote one of a series of long-crested, large waves which roll in on a shore, as after a storm.

ROTARY CURRENT, TIDAL A tidal current that flows continually with the direction of flow changing through all points of the compass during the tidal period. Rotary currents are usually found offshore where the direction of flow is not restricted by any barriers. The tendency for the rotation in direction has its origin in the deflecting force of the earth's rotation and, unless modified by local conditions, the change is clockwise in the Northern Hemisphere and counterclockwise in the Southern Hemisphere. The velocity of the current usually varies throughout the tidal cycle, passing through two maxima in approximately opposite directions and two minima with the direction of the current at approximately ninety degrees from the direction at the time of maximum velocity.

RUBBLE (1) Loose angular waterworn stones along a beach. (2) Rough, irregular fragments of broken rock.

RUBBLE-MOUND STRUCTURE A mound of random-shaped and random-placed stones protected with a cover layer of selected stones or specially shaped concrete armor units. (Armor units in a primary cover layer may be placed in an orderly manner or dumped at random.)

RUN-UP, RUN-DOWN The upper and lower levels reached by a wave on a beach or coastal structure, relative to still-water level.

RUNNEL A corrugation or trough formed in the foreshore or in the bottom just offshore by waves or tidal currents.

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S-SLOPE BREAKWATER Rubble mound with gentle slope around still-water level and steeper slopes above and below.

SALIENT Coastal formation of beach material developed by WAVE REFRACTION and diffraction and long shore drift comprising of a bulge in the coastline towards an offshore island or breakwater, but not connected to it as in the case of a TOMBOLO - see also Ness and Cusp.

SALINITY Number of grams of salt per thousand grams of sea water, usually expressed in parts per thousand (ppt).

SALINITY GRADIENT Change in salinity with expressed in parts per thousand per foot.

SALT MARSH A marsh periodically flooded by salt water (also tidal marsh; sea marsh).

SALT-WEDGE ESTUARY In this circulation type, the density-driven component dominates and two well-mixed layers are separated by a sharp HALOCLINE. The seawater entering the channel appears as a tongue or wedge.

SALTATION That method of sand movement in a fluid in which individual particles leave the bed by bounding nearly vertically and, because the motion of the fluid is not strong or turbulent enough to retain them in suspension, return to the bed at some distance downstream. The travel path of the particles is a series of hops and bounds.

SAND Sediment particles, often largely composed of quartz, with a diameter of between 0.062 mm and 2 mm, generally classified as fine, medium, coarse or very coarse. Beach sand may sometimes be composed of organic sediments such as calcareous reef debris or shell fragments.

SAND BAR (1) See BAR. (2) In a river, a ridge of sand built to or near the surface by river currents.

SAND BYPASSING See BYPASSING, SAND.

SAND DUNE A DUNE formed of sand.

SAND REEF See BAR.

SAND SPIT A narrow sand EMBANKMENT, created by an excess of deposition at its seaward terminus, with its distal end (the end away from the point of origin) terminating in open water.

SAND WAVES (1) Longshore sand waves are large-scale features that maintain form while migrating along the shore with speeds on the order of kilometers per year. (2) Large-scale asymmetrical bedforms in sandy river beds having high length to height ratios and continuous crestlines.

SCARP, BEACH An almost vertical slope along the beach caused by erosion by wave action. It may vary in height from a few cm to a meter or so, depending on wave action and the nature and composition of the beach. See also ESCARPMENT.

SCATTER DIAGRAM A two-dimensional histogram showing the joint probability density of two variables within a data sample.

SCOUR Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.

SCOUR PROTECTION Protection against erosion of the seabed in front of the toe.

SEA (1) A large body of salt water, second in rank to an ocean, more or less landlocked and generally part of, or connected with, an ocean or a larger sea. Examples: Mediterranean Sea; South China Sea. (2) Waves caused by wind at the place and time of observation. (3) State of the ocean or lake surface, in regard to waves.

SEA BREEZE A light wind blowing from the sea toward the land caused by unequal heating of land and water masses.

SEA CHANGE (1) A change wrought by the sea. (2) A marked transformation.

SEA CLIFF A cliff situated at the seaward edge of the coast.

SEA GRASS Members of marine seed plants that grow chiefly on sand or sand-mud bottom. They are most abundant in water less than 9 m deep. The common types are: Eel grass (Zostera), Turtle grass (Thallasia), and Manatee grass (Syringodium).

SEA LEVEL See MEAN SEA LEVEL.

SEA LEVEL RISE The long-term trend in MEAN SEA LEVEL.

SEA PUSS A dangerous longshore current; a rip current caused by return flow; loosely, the submerged channel or inlet through a bar caused by those currents.

SEA STATE Description of the sea surface with regard to wave action. Also called state of sea.

SEACOAST The coast adjacent to the sea or ocean.

SEAMOUNT An elevation rising more than 1000 meters above the ocean floor, and of limited extent across the summit. Compare KNOLL.

SEAS Waves caused by wind at the place and time of observation.

SEASHORE (1) (Law) All ground between the ordinary high-water and low-water mark. (2) The shore of the sea or ocean, often used in a general sense (e.g., to visit the seashore).

SEAWALL (1) A structure, often concrete or stone, built along a portion of a coast to prevent erosion and other damage by wave action. Often it retains earth against its shoreward face. (2) A structure separating land and water areas to alleviate the risk of flooding by the sea. Generally shore-parallel, although some reclamation SEAWALLS may include lengths that are normal or oblique to the (original) shoreline. A SEAWALL is typically more massive and capable of resisting greater wave forces than a BULKHEAD.

SECHHI DISK Visibility disk (white and black, 30 cm diameter) used to measure the transparency of the water.

SEDIMENT (1) Loose, fragments of rocks, minerals or organic material which are transported from their source for varying distances and deposited by air, wind, ice and water. Other sediments are precipitated from the overlying water or form chemically, in place. Sediment includes all the unconsolidated materials on the sea floor. (2) The fine grained material deposited by water or wind.

SEDIMENT CELL In the context of a strategic approach to coastal management, a length of coastline in which interruptions to the movement of sand or shingle along the beaches or near shore sea bed do not significantly affect beaches in the adjacent lengths of coastline.

SEDIMENT SINK Point or area at which beach material is irretrievably lost from a coastal cell, such as an estuary, or a deep channel in the seabed.

SEDIMENT SOURCE Point or area on a coast from which beach material is supplied, such as an eroding cliff, or river mouth.

SEDIMENT TRANSPORT The main agencies by which sedimentary materials are moved are: gravity (gravity transport); running water (rivers and streams); ice (glaciers); wind; the sea (currents and LONGSHORE DRIFT). Running water and wind are the most widespread transporting agents. In both cases, three mechanisms operate, although the particle size of the transported material involved is very different, owing to the differences in density and viscosity of air and water. The three processes are: rolling or traction, in which the particle moves along the bed but is too heavy to be lifted from it; SALTATION; and suspension, in which particles remain permanently above the bed, sustained there by the turbulent flow of the air or water.

SEDIMENT TRANSPORT PATHS The routes along which net sediment movement occurs.

SEEPAGE The movement of water through small cracks, pores, interstices, out of a body of surface of subsurface water. The loss of water by infiltration from a canal, reservoir or other body of water or from a field. It is generally expressed as flow volume per unit of time.

SEICHE (1) A standing wave oscillation of an enclosed waterbody that continues, pendulum fashion, after the cessation of the originating force, which may have been either seismic or atmospheric. (2) An oscillation of a fluid body in response to a disturbing force having the same frequency as the natural frequency of the fluid system. Tides are now considered to be seiches induced primarily by the periodic forces caused by the Sun and Moon. (3) In the Great Lakes area, any sudden rise in the water of a harbor or a lake whether or not it is oscillatory (although inaccurate in a strict sense, this usage is well established in the Great Lakes area).

SEISMIC REFLECTION The return of part of the energy of seismic waves to the earth=s surface after the waves bounce off an acoustic boundary (typically rock or material of different density).

SEISMIC REFRACTION The bending of seismic waves as they pass from one material to another.

SEISMIC SEA WAVE See TSUNAMI.

SELECTIVE SORTING A process occurring during sediment transport that tends to separate particles according to their size, density, and shape. A well-sorted distribution contains a limited range of grain sizes and usually indicates that the depositional environment contains a narrow range of sediment sizes or a narrow band of depositional energy. A poorly-sorted distribution contains a wide range of grain sizes indicating multiple sources of sediment or a wide range of deposition energies.

SELF-SUSTAINING BEACH A BEACH that has either natural or engineered sand retention and that can be stable through the continued supply of natural sediment sources, without any mechanical nourishment over a long period. Subsets include: Natural or Geomorphically Self-sustaining Beaches: self-sustaining naturally without the construction of retaining structures and with no continued mechanical sand nourishment. Anthropogenically Self-sustaining Beaches: self-sustaining by the construction of retaining structure(s) with or without initial beach fill but with no continued mechanical sand nourishment.

SEMIDIURNAL Having a period or cycle of approximately one-half of a tidal day (12.4 hours). The predominating type of tide throughout the world is semidiurnal, with two high waters and two low waters each tidal day. The tidal current is said to be semidiurnal when there are two flood and two ebb periods each day.

SENSING, REMOTE The response of an instrument or organism to stimuli from a distant source.

SETBACK A required open space, specified in shoreline master programs, measured horizontally upland from an perpendicular to the ordinary high-water mark.

SETUP, WAVE Superelevation of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone.

SETUP, WIND See WIND SETUP.

SHALLOW WATER (1) Commonly, water of such a depth that surface waves are noticeably affected by bottom topography. It is customary to consider water of depths less than one-half the surface wavelength as shallow water. See TRANSITIONAL ZONE and DEEP WATER. (2) More strictly, in hydrodynamics with regard to progressive gravity waves, water in which the depth is less than 1/25 the wavelength.

SHALLOW WATER WAVE A PROGRESSIVE WAVE which is in water less than 1/25 the wave length in depth.

SHEAR INSTABILITIES Instabilities of the surf zone longshore current commonly found on beaches with barred depth profiles. These instabilities are vertical motions with little surface elevation expression. Conservation of vorticity is the restoring mechanism.

SHEAR WAVES See SHEAR INSTABILITIES

SHEET EROSION The removal of a thin layer of surface material, usually topsoil, by a flowing sheet of water.

SHEET FLOW Sediment grains under high sheer stress moving as a layer that extends from the bed surface to some distance below (on the order of a few cm). Grains are transported in the direction of fluid flow.

SHEET PILE See PILE, SHEET.

SHEET, SMOOTH A sheet on which field control and hydrographic data such as soundings, depth curves, and regions surveyed with a wire drag are finally plotted before being used in making a final chart.

SHELF, CONTINENTAL See CONTINENTAL SHELF.

SHELF, INSULAR See INSULAR SHELF.

SHINGLE (1) Loosely and commonly, any beach material coarser than ordinary gravel, especially any having flat or flattish pebbles. (2) Strictly and accurately, beach material of smooth, well-rounded pebbles that are roughly the same size. The spaces between pebbles are not filled with finer materials. Shingle often gives out a musical sound when stepped on. The term is more widely used in Great Britain than in the U.S.

SHOAL (1) (noun) A detached area of any material except rock or coral. The depths over it are a danger to surface navigation. Similar continental or insular shelf features of greater depths are usually termed BANKS. (2) (verb) To become shallow gradually. (3) To cause to become shallow. (4) To proceed from a greater to a lesser depth of water.

SHOALING Decrease in water depth. The transformation of wave profile as they propagate inshore.

SHOALING COEFFICIENT The ratio of the height of a wave in water of any depth to its height in deep water with the effects of refraction, friction, and percolation eliminated. Sometimes SHOALING FACTOR or DEPTH FACTOR. See also ENERGY COEFFICIENT and REFRACTION COEFFICIENT.

SHOALING FACTOR See SHOALING COEFFICIENT.

SHORE The narrow strip of land in immediate contact with the sea, including the zone between high and low water lines. A shore of unconsolidated material is usually called a BEACH. Also used in a general sense to mean the coastal area (e.g., to live at the shore).

SHORE NORMAL A line at right-angles to the contours in the surf zone.

SHORE TERRACE A terrace made along a COAST by the action of waves and shore currents; it may become dry land by the uplifting of the shore or the lowering of the water. Also known as shore platform or wave-cut platform.

SHOREFACE The narrow zone seaward from the low tide SHORELINE, covered by water, over which the beach sands and gravels actively oscillate with changing wave conditions. See INSHORE (ZONE).

SHORELINE The intersection of a specified plane of water with the shore or beach (e.g., the high water shoreline would be the intersection of the plane of mean high water with the shore or beach). The line delineating the shoreline on National Ocean Service nautical charts and surveys approximates the mean high water line.

SHORELINE MANAGEMENT The development of strategic, long-term and sustainable Coastal defense and land-use policy within a sediment cell.

SHORT-CRESTED WAVE A wave, the crest length of which is of the same order of magnitude as the wave length. A system of short-crested waves has the appearance of hills being separated by troughs.

SIGNIFICANT WAVE A statistical term relating to the one-third highest waves of a given wave group and defined by the average of their heights and periods. The composition of the higher waves depends upon the extent to which the lower waves are considered. Experience indicates that a careful observer who attempts to establish the character of the higher waves will record values which approximately fit the definition of the significant wave.

SIGNIFICANT WAVE HEIGHT The average height of the one-third highest waves of a given wave group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, the average height of the highest one-third of a selected number of waves, this number being determined by dividing the time of record by the significant period. Also CHARACTERISTIC WAVE HEIGHT.

SIGNIFICANT WAVE PERIOD An arbitrary period generally taken as the period of the one-third highest waves within a given group. Note that the composition of the highest waves depends upon the extent to which the lower waves are considered. In wave record analysis, this is determined as the average period of the most frequently recurring of the larger well-defined waves in the record under study.

SILL (1) A submerged structure across a river to control the water level upstream. (2) The crest of a spillway.

SILT Sediment particles with a grain size between 0.004 mm and 0.062 mm, i.e. coarser than clay particles but finer than sand. See SOIL CLASSIFICATION.

SINUSOIDAL WAVE An oscillatory wave having the form of a sinusoid.

SLACK TIDE (SLACK WATER) The state of a tidal current when its velocity is near zero, especially the moment when a reversing current changes direction and its velocity is zero. Sometimes considered the intermediate period between ebb and flood currents during which the velocity of the currents is less than 0.05 meter per second (0.1 knot). See STAND OF TIDE.

SLIDE In mass wasting, movement of a descending mass along a plane approximately parallel to the slope of the surface.

SLIP A berthing space between two piers.

SLIP FACE The steep, downwind slope of a DUNE; formed from loose, cascading sand that generally keeps the slope at the ANGLE OF REPOSE (about 34 deg.).

SLOPE The degree of inclination to the horizontal. Usually expressed as a ratio, such as 1:25, indicating one unit rise in 25 units of horizontal distance; or in a decimal fraction (0.04). Also called GRADIENT.

SLOUGH A small muddy marshland or tidal waterway which usually connects other tidal areas. See BAYOU.

SLUICE A structure containing a gate to control the flow of water from one area to another.

SLUMP In mass wasting, movement along a curved surface in which the upper part moves vertically downward while the lower part moves outward.

SOFT DEFENSES Usually refers to beaches (natural or designed) but may also relate to energy-absorbing beach-control structures, including those constructed of rock, where these are used to control or redirect coastal processes rather than opposing or preventing them.

SOIL A layer of weathered, unconsolidated material on top of bed rock; in geologic usage, usually defined as containing organic matter and being capable of supporting plant growth.

SOIL CLASSIFICATION (size) An arbitrary division of a continuous scale of grain sizes such that each scale unit or grade may serve as a convenient class interval for conducting the analysis or for expressing the results of an analysis. There are many classifications used.

SOLITARY WAVE A wave consisting of a single elevation (above the original water surface), whose height is not necessarily small compared to the depth, and neither followed nor preceded by another elevation or depression of the water surfaces.

SORTING Process of selection and separation of sediment grains according to their grain size (or grain shape or specific gravity).

SORTING COEFFICIENT A coefficient used in describing the distribution of grain sizes in a sample of unconsolidated material. It is defined as $So = Q_1/Q_3$, where Q_1 is the diameter (in millimeters) which has 75 percent of the cumulative size-frequency (by weight) distribution smaller than itself and 25 percent larger than itself, and Q_3 is that diameter having 25 percent smaller and 75 percent larger than itself.

SOUND (1) (noun) a relatively long arm of the sea or ocean forming a channel between an island and a mainland or connecting two larger bodies, as a sea and the ocean, or two parts of the same body; usually wider and more extensive than a STRAIT. Example: Long Island Sound. (2) (verb) To measure the depth of the water.

SOUNDING A measured depth of water. On hydrographic charts the soundings are adjusted to a specific plane of reference (SOUNDING DATUM).

SOUNDING DATUM The plane to which soundings are referred. See also CHART DATUM.

SOUNDING LINE A line, wire, or cord used in sounding, which is weighted at one end with a plummet (sounding lead). Also LEAD LINE.

SPILLING BREAKER See BREAKER.

SPILLOVER LOBE Linguoid, bar-like feature formed by ebb tidal current flow over a low area of an ebb shield.

SPILLWAY A structure over or through a dam for discharging flood flows.

SPIT A small point of land or a narrow shoal projecting into a body of water from the shore.

SPOIL Overburden or other waste material removed in mining, dredging, and quarrying.

SPRING RANGE The average SEMIDIURNAL range occurring at the time of SPRING TIDES and most conveniently computed from the harmonic constants. It is larger than the MEAN RANGE where the type of tide is either SEMIDIURNAL or MIXED, and is of no practical significance where the type of tide is DIURNAL.

SPRING TIDAL CURRENTS Tidal currents of increased velocity occurring semimonthly as the result of the moon being new or full.

SPRING TIDE A tide that occurs at or near the time of new or full moon (SYZYGY) and which rises highest and falls lowest from the mean sea level.

SPUR-DIKE See GROIN.

STACK An isolated, pillar-like rocky island isolated from a nearby headland by wave erosion; a needle or chimney rock.

STAND OF TIDE A interval at high or low water when there is no sensible change in the height of the tide. The water level is stationary at high and low water for only an instant, but the change in level near these times is so slow that it is not usually perceptible. See SLACK TIDE.

STANDARD PROJECT HURRICANE See HYPOTHETICAL HURRICANE.

STANDING WAVE A type of wave in which the surface of the water oscillates vertically between fixed points, called nodes, without progression. The points of maximum vertical rise and fall are called antinodes or loops. At the nodes, the underlying water particles exhibit no vertical motion, but maximum horizontal motion. At the antinodes, the underlying water particles have no horizontal motion, but maximum vertical motion. They may be the result of two equal progressive wave trains traveling through each other in opposite directions. Sometimes called CLAPOTIS or STATIONARY WAVE.

STATION, CONTROL A point on the ground whose horizontal or vertical location is used as a basis for obtaining locations of other points.

STATIONARY WAVE A wave of essentially stable form which does not move with respect to a selected reference point; a fixed swelling. Sometimes called STANDING WAVE.

STEP The nearly horizontal section which more or less divides the BEACH from the SHOREFACE.

STILLWATER LEVEL (SWL or SWEL) The surface of the water resulting from astronomical tides, storm surge and freshwater inputs, but excluding wave setup contributions. In deep water this level approximates the midpoint of the wave height. In shallow water it is nearer to the trough than the crest. Also called the UNDISTURBED WATER LEVEL.

STOCHASTIC Having random variation in statistics.

STOCKPILE Sand piled on a beach foreshore to nourish downdrift beaches by natural littoral currents or forces. See FEEDER BEACH.

STONE Quarried or artificially-broken rock for use in construction, either as aggregate or cut into shaped blocks as dimension stone.

STONE, DERRICK Stone heavy enough to require handling individual pieces by mechanical means, generally weighing 900 kg (1 ton) and up.

STORM SURGE A rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress. See WIND SETUP.

STORM TIDE See STORM SURGE.

STRAIT A relatively narrow waterway between two larger bodies of water (e.g., Strait of Gibraltar). See also SOUND.

STRAND (1) The shore or beach of the ocean or a large lake. The land bordering any large body of water, especially a sea or an arm of the ocean. (2) WHARF, QUAY, or roadway along a water body, esp. in a city.

STRAND PLAIN A prograded shore built seawards by waves and currents.

STRANDFLAT A wave-cut platform; an elevated wave-cut terrace

STRANDING The running aground of a ship upon a STRAND, ROCK, or bottom so that it is fast for a time.

STRANDLINE An accumulation of debris (e.g. seaweed, driftwood and litter) cast up onto a beach, and lying along the limit of wave up rush. A shoreline above the present water level.

STRATIGRAPHY (1) The study of stratified rocks (sediments and volcanics) especially their sequence in time. (2) The character of the rocks and the correlation of beds in different localities.

STREAM (1) A course of water flowing along a bed in the Earth. (2) A current in the sea formed by wind action, water density differences, etc.; e.g. the Gulf Stream. See also CURRENT, STREAM.

STREAM CURRENT A narrow, deep and swift ocean current, such as the Gulf Stream. Opposite of DRIFT CURRENT.

STRUCTURAL GEOLOGY The branch of geology concerned with the internal structure of bed rock and the shapes, arrangement, and interrelationships of rock units.

SUBAERIAL Situated or occurring on or adjacent to the surface of the earth, usually meaning above the water surface.

SUBAERIAL BEACH That part of the beach which is uncovered by water (e.g. at low tide sometimes referred to as drying beach).

SUBAQUEOUS Existing, formed, or taking place under water; submerged.

SUB-TIDAL BEACH The part or the beach (where it exists) which extends from low water out to the approximate limit of storm erosion. The latter is typically located at a maximum water depth of 8 to 10 meters and is often identifiable on surveys by a break in the slope of the bed.

SUBCRITICAL FLOW Flow for which the Froude number is less than unity; surface disturbances can travel upstream.

SUBDUCTION ZONE Elongate region in which the sea floor slides beneath a continent or island arc.

SUBMARINE CANYON V-shaped valleys that run across the CONTINENTAL SHELF and down the CONTINENTAL SLOPE.

SUBMERGENT COAST A COAST in which formerly dry land has been recently drowned, either by land subsidence or a rise in seal level.

SUBORDINATE STATION A tide or current station at which a short series of observations has been obtained, which is to be reduced by comparison with simultaneous observations at another station having well-determined tidal or current constants.

SUBSIDENCE Sinking or downwarping of a part of the earth's surface.

SUBTIDAL Below the low-water datum; thus permanently.

SUPER-CRITICAL FLOW Flow for which the Froude number is greater than unity; surface disturbances will not travel upstream.

SURF (1) Collective term for BREAKERS. (2) The wave activity in the area between the shoreline and the outermost limit of breakers. (3) In literature, the term surf usually refers to the breaking waves on shore and on reefs when accompanied by a roaring noise caused by the larger waves breaking.

SURF BEAT Irregular oscillations of the nearshore water level with periods on the order of several minutes.

SURF ZONE The zone of wave action extending from the water line (which varies with tide, surge, set-up, etc.) out to the most seaward point of the zone (breaker zone) at which waves approaching the coastline commence breaking, typically in water depths of between 5 to 10 meters.

SURFACE GRAVITY WAVE (PROGRESSIVE) (1) this is the term which applies to the WIND WAVES and SWELL of lakes and oceans, also called SURFACE WATER WAVE, SURFACE WAVE or DEEP WATER WAVE, (2) a progressive GRAVITY WAVE in which the disturbance is confined to the upper limits of a body of water. Strictly speaking this term applies to those progressive GRAVITY WAVES whose celerity depends only upon the wave length.

SURFACE WATER WAVE See SURFACE GRAVITY WAVE (PROGRESSIVE).

SURGE (1) The name applied to wave motion with a period intermediate between that of the ordinary wind wave and that of the tide, say from 2 to 60 min. It is low height, usually less than 0.9 m (3 ft). See also SEICHE. (2) In fluid flow, long interval variations in velocity and pressure, not necessarily periodic, perhaps even transient in nature. (3) see STORM SURGE.

SURGING BREAKER See BREAKER.

SURVEY, CONTROL A survey that provides coordinates (horizontal or vertical) of points to which supplementary surveys are adjusted.

SURVEY, HYDROGRAPHIC A survey that has as its principal purpose the determination of geometric and dynamic characteristics of bodies of water.

SURVEY, PHOTOGRAMMETRIC A survey in which monuments are placed at points that have been determined photogrammetrically.

SURVEY, TOPOGRAPHIC A survey which has, for its major purpose, the determination of the configuration (relief) of the surface of the land and the location of natural and artificial objects thereon.

SUSPENDED LOAD (1) The material moving in suspension in a fluid, kept up by the upward components of the turbulent currents or by colloidal suspension. (2) The material collected in or computed from samples collected with a SUSPENDED LOAD SAMPLER. Where it is necessary to distinguish between the two meanings given above, the first one may be called the "true suspended load."

SUSPENDED LOAD SAMPLER A sampler which attempts to secure a sample of the water with its sediment load without separating the sediment from the water.

SUSTAINABLE BEACH A beach area that is now and will continue to receive sufficient sediment input over a long period (years or decades) to remain stable. Such sediment input can be through either natural supplies of sediment or various forms of mechanical beach nourishment (placement by hydraulic dredge, land haul of material, nearshore deposition, etc.)

SWALE The depression between two beach ridges.

SWASH The rush of water up onto the beach face following the breaking of a wave. Also UPRUSH, RUNUP.

SWASH BARS Low broad sandy bars formed by sediment in the surf and swash zones, separated by linear depressions, or RUNNELS, running parallel to the shore. Sand bodies that form and migrate across ebb-tidal shoals because of currents generated by breaking waves.

SWASH CHANNEL (1) On the open shore, a channel cut by flowing water in its return to the present body (e.g., a rip channel). (2) A secondary channel passing through or shoreward of an inlet or river bar.

SWASH MARK The thin wavy line of fine sand, mica scales, bits of seaweed, etc., left by the uprush when it recedes from its upward limit of movement on the beach face.

SWASH PLATFORM Sand sheet located between the main ebb channel of a coastal inlet and an adjacent barrier island.

SWASH ZONE The zone of wave action on the beach, which moves as water levels vary, extending from the limit of run-down to the limit of run-up.

SWELL Wind-generated waves that have traveled out of their generating area. Swell characteristically exhibits a more regular and longer period and has flatter crests than waves within their fetch (SEAS).

SYNOPTIC CHART A chart showing the distribution of meteorological conditions over a given area at a given time. Popularly called a weather map.

SYZYGY The two points in the Moon's orbit when the Moon is in conjunction or opposition to the Sun relative to the Earth; time of new or full Moon in the cycle of phases.

---------- T ----------

T-GROIN A GROIN built in the shape of a letter T with the trunk section connected to land.

TECTONIC FORCES Forces generated from within the earth that result in uplift, movement, or deformation of part of the earth’s crust.

TECTONICS The study of the major structural features of the Earth’s crust or the broad structure of a region.

TERMINAL GROIN A GROIN, often at the end of a barrier spit, intended to prevent sediment passage into the channel beyond.

TERRACE A horizontal or nearly horizontal natural or artificial topographic feature interrupting a steeper slope, sometimes occurring in a series.

TERRIGENOUS SEDIMENTS Literally land-formed sediment that has found its way to the sea floor. The term is applied (a) to sediments formed and deposited on land (e.g., soils, sand DUNES), and (b) to material derived from the land when mixed in with purely marine material (e.g., sand or clay in a shelly limestone).

THALWEG In hydraulics, the line joining the deepest points of an inlet or stream channel.

THRESHOLD OF MOTION The point at which the forces imposed on a sediment particle overcome its inertia and it starts to move.

THRESHOLD VELOCITY The maximum orbital velocity at which the sediment on the BED begins to move as waves approach shallow water.

TIDAL CREEK A creek draining back-barrier areas with a current generated by the rise and fall of the tide.

TIDAL CURRENT See CURRENT, TIDAL.

TIDAL DATUM See CHART DATUM and DATUM PLANE.

TIDAL DAY The time of the rotation of the Earth with respect to the Moon, or the interval between two successive upper transits of the Moon over the meridian of a place, approximately 24.84 solar hours (24 hours and 50 minutes) or 1.035 times the mean solar day. Also called lunar day.

TIDAL DELTA See DELTA.

TIDAL FLATS (1) Marshy or muddy areas covered and uncovered by the rise and fall of the tide. A TIDAL MARSH. (2) Marshy or muddy areas of the seabed which are covered and uncovered by the rise and fall of tidal water.

TIDAL INLET (1) A natural inlet maintained by tidal flow. (2) Loosely, any inlet in which the tide ebbs and flows. Also TIDAL OUTLET.

TIDAL MARSH Same as TIDAL FLATS.

TIDAL PERIOD The interval of time between two consecutive, like phases of the tide.

TIDAL POOL A pool of water remaining on a beach or reef after recession of the tide.

TIDAL PRISM (1) The total amount of water that flows into a harbor or out again with movement of the tide, excluding any fresh water flow. (2) The volume of water present between MEAN LOW and MEAN HIGH TIDE.

TIDAL RANGE The difference in height between consecutive high and low (or higher high and lower low) waters.

TIDAL RISE The height of tide as referred to the datum of a chart.

TIDAL RIVER That part of a river where the water level is influenced by the tide.

TIDAL SHOALS Shoals that accumulate near inlets due to the transport of sediments by tidal currents associated with the inlet.

TIDAL STAND An interval at high or low water when there is no observable change in the height of the tide. The water level is stationary at high and Low water for only an instant, but the change in level near these times is so slow that it is not usually perceptible.

TIDAL WAVE (1) The wave motion of the tides. (2) In popular usage, any unusually high and destructive water level along a shore. It usually refers to STORM SURGE or TSUNAMI.

TIDALLY DRIVEN CIRCULATION The movement of fresh water and seawater that are mixed by the sloshing back and forth of the ESTUARY in response to ocean tides.

TIDE The periodic rising and falling of the water that results from gravitational attraction of the Moon and Sun and other astronomical bodies acting upon the rotating Earth. Although the accompanying horizontal movement of the water resulting from the same cause is also sometimes called the tide, it is preferable to designate the latter as TIDAL CURRENT, reserving the name TIDE for the vertical movement.

TIDE, DAILY RETARDATION OF The amount of time by which corresponding tides grow later day by day (about 50 minutes). Also LAGGING.

TIDE, DIURNAL A tide with one high water and one low water in a day.

TIDE, EBB See EBB TIDE.

TIDE, FLOOD See FLOOD TIDE.

TIDE, MIXED See MIXED TIDE.

TIDE, NEAP See NEAP TIDE.

TIDE, SEMIDIURNAL See SEMIDIURNAL TIDE.

TIDE, SLACK See SLACK TIDE.

TIDE, SPRING See SPRING TIDE.

TIDE STAFF A tide gage consisting of a vertical graduated staff from which the height of the tide can be read directly. It is called a fixed staff when it is secured in place so that it cannot be easily removed. A portable staff is designed for removal from the water when not in use.

TIDE STATION A place at which tide observations are being taken. It is called a primary tide station when continuous observations are to be taken over a number of years to obtain basic tidal data for the locality. A secondary tide station is one operated over a short period of time to obtain data for a specific purpose.

TIDE, STORM See STORM SURGE.

TIDE TABLES Tables which give daily predictions of the times and heights of the tide. These predictions are usually supplemented by tidal differences and constants by means of which additional predictions can be obtained for numerous other places.

TIDE, WIND See WIND TIDE.

TIDES, RIP See RIP.

TOE Lowest part of sea- and portside BREAKWATER slope, generally forming the transition to the seabed.

TOMBOLO A bar or spit that connects or "ties" an island to the mainland or to another island. See CUSPATE SPIT. Also applied to sand accumulation between land and a DETACHED BREAKWATER.

TONGUE A long narrow strip of land, projecting into a body of water.

TOPOGRAPHIC MAP A map on which elevations are shown by means of contour lines.

TOPOGRAPHY The configuration of a surface, including its relief and the positions of its streams, roads, building, etc.

TOTAL STILLWATER LEVEL See MEAN WATER LEVEL.

TRAINING WALL A wall or jetty to direct current flow.

TRANSGRESSION, MARINE The invasion of a large area of land by the sea in a relatively short space of time (geologically speaking). Although the observable result of a marine transgression may suggest an almost instantaneous process, it is probable that the time taken is in reality is thousands or millions of years. The plane of marine transgression is a plane of UNCONFORMITY.

TRANSITIONAL ZONE (TRANSITIONAL WATER) In regard to progressive gravity waves, water whose depth is less than 2 but more than 1/25 the wavelength. Often called shallow water.

TRANSLATORY WAVE See WAVE OF TRANSLATION.

TRANSPOSED HURRICANE See HYPOTHETICAL HURRICANE.

TRANSVERSE BAR A bar which extends approximately right angles to shorelines.

TRANSVERSE WAVE Waves that propagate along a sailing line of a vessel.

TRENCH A long narrow submarine depression with relatively steep sides.

TROCHOIDAL WAVE A theoretical, progressive oscillatory wave first proposed by Gerstner in 1802 to describe the surface profile and particle orbits of finite amplitude, nonsinusoidal waves. The wave form is that of a prolate cycloid or trochoid, and the fluid particle motion is rotational as opposed to the usual irrotational particle motion for waves generated by normal forces. Compare IRROTATIONAL WAVE.

TROPICAL CYCLONE See HURRICANE.

TROPICAL STORM A tropical cyclone with maximum winds less than 34 m/sec (75 mile per hour). Compare with HURRICANE or TYPHOON (winds greater than 34 m/sec).

TROUGH A long and broad submarine DEPRESSION with gently sloping sides.

TROUGH OF WAVE The lowest part of a waveform between successive crests. Also, that part of a wave below still-water level.

TRUNCATED LANDFORM A landform cut off, especially by EROSION, and forming a steep side or CLIFF.

TSUNAMI A long-period wave caused by an underwater disturbance such as a volcanic eruption or earthquake. Also SEISMIC SEA WAVE. Commonly miscalled "tidal wave."

TURBIDITY (1) A condition of a liquid due to fine visible material in suspension, which may not be of sufficient size to be seen as individual particles by the naked eye but which prevents the passage of light through the liquid. (2) A measure of fine suspended matter in liquids.

TURBIDITY CURRENT A flowing mass of sediment-laden water that is heavier than clear water and therefore flows downslope along the bottom of the sea or a lake.

TURBULENT FLOW Any flow which is not LAMINAR, i.e., the stream lines of the fluid, instead of remaining parallel, become confused and intermingled.

TYPHOON See HURRICANE. The term typhoon is applied to tropical cyclones in the western Pacific Ocean.

---------- U ----------

UNCONFORMITY A surface that represents a break in the geologic record, with the rock unit immediately above it being considerably younger than the rock beneath. There are three major aspects to consider: (1) Time. An unconformity develops during a period of time in which no sediment is deposited. This concept equates deposition and time, and an unconformity represents unrecorded time. (2) Deposition. Any interruption of deposition, whether large or small in extent, is an unconformity. This aspect of unconformity pre-supposes a standard scale of deposition which is complete. Major breaks in sedimentation can usually be demonstrated easily, but minor breaks may go unrecorded until highly detailed investigations are made. (3) Structure. Structurally, unconformity may be regarded as planar structures separating older rocks below from younger rocks above, representing the break as defined in (1) and (2) above. A plane of unconformity may be a surface of weathering, Erosion or denudation, or a surface of non-deposition, or possibly some combination of these factors. It may be parallel to the upper strata, make an angle with the upper strata, or be irregular. Subsequent Earth movements may have folded or faulted it.

UNCONSOLIDATED In referring to sediment grains, loose, separate, or unattached to one another.

UNDERCUTTING Erosion of material at the foot of a Cliff or bank, e.g., a sea cliff, or river bank on the outside of a meander. Ultimately, the overhang collapses, and the process is repeated.

UNDERTOW (1) A current below water surface flowing seaward; the receding water below the surface from waves breaking on a shelving beach. (2) Actually undertow is largely mythical. As the BACKWASH of each wave flows down the BEACH, a current is formed which flows seaward. However, it is a periodic phenomenon. The most common phenomena expressed as undertow are actually RIP CURRENTS.

UNDERWATER GRADIENT The slope of the sea bottom. See SLOPE.

UNDEVELOPED COASTAL BARRIER A depositional geologic feature that is subject to wave, tidal, and wind energies, and protects landward aquatic habitats from direct wave attack, and all associated aquatic habitats, including adjacent wetlands, marshes, estuaries, inlets, and Nearshore waters, but only if there are few manmade structures and human activities do not significantly impede geomorphic and ecological processes.

UNDISTURBED WATER LEVEL Same as STILL WATER LEVEL.

UNDULATION A continuously propagated motion to and fro, in any fluid or elastic medium, with no permanent translation of the particles themselves.

UPCOAST In United States usage, the coastal direction generally trending toward the north.

UPDRIFT The direction opposite that of the predominant movement of littoral materials.

UPLAND Dry land area above and landward of the ORDINARY HIGH-WATER MARK (OHWM). Often used as a general term to mean high land far from the COAST and in the interior of the country.

UPLIFT The upward water pressure on the base of a structure or pavement.

UPRUSH The rush of water up the FORESHORE following the breaking of a wave, also called SWASH or RUNUP.

UPSTREAM Along coasts with obliquely approaching waves there is a longshore (wave-driven) current. For this current one can define an upstream and a DOWNSTREAM direction. For example, on a beach with an orientation west-east with the sea to the north, the waves come from NW. Then the current flows from West to East. Here, upstream is West of the observer, and East is DOWNSTREAM of the observer.

UPWELLING The process by which water rises from a deeper to a shallower depth, usually as a result of offshore surface water flow. It is most prominent where persistent wind blows parallel to a coastline so that the resultant Ekman transport moves surface water away from the coast.

---------- V ----------

VALLEY An elongated depression, usually with an outlet, between BLUFFS or between ranges of hills or mountains.

VALLEY, SEA A submarine depression of broad valley form without the steep side slopes which characterize a canyon.

VALLEY, SUBMARINE A prolongation of a land valley into or across a continental or insular shelf, which generally gives evidence of having been formed by stream erosion.

VELOCITY OF WAVES The speed at which an individual wave advances. See WAVE CELERITY.

VELOCITY PROFILE The velocity gradient within the BOTTOM BOUNDARY LAYER, displayed as a graph of height above the bed against the velocity of the flow.

VISCOSITY (or internal friction) That molecular property of a fluid that enables it to support tangential stresses for a finite time and thus to resist deformation. Resistance to flow.

---------- W ----------

WASH LOAD Part of the suspended load with particle sizes smaller than found in the bed; it is in near-permanent suspension and transported without deposition; the amount of wash load transported through a reach does not depend on the transport capacity of the flow; the load is expressed in mass or volume per unit of time.

WASHOVER Sediment deposited inland of a beach by overwash processes.

WATER DEPTH Distance between the seabed and the still water level.

WATER LEVEL Elevation of still water level relative to some datum.

WATERLINE A juncture of land and sea. This line migrates, changing with the tide or other fluctuation in the water level. Where waves are present on the beach, this line is also known as the limit of backrush (approximately, the intersection of the land with the still-water level.)

WAVE A ridge, deformation, or undulation of the surface of a liquid.

WAVE AGE The ratio of wave speed to windspeed.

WAVE, CAPILLARY See CAPILLARY WAVE.

WAVE CELERITY The speed of wave propagation.

WAVE CLIMATE The seasonal and annual distribution of wave height, period and direction.

WAVE CLIMATE ATLAS Series of maps showing the variability of wave conditions over a long coastline.

WAVE CREST See CREST OF WAVE.

WAVE CREST LENGTH See CREST LENGTH, WAVE.

WAVE, CYCLOIDAL See CYCLOIDAL WAVE.

WAVE DECAY See DECAY OF WAVES.

WAVE DIRECTION The direction from which a wave approaches.

WAVE DIRECTIONAL SPECTRUM Distribution of wave energy as a function of wave frequency and direction.

WAVE FORECASTING The theoretical determination of future wave characteristics, usually from observed or predicted meteorological phenomena.

WAVE FREQUENCY The inverse of wave period.

WAVE FREQUENCY SPECTRUM Distribution of wave energy as a function of frequency.

WAVE, GRAVITY See GRAVITY WAVE.

WAVE GROUP A series of waves in which the wave direction, wavelength, and wave height vary only slightly. See also GROUP VELOCITY.

WAVE HEIGHT The vertical distance between a crest and the preceding trough. See also SIGNIFICANT WAVE HEIGHT.

WAVE HEIGHT COEFFICIENT The ratio of the wave height at a selected point to the deepwater wave height. The REFRACTION COEFFICIENT multiplied by the shoaling factor.

WAVE HINDCASTING See HINDCASTING, WAVE.

WAVE, INFRAGRAVITY See INFRAGRAVITY WAVE.

WAVE, IRROTATIONAL See IRROTATIONAL WAVE.

WAVE, MONOCHROMATIC See MONOCHROMATIC WAVES.

WAVE OF TRANSLATION A wave in which the water particles are permanently displaced to a significant degree in the direction of wave travel. Distinguished from an OSCILLATORY WAVE.

WAVE, OSCILLATORY See OSCILLATORY WAVE.

WAVE PEAK FREQUENCY The inverse of wave peak period.

WAVE PERIOD The time for a wave crest to traverse a distance equal to one wavelength. The time for two successive wave crests to pass a fixed point. See also SIGNIFICANT WAVE PERIOD.

WAVE, PROGRESSIVE See PROGRESSIVE WAVE.

WAVE PROPAGATION The transmission of waves through water.

WAVE RAY See ORTHOGONAL.

WAVE, REFLECTED That part of an incident wave that is returned seaward when a wave impinges on a steep beach, barrier, or other reflecting surface.

WAVE REFRACTION See REFRACTION (of water waves).

WAVE ROSE Diagram showing the long-term distribution of wave height and direction.

WAVE SETDOWN Drop in water level outside of the breaker zone to conserve momentum as wave particle velocities and pressures change prior to wave breaking.

WAVE SETUP See SETUP, WAVE.

WAVE, SINUSOIDAL An oscillatory wave having the form of a sinusoid.

WAVE, SOLITARY See SOLITARY WAVE.

WAVE SPECTRUM In ocean wave studies, a graph, table, or mathematical equation showing the distribution of wave energy as a function of wave frequency. The spectrum may be based on observations or theoretical considerations. Several forms of graphical display are widely used.

WAVE, STANDING See STANDING WAVE.

WAVE STEEPNESS The ratio or wave height to wavelength also known as sea steepness.

WAVE TRAIN A series of waves from the same direction.

WAVE TRANSFORMATION Change in wave energy due to the action of physical processes.

WAVE, TROCHOIDAL See TROCHOIDAL WAVE.

WAVE TROUGH The lowest part of a wave form between successive crests. Also that part of a wave below still-water level.

WAVE VELOCITY The speed at which an individual wave advances.

WAVE, WIND See WIND WAVES.

WAVELENGTH The horizontal distance between similar points on two successive waves measured perpendicular to the crest.

WAVES, INTERNAL See INTERNAL WAVES.

WEIBULL DISTRIBUTION A model probability distribution, commonly used in wave analysis.

WEIR A low dam or wall across a stream to raise the upstream water level. Termed fixed crest weir when uncontrolled.

WEIR JETTY A jetty with a low section or weir over which littoral drift moves into a predredged deposition basin which is then dredged periodically.

WETLANDS Lands whose saturation with water is the dominant factor determining the nature of soil development and the types of plant and animal communities that live in the soil and on its surface (e.g. Mangrove forests).

WELL-SORTED Clastic sediment or rock that consists of particles all having approximately the same size. Example: sand dunes.

WHARF A structure built on the shore of a harbor, river, or canal, so that vessels may lie alongside to receive and discharge cargo and passengers.

WHITECAP On the crest of a wave, the white froth caused by wind.

WICKER FAGGOT Bundles of twigs or sticks, often willow, used in building earthworks or levees (traditional practice in Holland and China.). Alternate term: fascine

WIND CHOP See CHOP.

WIND, FOLLOWING See FOLLOWING WIND.

WIND, KATABATIC See KATABATIC WIND

WIND, OFFSHORE A wind blowing seaward from the land in a coastal area.

WIND, ONSHORE A wind blowing landward from the sea in a coastal area.

WIND, OPPOSING See OPPOSING WIND.

WIND ROSE Diagram showing the long-term distribution of windspeed and direction.

WIND SEA Wave conditions directly attributable to recent winds, as opposed to swell.

WIND SETDOWN Drop in water level below the still water level on the windward ends of enclosed bodies of water and semi- enclosed bays.

WIND SETUP On reservoirs and smaller bodies of water (1) the vertical rise in the still-water level on the leeward side of a body of water caused by wind stresses on the surface of the water; (2) the difference in still-water levels on the windward and the leeward sides of a body of water caused by wind stresses on the surface of the water. STORM SURGE (usually reserved for use on the ocean and large bodies of water).

WIND STRESS The way in which wind transfers energy to the sea surface.

WIND TIDE See WIND SETUP, STORM SURGE.

WIND WAVES (1) Waves being formed and built up by the wind. (2) Loosely, any wave generated by wind.

WINDWARD The direction from which the wind is blowing.