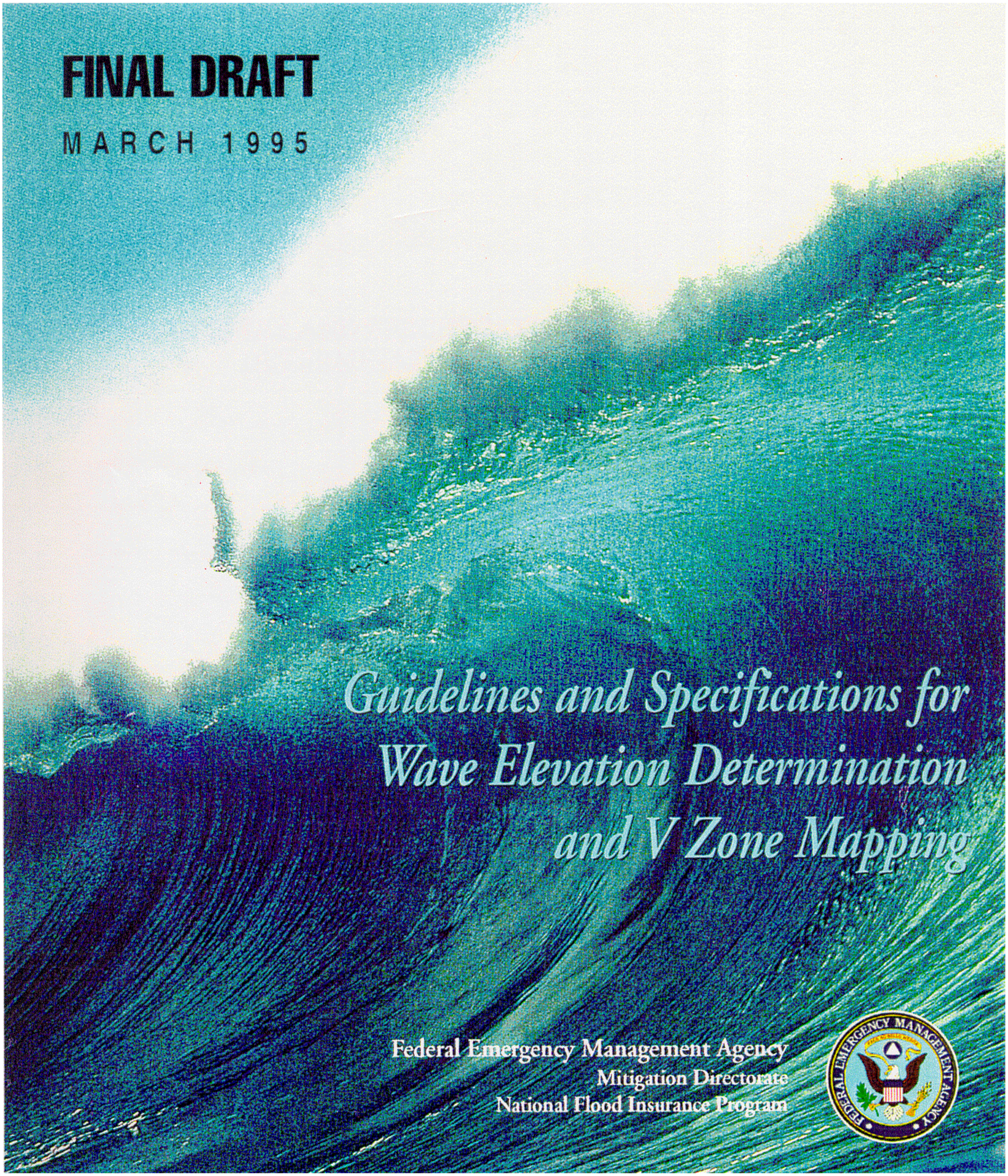

FINAL DRAFT

MARCH 1995

Guidelines and Specifications for Wave Elevation Determination and V Zone Mapping

Federal Emergency Management Agency
Mitigation Directorate
National Flood Insurance Program

GUIDELINES AND SPECIFICATIONS FOR WAVE ELEVATION DETERMINATION AND V ZONE MAPPING

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b	horizontal spacing of rigid vegetation
C	elevation of barrier crest
C_D	drag coefficient
d	water depth
d_t	water depth at toe of barrier
D	diameter of vegetation (stem or trunk)
F	freeboard of barrier above stillwater elevation
F_{cov}	fractional coverage of plants
g	gravitational acceleration
h	height of vegetation
H	wave height (crest to trough)
H'	limited valve of incident wave height
H_c	controlling wave height
H_{mo}	zero-moment wave height
L	wavelength
m	slope (tangent)
n	Manning's coefficient describing roughness
N	numerical density of vegetation
Q	discharge rate of wave overtopping
r	roughness coefficient for barrier
R	vertical wave runup dimension above stillwater level
R'	excess runup height above bluff crest
R_a	adjusted wave runup elevation
S	wave setup at shore, above stillwater level
T	wave period
X	distance inland to runup limit

Overbar denotes mean over time
Subscript o denotes deep-water value
Subscript p denotes energy peak
Subscript s denotes significant wave description
Superscript * denotes dimensionless parameter

GUIDELINES AND SPECIFICATIONS FOR WAVE ELEVATION DETERMINATION AND V ZONE MAPPING

1.0 INTRODUCTION

1.1 Authority and Purpose

The National Flood Insurance Program (NFIP) was established by the National Flood Insurance Act of 1968 and further defined by the Flood Disaster Protection Act of 1973. The 1968 Act provided for the availability of flood insurance within communities that were willing to adopt floodplain management programs to mitigate future flood losses. The act also required the identification of all floodplain areas within the United States and the establishment of flood risk zones within those areas.

A vital step toward meeting these goals is the conduct of Flood Insurance Studies (FISs) and restudies for flood-prone communities in the United States. These studies provide communities with sufficient technical information to enable them to adopt the floodplain management measures required for participation in the NFIP. FISs also provide the necessary flood risk information to establish actuarial flood insurance premiums.

Coastal communities generally have unique flood hazards because of storm surges and wave action from large open water bodies. Defining

Coastal High Hazard Areas (V zones) requires determination of wave elevations associated with the 100-year flood. The Federal Emergency Management Agency (FEMA), Mitigation Directorate, has compiled these Guidelines and Specifications for Wave Elevation Determination and V Zone Mapping (referred to herein as these Guidelines) to specify technical policies and procedures to be employed in the preparation of coastal FISs and restudies. These Guidelines are a supplement to the Flood Insurance Study Guidelines and Specifications for Study Contractors (Reference 1), and may be superseded by future instructions that reflect updated policies and procedures. The present Guidelines are not applicable to studies on Great Lakes coasts because different analysis procedures and computer models are applied there, as described in Reference 2.

Detailed guidance is provided for the determination of wave elevations associated with the 100-year coastal flood and for the identification of resultant V zones. Methodologies and models for the determination of wave heights, wave crest elevations, wave runup, and coastal erosion have been adopted and refined by FEMA. Various available documents describing the development, basis, and application of these methodologies are referenced, but not discussed in detail. These Guidelines have been compiled to provide unified instructions on the application of these methodologies to determine the coastal flooding elevations and hazards set forth in the FIS, and on the delineation of the flood elevations and hazards on the Flood Insurance Rate Map (FIRM). The FIRM provides base flood elevations (BFEs) and divides the community into flood hazard zones that are used to establish actuarial insurance rates.

1.2 Background

The mapping of V zones under the NFIP first 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: "areas subject to high velocity waters, including but not limited to hurricane wave wash." The initial technical guidance for identifying V zones was provided in General Guidelines for Identifying Coastal High Hazard Zone, Flood Insurance Study - Texas Gulf Coast Case Study, prepared by the Galveston District, Corps of Engineers (COE) (Reference 3). This 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.

The COE issued a follow-up report, Guidelines for Identifying Coastal High Hazard Zones, which maintained the basic recommendations contained in the previous report for identifying V zones in undeveloped and developed areas (Reference 4). However, this 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 have impacted developed areas along the Atlantic and Gulf coasts.

Between 1975 and 1980, FIRMs with V zones were published for approximately 270 communities along the Atlantic and Gulf coasts using the COE guidance for V zone mapping. During this period, the procedures for the determination and delineation of V zones in developed areas lacked uniformity among studies. The regulatory BFEs, at that time, for both insurance and construction purposes, were the 100-year stillwater elevations 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. It was 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.

In 1976, FEMA contracted the National Academy of Sciences (NAS) to provide recommendations about how calculations of wave height and runup should be incorporated in FISs of coastal communities to provide an estimate of the areal extent and height of stormwater inundation having specified recurrence intervals. The NAS concluded that the prediction of wave heights should be included in FISs of coastal communities and provided a methodology for the open coast and shores of embayments and estuaries on the Atlantic and Gulf coasts. The Methodology for Calculating Wave Action Effects Associated with Storm Surges included means for taking account of varying fetch lengths, barriers to wave transmission, and the regeneration of waves likely to occur over flooded land areas (Reference 5). The extent and elevation of wave runup, amount of barrier overtopping, and coastal erosion were not addressed by the NAS.

The NAS methodology was adopted by FEMA in 1979, and a Users Manual was issued in 1980 (Reference 6). The computer program Wave Height Analyses for Flood Insurance Studies (WHAFIS) was also made available in 1980 (Reference 7). With WHAFIS, FEMA initiated a large effort to incorporate the effects of wave action on the FIRMs for coastal communities along the Atlantic Ocean and Gulf of Mexico.

Along the New England coast with its very steep shore, structures identified as being outside of the flood hazard areas using the NAS methodology had experienced considerable wave damage from recent storms, most notably the northeaster of February 1978, a near 100-year event. The need to account for the effects of wave runup was recognized, and in 1981 FEMA approved a methodology that determined the height of wave runup landward of the stillwater line (Reference 8). FEMA's computer model for runup elevations was slightly modified in 1987 to increase the convenience of preparing input conditions, and again in 1990 to improve computational procedures and application instructions to conform with the best available guidance on wave runup (Reference 9).

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. It was surmised that the motion of submerged marsh plants absorbed wave energy, reducing wave heights. 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 (Reference 10). This addition has been incorporated into WHAFIS.

The muddy bottom situation occurs only at the Mississippi 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 waves. A methodology was developed for FEMA to calculate the wave energy losses due to muddy bottoms (Reference 11). Waves in the offshore areas are tracked over the mud bottom, resulting in lower incident wave heights at the shoreline. This is a phenomenon unique to the

Mississippi Delta, so the methodology has not been incorporated into WHAFIS and is not further discussed in these Guidelines.

In 1988, FEMA upgraded WHAFIS to incorporate revised wave forecasting methodologies described in the 1984 Edition of the Shore Protection Manual (Reference 12) and to compute an appropriately gradual increase or decrease of stillwater elevations between two given values (Reference 13).

In the performance of wave height analyses and the preparation of FISs, erosion considerations were left to the judgment of the contractors. General guidance directed that coastal erosion should be assumed where there was evidence of erosion from historical storms, but objective procedures for treating erosion were not provided. 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 coastal FISs.

In response to criticisms indicating 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 presented a number of recommendations to allow for a more realistic delineation of V zones and to better meet the objectives of the NFIP for actuarial soundness and prudent floodplain development (Reference 14). One recommendation was for full consideration of storm-induced erosion and wave runup in determining base flood elevations and mapping V zones. As part of that investigation, a study was made 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 100-year storm, it was determined that on the average, to prevent dune breaching or removal, a cross-sectional area of 540 square feet (ft^2) is required above the stillwater flood elevation and seaward of the dune crest. That standard for dune cross section has a central role in erosion assessment procedures presented later in these Guidelines.

The COE, Coastal Engineering Research Center (CERC), performed a study of the available quantitative erosion models for FEMA (Reference 15). This study 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 extremely simplified procedures for objective erosion assessment (Reference 16). These procedures have a direct basis in documented effects due to extreme storms, and are judged appropriate for treating dune erosion in coastal FISs.

As the official basis for treating flood hazards near coastal sand dunes, the Federal Register published new rules and definitions having an effective date of October 1, 1988. This included a revised definition of Coastal High Hazard Area in Section 59.1:

> "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.

As additional clarification of this matter, a definition of Primary Frontal Sand Dune was added in Section 59.1:

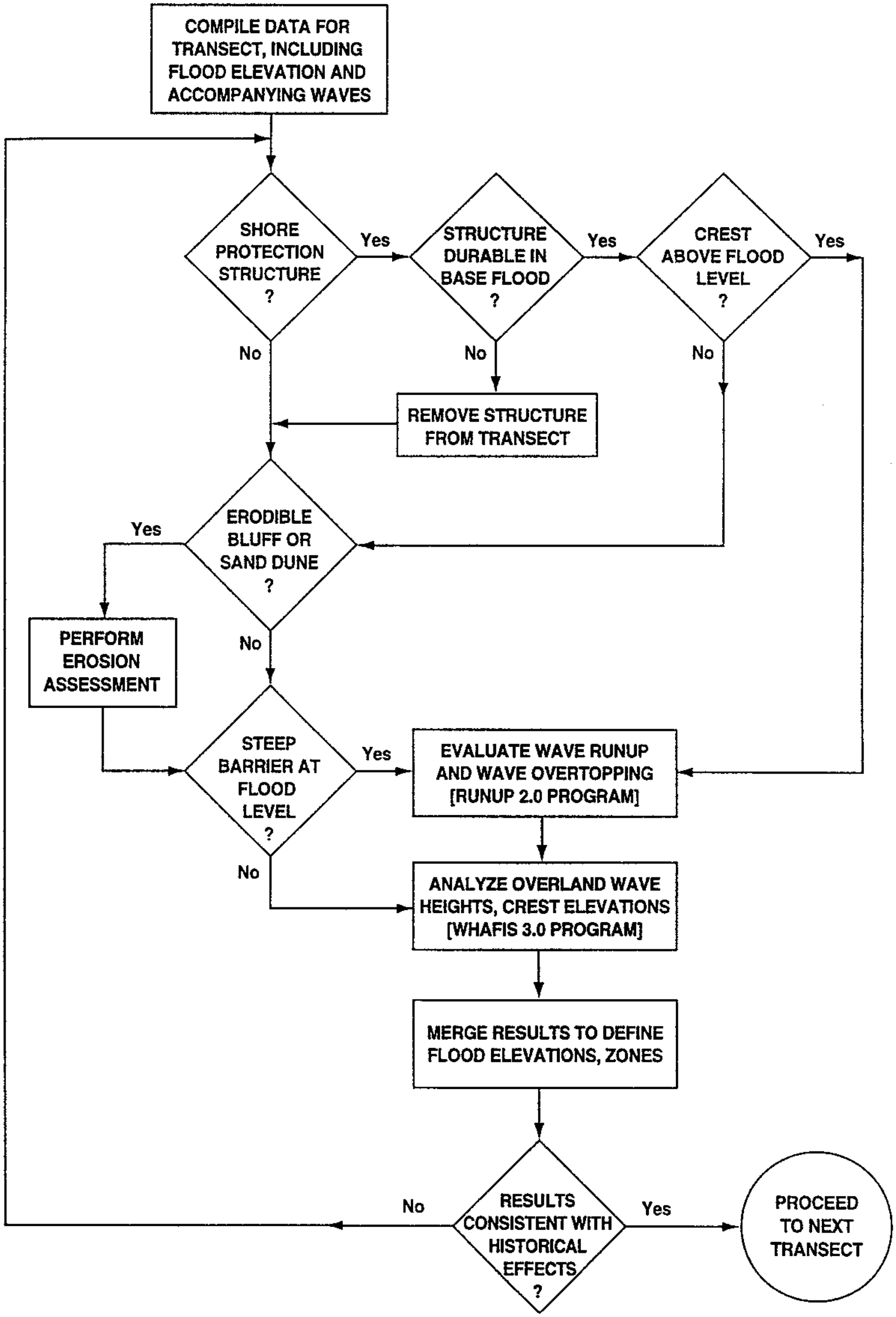
> "Primary frontal dune" means a continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent to the beach and subject to erosion and overtopping from high tides and waves during major coastal storms. The inland limit of the primary frontal dune occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope.

Also, a new section is included in Part 65 which identifies a cross-sectional area of 540 square feet as the basic criterion to be used in evaluating whether a primary frontal dune will act as an effective barrier during the base flood. Another consideration is the documented historical performance of coastal sand dunes in extreme local storms.

In 1989, the COE completed a review for the NFIP regarding coastal structures as protection against the base flood (Reference 17). Among technical topics addressed were predictions of wave forces, wave overtopping, and wave transmission for commonly occurring structures. These Guidelines incorporate procedural criteria recommended by the COE for evaluating structural stability.

1.3 Organization and Overview

Figure 1 presents a flowchart of appropriate procedures for defining coastal hazards of the base flood. Fundamental aspects of the 100-year flood are addressed in this sequence: stillwater elevation, accompanying wave conditions, stability of coastal structures, storm-induced erosion, wave runup and overtopping, and, finally, overland wave heights. Determination of stillwater elevations usually involves detailed statistical analyses, but added effects due to surface wave action are treated by simplified deterministic methodologies. This strategy avoids any potential complications due to conditional probabilities for simultaneous flooding effects. The sequence for treating these effects is entirely consistent in principle; for example, added wave effects are not resolved within



CHAPTER 2

CHAPTER 3

CHAPTER 4

CHAPTER 5

CHAPTER 6

CHAPTER 7

Figure 1. Procedural Flowchart for Defining Coastal Flood Hazards.

the equations commonly used to simulate coastal storm surges and establish stillwater elevation for the 100-year flood.

The order indicated in Figure 1 for activities, assessments, and analyses also outlines the appropriate organization of topics treated in these Guidelines. Chapter 2 describes general data requirements for conducting a coastal FIS, including that data needed as input to computer models. Chapter 3 discusses requisite evaluation of coastal structures potentially providing wave and/or flood protection. Chapter 4 considers the erosion assessment needed to project the configuration of a shore site during the base flood. Chapter 5 treats wave runup and overtopping occurring at shore barriers in flood conditions. Chapter 6 addresses the analysis of nearshore wave heights and wave crest elevations relevant to an FIS. All that material provides guidance on the models and procedures for treating individual transects at a study site.

FEMA has established specific models and procedures for the evaluation of shore structures, erosion, wave runup, and wave heights in the determination of coastal flood hazards. For many coastal areas, all four topics must be considered for an adequate treatment; for other coastal areas, application of only one or two of the FEMA methodologies may be required to produce reasonable results. Table 1 lists some typical shoreline types and the models that should be used for them.

Table 1

Model Selection for Typical Shorelines

Type of Shoreline	Models to be Applied		
	Erosion	Runup	WHAFIS
Rocky bluffs		x	x
Sandy bluffs, little beach	x	x	x
Sandy beach, small dunes	x		x
Sandy beach, large dunes	x	x	x
Open wetlands			x
Protected by rigid structure		x	x

The remaining material in these Guidelines adopts a more comprehensive view towards FIS completion. Chapter 7 deals with the integration of basic results into a coherent map for flood elevations and hazard zones. Chapter 8 defines required documentation of the process, decisions, and data used in determining coastal flood hazards for a community. Appendix A carries through a complete example study employing procedures and models applicable to a wide range of situations and conditions. For consistency with the NFIP and compatibility with FISs, these Guidelines use standard English units for all variables.

2.0 DATA REQUIREMENTS FOR COASTAL FLOOD HAZARD ANALYSES

To conduct a Flood Insurance Study for a coastal community, the initial effort must be to collect the wide variety of quantitative data and other site information required in ensuing analyses. This chapter describes the basic facts determining coastal flood elevations and their areal limits, including an outline for the storm expected to cause the local base flood, and characteristics of nearshore seabed through upland regions. Some data is directly input to computer models of flood effects, and other information finds application in interpreting and integrating the calculated results.

Each computer model of a separate flood effect is executed along transects, 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, in turn situated on work maps at the final scale of the FIRM. Work maps are used both to locate and develop the transects, and to interpolate and delineate the flood zones and elevations.

Aside from needed quantitative information, descriptions of previous flooding and the community in general should also be collected to aid in the evaluation of flood hazards and for inclusion in the FIS text. The data collection should start at the community level and proceed by contacting county, state, and Federal agencies. Private firms specializing in topographic mapping and/or aerial photography should also be contacted, following up suggestions provided by government agencies.

2.1 Stillwater Elevations

The stillwater elevations must be determined in a rational, defensible manner, and should not include contributions from wave action either as a result of the mathematics of the predictive model, or due to the data used to calibrate the model. Only the 100-year stillwater elevation is required for the coastal analyses, although the 10-, 50-, and 500-year elevations are provided in the FIS text, and the 500-year flood boundary is mapped on the FIRM.

Stillwater elevations may be defined by statistical analysis of available tide gage records, or by calculation using a storm surge computer model. FEMA has available a self-contained hurricane storm surge model that can provide flood elevations (Reference 18), and a synthetic northeaster model that simulates the wind and pressure fields of an extratropical storm for input to a storm surge computer model (Reference 19). These computer models are used for complex shorelines where gage records are limited, nonexistent, or non-representative, and usually indicate appreciable variations in flood elevations within a community. Reference 1 specifies procedure and documentation for coastal flood studies using a storm surge model. Of particular importance here, the surge model study can provide winds and water levels over time likely with the 100-year flood.

2.2 Selected Transects

Transects should be located with careful consideration given to the physical and cultural characteristics of the land so that they will closely represent conditions in their locality. They should be placed closer together in areas of complex topography, dense development, unique flooding, and where computed wave heights and runup may be expected to vary significantly. Wider spacing may be appropriate in areas having more uniform characteristics. For example, a long stretch of undeveloped shoreline with a continuous dune or bluff having a fairly constant height and shape, and similar landward features, may only require a transect every one to two miles; whereas, a developed area with various building densities, protective structures, and vegetation cover may require a transect every 1,000 feet or so.

Good judgment exercised in placing required transects 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 will be governed by variations in shore slope or gradient. On coasts with sand dunes, transects will be sited according to major variations in the dune geometry and the upland characteristics. In other areas where dissipation of wave heights may be most significant to the computation of flood hazards, transect location will be based on variations in land cover: buildings, vegetation, etc. A separate transect will usually be appropriate at each flood-protection structure. Areas with similar characteristics may be scattered throughout a community, so results from one transect can be applied at various locations.

Transects are located on the work map with the input data compiled on a separate sheet for each transect. The data for the transect are not taken directly along the line on the work map; they are taken from the area, or length of shoreline, to be represented by the transect so that the input data depict average characteristics of the area. Because of this, it is useful to divide the work map into transect areas for data compilation.

2.3 Topography

The topographic data must have a contour interval of 5 feet or 1.5 meters, or less. While more detailed information such as spot elevations or a smaller contour interval can be useful in the definition of the dune or bluff profile, and in the delineation of flood boundaries, it is not required. The data, usually in the form of maps, should be recent and reflect current conditions, or at a minimum, conditions at a clearly defined time. Note that transects need not be specially surveyed, unless available topographic data are unsuitable or incomplete.

If possible, the shore topography should be field-checked to note any changes due to construction, erosion, coastal engineering, etc. Any significant changes should be documented with location descriptions, drawings, and/or photographs. The community, county, and state are usually the best sources for topographic data. The U.S. Geological Survey (USGS) 7.5-minute series topographic maps should also be examined. The USGS maps may have a 5-foot contour interval, and if not, they are still often useful as a reference or base map.

2.4 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 aerial photographs must not be more than 5 years old unless they can be updated by surveys. A local, county, or state agency may have the coastline photographed on a periodic basis. They may provide photographs, or give permission to obtain them from their contractor. Because topographic maps are often developed from aerial photographs, the mapping contractor should also be contacted for data.

Aerial photographs can provide the required data on tree- and bush-type vegetation, but are only useful in identifying areas of grass-like vegetation, not 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. Also, local universities with coastal studies and/or Sea Grant programs should be contacted. Field surveys may be conducted in lieu of the above sources, but are more cost effective when used only to supplement or verify some of the data obtained from these sources.

2.5 Bathymetry

Bathymetric data can be acquired from National Ocean Service nautical charts, although any reliable source can be used. The bathymetry must extend far enough offshore to include the breaker location for the 100-year flood; although that depth may not be exactly known in the data collection phase, it can be assumed that a mean water depth of 40 feet will encompass all typical breaker depths. Bathymetry further offshore may also be useful in interpreting likely differences between nearshore and offshore wave conditions, necessary where offshore waves are more readily specified.

2.6 Storm Meteorology

The 100-year flood elevations represent a statistical summary, and likely do not correspond exactly with any particular storm event.

However, the meteorology of storms expected to provide approximate realizations of the base flood can be useful information in deciding recurrence intervals for historical events and in assessing wave characteristics likely associated with the 100-year flood. An important distinction is whether the 100-year flood is more likely to be caused by an extratropical storm or by a hurricane. This answer should be clearly established in the course of defining the stillwater elevations, since time history of water levels can be radically different in the two possible cases (Figure 2).

For an extratropical storm, commonly a winter storm occurring between October and Marsh, sustained winds seldom reach much above 60 mph, storm surge has relatively modest magnitude, and surge coincidence with spring high tides is usually required to attain the 100-year stillwater elevation. Extreme storms which occurred with lower tides can indicate wind and wave conditions also likely to accompany the 100-year flood. Thus, a fair amount of pertinent historical evidence may be assembled regarding expected meteorological conditions for the base flood arising from an extratropical storm. The dominant conditions include speed and duration of sustained winds, along with the storm size controlling fetch along which waves may be generated.

Where hurricanes are of primary importance, the 100-year flood is likely associated with central pressure deficits having exceedence probabilities between 5 and 10 percent (Reference 18). That

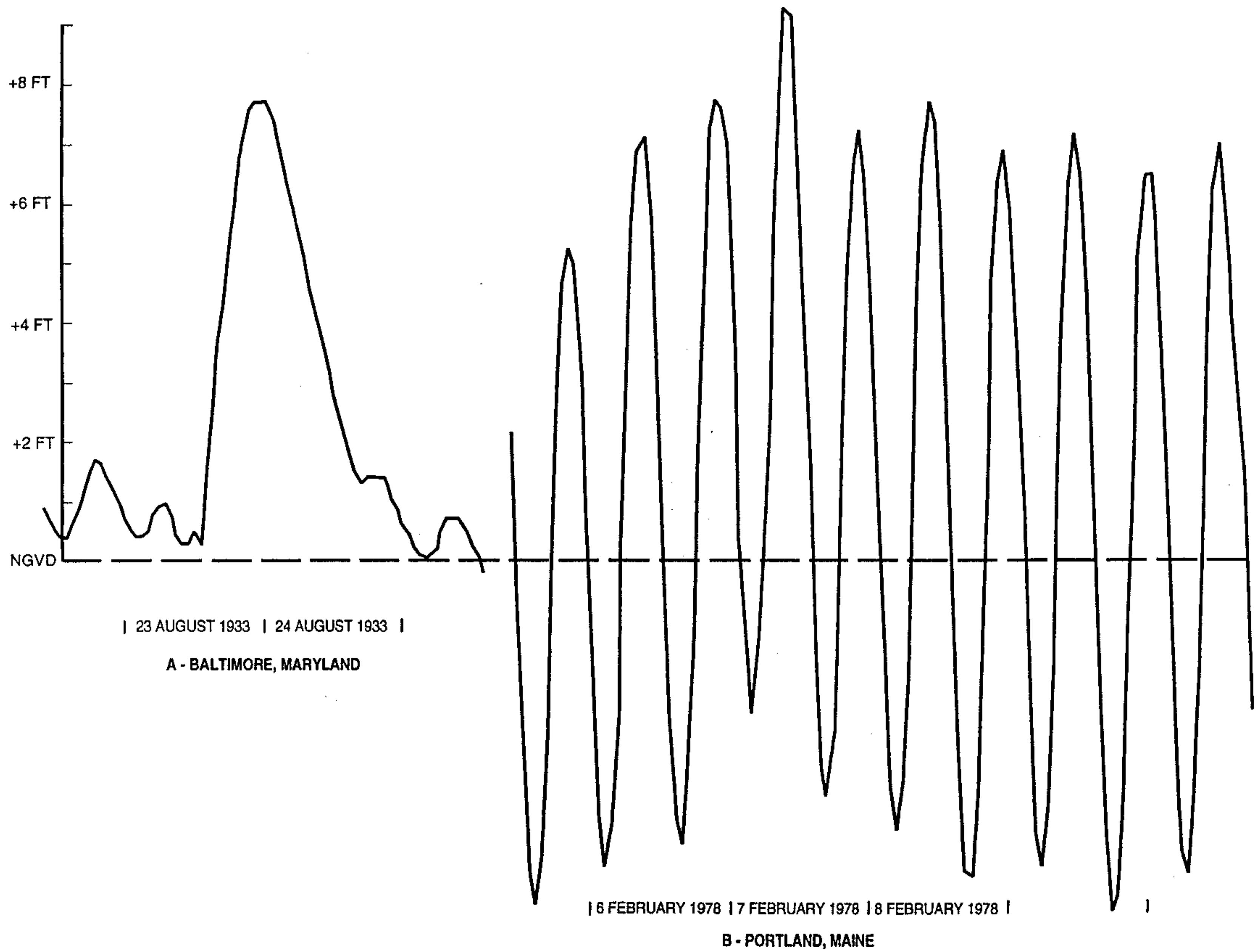


Figure 2. Gage Records of Floods Peaking Near the Local 100-Year Stillwater Elevations, Caused by, A- a Hurricane; and B- an Extratropical Storm.

description generally corresponds to a major hurricane, where sustained winds exceed 120 mph. Other meteorological characteristics are likely to be fairly typical for the study area, and may be determined using the hurricane climatology documented in Reference 20. That guidance includes localized probabilities for central pressure deficit, radius to maximum winds, and speed and direction of storm motion.

2.7 Storm Wave Characteristics

The basic presumption in conducting coastal wave analyses for an FIS is that wave direction must have some onshore component, so that wave hazards occur coincidentally with the base flood. That presumption appears generally appropriate for open coasts and along many mainland shores of large bays, where the 100-year stillwater elevation 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 base 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 2) 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 FIS models and analyses, the different wave descriptions include: 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 waves actually reaching a structure in shallow water for most stability or overtopping considerations). In following chapters on separate wave assessments, careful attention must be given to the stated requirements for wave description.

To proceed with general orientation, storm wave conditions may be developed from actual wave measurements, from wave hindcasts or numerical computations based on historical effects, and from specific calculations based on assumed storm meteorology. Where possible, two or all three of these possibilities should be pursued in estimating wave conditions expected to accompany the base flood at a study site. Such procedure can improve the level of certainty in estimated storm wave characteristics, by utilizing all available

Table 2. 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 Reference 5
$\overline{H}$	mean	average over all waves
H_{mo}	zero moment	defined by the variance of water surface, and 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

information. The following material surveys general sources for wave measurements or hindcasts, and then outlines current procedures of simplified wave estimation.

Wave measurements for many sites over various intervals have been reported primarily by the COE and by the National Data Buoy Center. Available data include records from nearshore gages in relatively shallow water (Reference 21) and from sites further offshore in moderate water depths (Reference 22). The potential sources of storm wave data also include other Federal agencies and some State or University programs.

The COE is a major source for long-term wave hindcasts along open coasts. That information is conveniently summarized as extreme wave conditions expected to recur at various intervals, for Atlantic hurricanes in Reference 23, and for extratropical storms in References 24 and 25, as examples. In some vicinities, other wave hindcasts may be available from the design activities for major coastal engineering projects.

Either measurements or hindcast results pertain to some specific (average) water depth, but such wave information may need to be converted into an equivalent condition at some other water depth for appropriate treatment of flood effects. References 12, 26, and 27 should be consulted for guidance regarding transformation of storm waves between offshore and nearshore regions, where processes to be considered include wave refraction, shoaling, and dissipation.

The other alternative in determining local storm wave conditions is to develop a specific estimate for the storm meteorology taken to correspond with the base flood. That can be done with relative ease for deep-water waves associated with a hurricane of specified meteorology, using the estimation technique provided in the Shore Protection Manual (Reference 12). For extratropical storms, a convenient PC-compatible program in the Automated Coastal Engineering System (ACES) (Reference 27) executes a modern method of wave estimation for specified water depth, incorporating some basic guidance from References 12 and 26. An outline of important considerations can assist preparations for developing a site-specific wave estimate.

Major factors in wave generation are wind speed and duration, water depth, and fetch length, the over-water distance towards the wind, along which waves arise (Reference 12). These factors determine flux of momentum and energy from the atmosphere into waves on the water surface. For some cases, fetch length might be estimated as straight-line distance in the wind direction, but current guidance (Reference 27) pertinent to many partially sheltered coastal sites indicates that a more involved analysis of restricted fetches must be performed for water basins of relatively complex geometry. The effective fetch length is derived as a weighted average of overwater distance with angle from the wind direction. With specified geometry for a restricted fetch, the cited ACES program (Reference 27) carries out computations necessary for the desired estimates of representative wave height and wave period.

The resulting wave field is commonly summarized by the significant wave height and wave period, namely, average height of the highest one-third of waves, and the corresponding time for a wave of that height to pass a point. Another useful measure is wave steepness, the ratio of wave height to wavelength: in deep water, the wavelength is 0.16 times the gravitational acceleration, times the wave period squared, that is, $(gT^2/2\pi)$. On larger water bodies and in relatively deep water, wave steepness is typically about 0.03 for extreme extratropical storms and about 0.04 for major hurricanes. These values can be used so that only a wave period or wave height may need to be determined.

2.8 Coastal Structures

Documentation gathered for each coastal structure possibly providing protection from base flood hazards should include the following:

- type and basic layout of structure
- dominant site particulars, such as local water depth, structure crest elevation, ice climate, etc.
- construction materials and present integrity

- historical record for structure, including construction date, maintenance plan, responsible party, repairs after storm episodes, etc.
- clear indications of effectiveness/ineffectiveness.

Much of this information may be developed through office activity, including a careful review of aerial photographs. In some cases of major coastal structures, site inspection could be advisable to confirm preliminary judgments.

2.9 Historical 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. 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 is available on historical flooding effects, special efforts should be made to acquire all recorded water elevations and wave conditions for the vicinity. That information can be used in estimating recurrence intervals for stillwater elevation and for wave action in the event, assisting an appropriate comparison to the base flood.

Local, county, and state agencies are usually good sources for historical data, especially the more recent events. It is becoming common practice for these agencies to record significant flooding with photographs, maps, and/or surveys. Some Federal agencies such as the COE, USGS, and National Research Council prepare post-storm reports for the more severe storms. Local libraries and historical societies may also be able to provide some useful data.

3.0 EVALUATION OF COASTAL STRUCTURES

The crux of the present evaluation is whether each individual coastal structure appears properly designed and maintained in order to protect against and withstand the base flood. If a particular structure can be expected to be stable through the base flood, the structure geometry may figure in all ensuing analyses of wave effects accompanying the flood: coastal erosion, runup and overtopping, and wave crest elevations). Otherwise, the coastal structure is considered to be destroyed during the base flood, and removed from the transect representation before proceeding with analyses of wave effects.

Reference 17 presents a technical review and recommends procedural criteria for evaluating coastal flood-protection structures in regard to the base flood. Reference 28 includes a self-contained account of the evaluation process, reproduced in Appendix B of this report. Reference 28 has been adopted as the basis for NFIP accreditation of new or proposed coastal structures in reducing effective flood hazard areas and elevations. Ideally, these evaluation criteria could be applied to existing coastal structures, but available information about older structures typically is not sufficient to complete the detailed evaluation. Where complete information is not available for existing structures, an engineering judgment about its likely stability can be based on visual inspection of physical condition along with any historical evidence of storm damage and maintenance.

Reference 17 addressed coastal flood-protection structures and identified the four primary types according to a functional standpoint: gravity seawalls, pile-supported seawalls, anchored bulkheads, and dikes or levees. Of particular note, Reference 17 recommended as a general policy that "FEMA not consider anchored bulkheads for flood-protection credit because of extensive failures of anchored bulkheads during large storms and difficulty in checking present conditions . . ."

Flood-protection structures can have a significant impact on a FIRM, perhaps directly justifying the removal of sizable regions from the Coastal High Hazard Area. The focus on flood-protection structures in Reference 28 (Appendix B) should not divert a recognition that similar considerations are appropriate in crediting the base-flood protection provided by structures in other categories, and that such credit can be important. In contrast to flood protection, a breakwater primarily may act to limit wave action and a revetment primarily may control shore erosion, but any stable coastal structure can notably affect results of various hazard analyses for the base flood and these effects should be taken into account. Reference 28 places the burden of proof or certification for new structures onto local interests, but the primary consideration in an FIS must be that the structure evaluation yields a correct judgment based on available evidence. This is necessary for accurate hazard assessments, since an effective structure might decrease flood impacts in one area while increasing erosion and wave hazards at adjacent sites. Of course, the more major the potential effects of a coastal structure, the more detailed should be the evaluation process.

4.0 EROSION ASSESSMENT

Coastal sand dunes usually extend above the 100-year stillwater elevation, but such barriers to flooding may not be durable due to massive shorefront erosion occurring during a 100-year flood. Storm-induced erosion will remove or significantly modify most frontal dunes on the U.S. coasts. This is particularly true on barrier islands known historically to be susceptible to storm overwash. Therefore, coastal erosion must be assessed before determining wave elevations and mapping V zones for the 100-year flood.

Available procedures for computing erosion show limited precision in documented hindcasts of recorded erosion quantities, and have questionable pertinence to the entire range of erosion effects possible on U.S. coasts. Therefore, a rather schematic treatment of expected erosion quantities and geometries has been developed as an appropriate approach for treating erosion in FISs at present. The overall rationale and level of detail in these erosion assessment procedures closely parallel the simple and effective NAS methodology for calculating wave action effects associated with storm surges (Reference 5).

The procedures described here are entirely objective, fundamentally reasonable, and empirically valid for treating dune erosion in the 100-year flood. These procedures are meant to give schematic estimates of eroded profile geometry suitable for the purposes of coastal FISs. The simplified estimates are suitable erosion approximations for extreme storms at sandy sites with typical open-coast wave and flood climate. The following erosion assessment procedures are intended for application to natural sites where there are no coastal structures such as breakwaters, groins, or revetments.

Quantitative considerations here are based on measured sand erosion accompanying extreme floods from hurricanes or extratropical storms on the U.S. Atlantic and Gulf coasts (Reference 16). For the study site, storm meteorology along with associated flood and wave characteristics may be used to assess whether such open-coast effects can be typical of anticipated local erosion for the base flood. Of course, any local historical evidence on storm erosion must also be examined in deciding applicability of the following procedures.

4.1 Basic Erosion Considerations

The primary factor controlling the basic type of dune erosion is the pre-storm cross section lying above the 100-year stillwater elevation (frontal dune reservoir). This area needs to be determined 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, 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 3 introduces terminology for two representative dune types. A frontal dune is a ridge or mound of unconsolidated sandy soil, extending continuously alongshore 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 backbeach berm on the seaward side. The dune toe is a crucial feature, and can be located as the junction between gentle slope seaward and a slope of 1 on 10 or steeper marking the front duneface. The rear shoulder, as shown on the mound-type dune of Figure 3, 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 3 shows the location of the "frontal dune reservoir," above 100-year stillwater elevation and seaward of the dune peak or rear shoulder. The amount of frontal dune reservoir determines dune integrity under storm-induced erosion.

To prevent dune removal in the 100-year storm, the frontal dune reservoir must typically have a cross-sectional area of at least 540

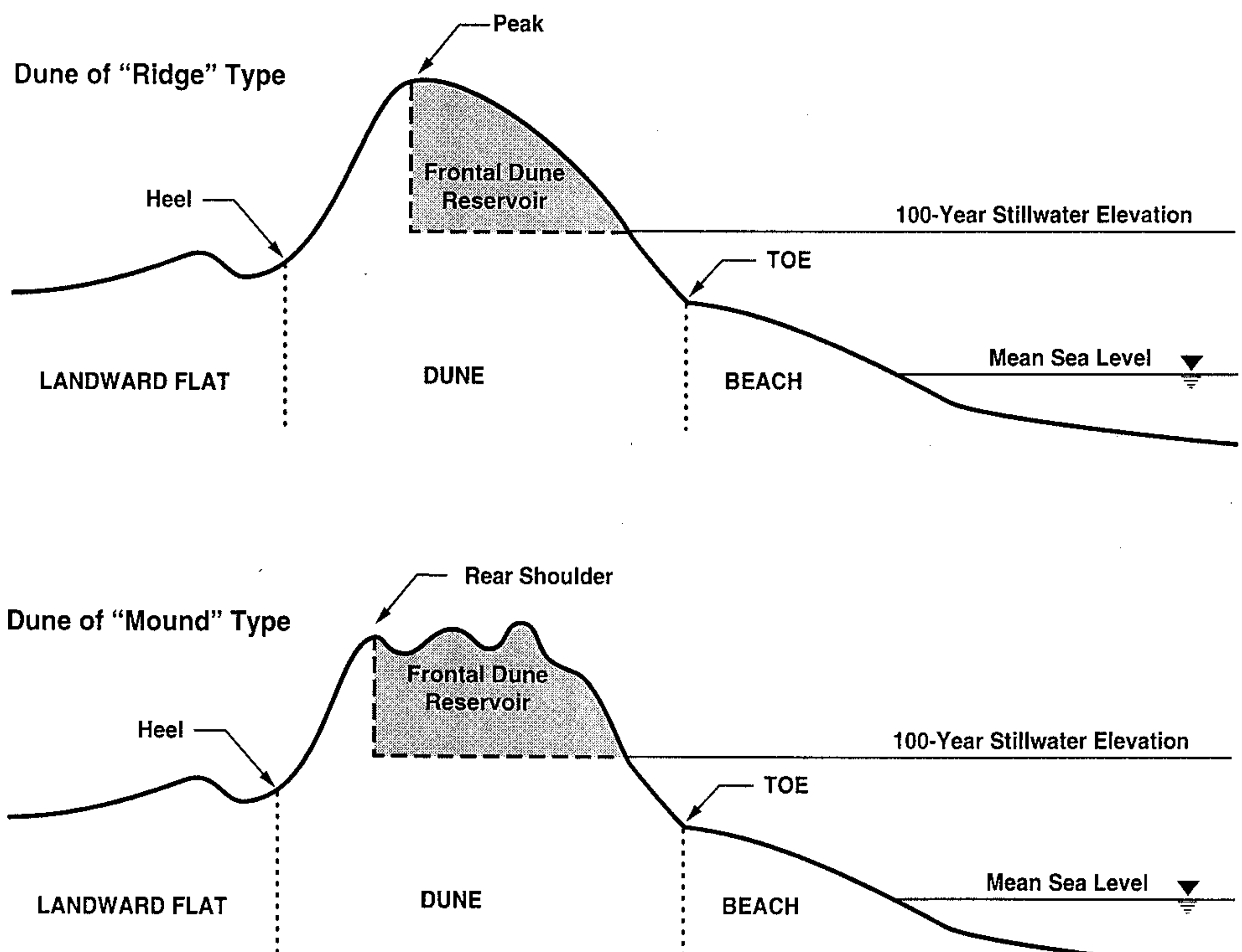


Figure 3. Dune Features and Present Terminology.

ft^2 (or 20 cubic yards volume per foot along the shore: References 14, 16). 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 100-year stillwater elevation. The simplified treatment of duneface retreat is described in Section 4.3.

If a dune has a frontal dune reservoir less than 540 ft^2, storm-induced erosion can be expected to obliterate the existing dune with sand transported both landward and seaward. The eroded profile should be estimated using procedures presented in Section 4.2. 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 feasible at present (Reference 15), so the frontal dune is simply removed in the present treatment.

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 3, judgment may be required to separate the sand reservoir expected to be effective in resisting dune removal from the landward portion of the pre-storm dune. The erosion assessment should usually address the summertime shore profile for hurricane impacts, and the wintertime profile for extratropical storms.

Figure 4 presents a complete flow chart of necessary erosion considerations, outlining the major alternatives of duneface retreat and dune removal. Figure 5 provides schematic sketches of the different geometries of dune erosion arising in coastal FIS assessments.

One additional factor complicating erosion assessment is the dissipative effect of wide sand beaches that shelter dunes from full storm impact and retard retreat or removal. If the existing slope between usual sea level and the 100-year stillwater elevation is 1 on 50 or gentler, careful examination of likely erosion during the 100-year flood will be required to avoid overestimation. This effect and other variables, such as sand size, dune vegetation, and actual storm characteristics at a specific site, emphasize the need for thorough comparison of estimated erosion to documented historical effects in extreme storms.

4.2 Treatment of Dune Removal

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 on 50 running through the dune toe. The eroded profile is taken to be

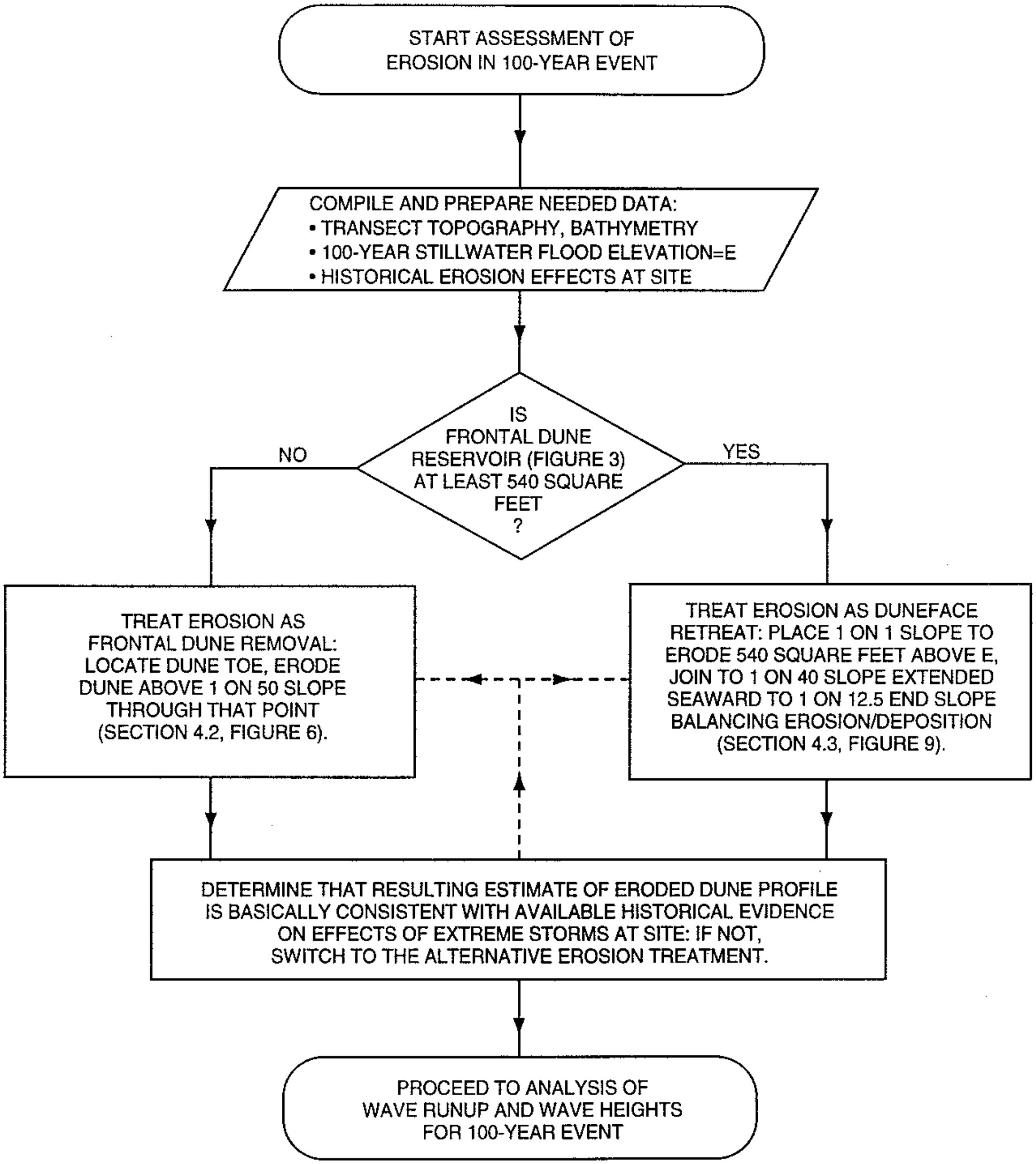


Figure 4. Flowchart of Erosion Assessment for a Coastal Flood Insurance Study.

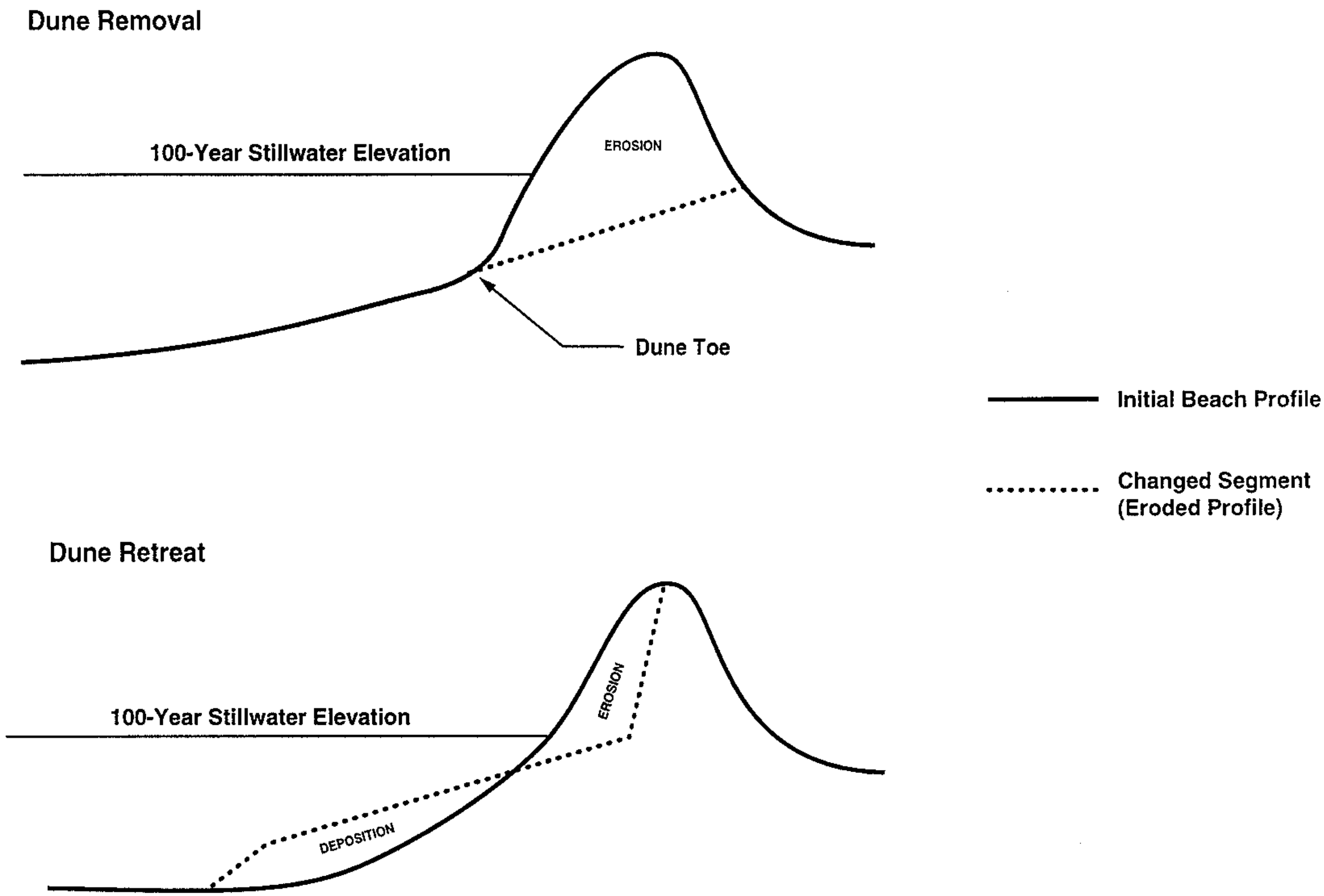


Figure 5. Schematic Cases of Eroded Dune Geometries with Planar Slopes.

that slope across the pre-storm dune, simply spliced onto the flanking segments of the given transect. This gives a gentle ramp across the extended storm surf zone 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 given transect.

Construction of an eroded profile focuses on the usually distinct feature termed the dune toe. That 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 backbeach 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 region. The alternative is to set the dune toe at the 10-year stillwater flood elevation in the vicinity: that appears to be a generally adequate approximation along U.S. 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 6-8 display examples of this treatment for a removed dune. These simple constructions give appropriate estimates for the limits of high ground removed during the 100-year flood, but cannot provide accurate representations of eroded profiles due to the complicated processes of dune failure. One example of overly simplified results

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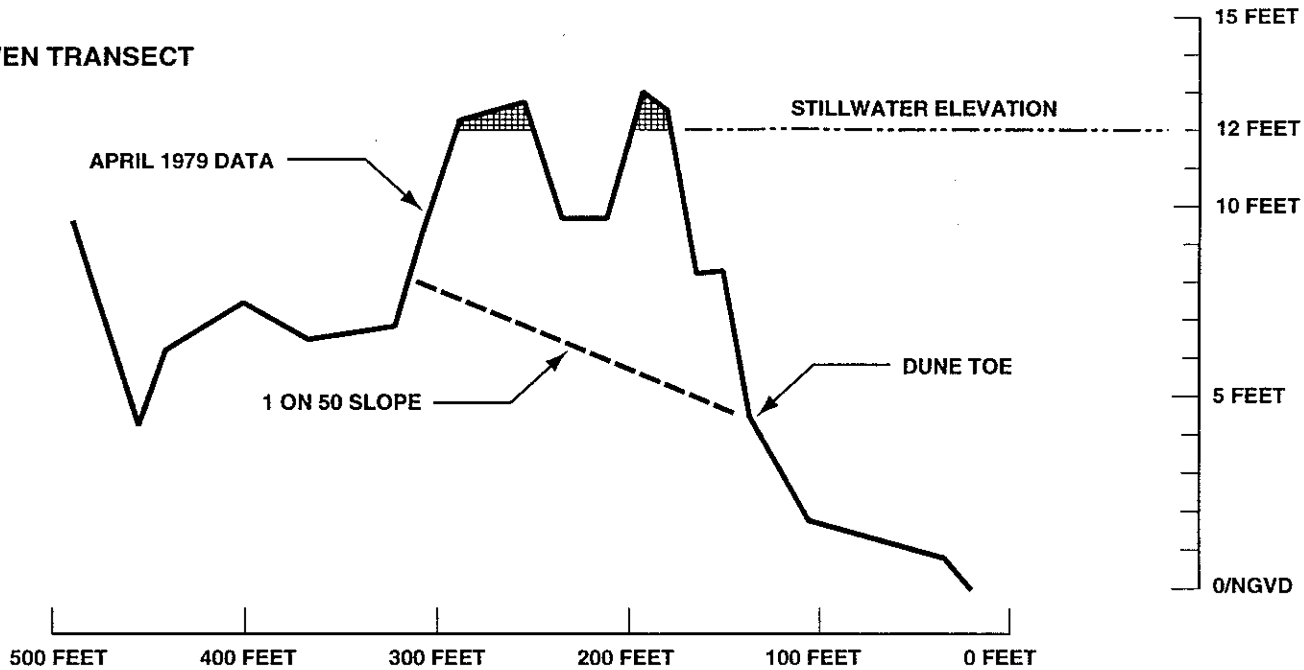


B - ANALYSIS

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.

C - ERODED TRANSECT

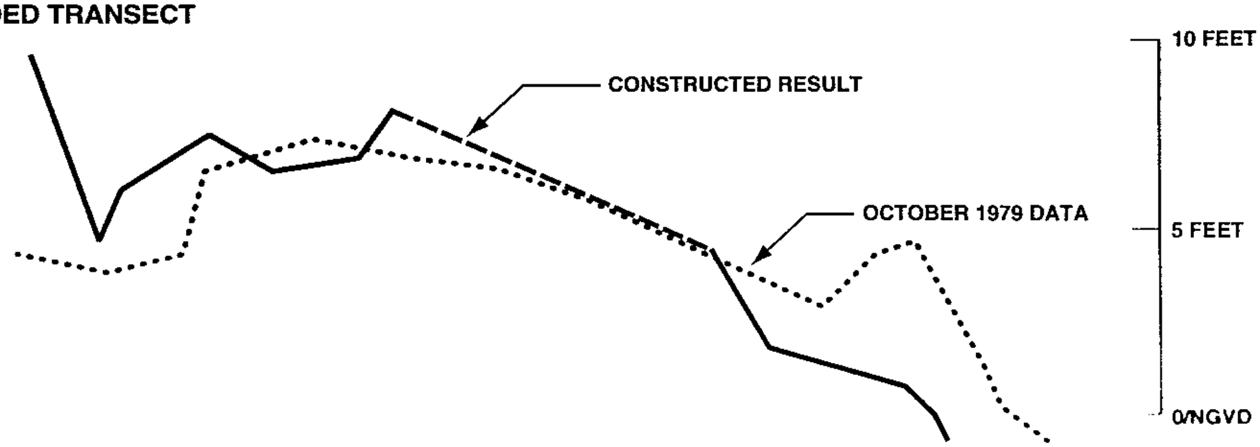


Figure 6. Quantitative example of dune removal treatment for Alabama profile eroded by the 1979 Hurricane Frederic. Situation is profile B-35 in Baldwin County, Alabama.

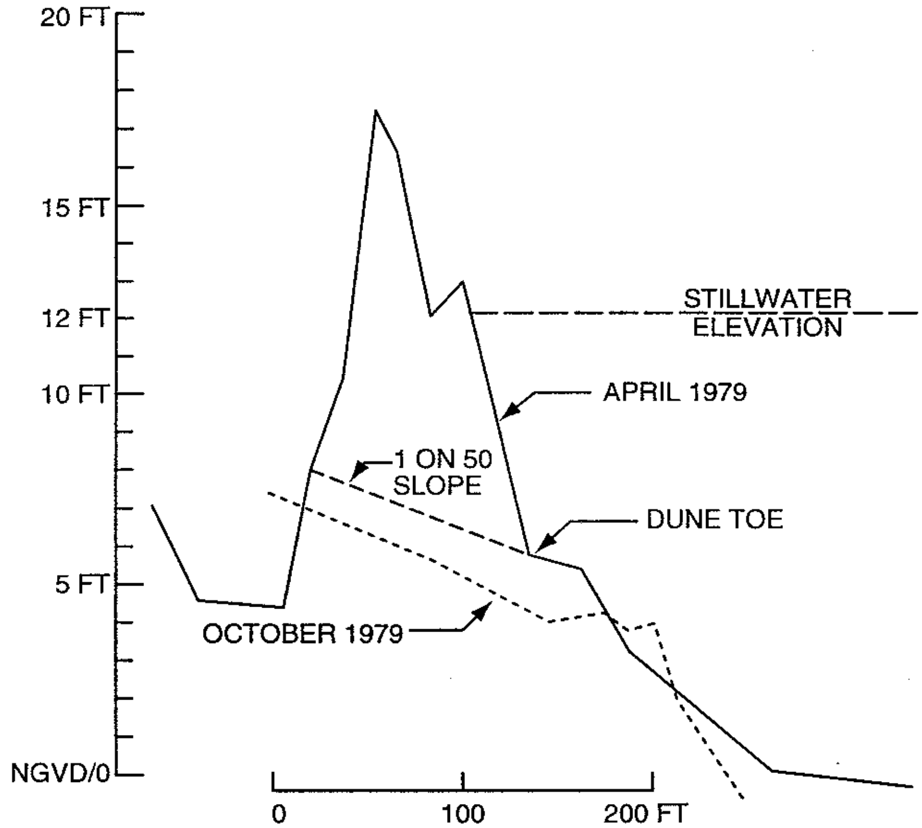


Figure 7 Case of relatively large dune removed by the 1979 Hurricane Frederic in Baldwin County, Alabama.

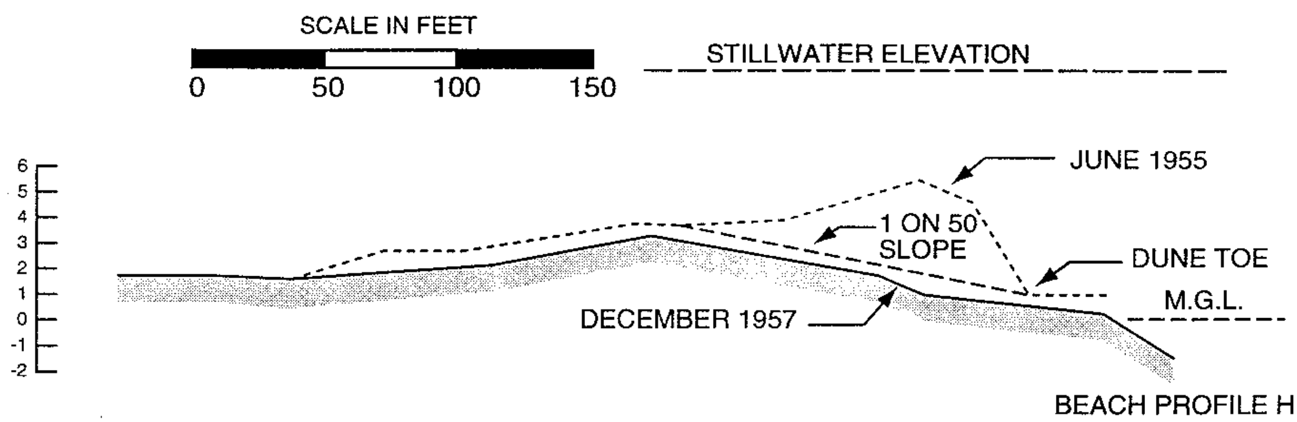


Figure 8. Erosion of relatively low profile by 1957 Hurricane Audrey in Cameron Parish, Louisiana.

is that deeper scour appears to occur where the frontal dune reservoir is relatively large.

The present viewpoint is consistent with this basic description of storm-induced erosion: greater erosion occurs where the pre-storm barrier provides more resistance, that is, has a relatively large cross section but still is removed during the 100-year flood. Net shore erosion appears to be maximum for situations where the dune barrier apparently just failed, and the eroded cross section can be much greater than in cases of duneface retreat. A slight opening to landward flow as an eroded dune becomes an overwash channel can result in much deeper scour than in cases of duneface retreat, where most shore erosion is above the stillwater elevation as duneface sand is continuously deposited in shallow water during the storm.

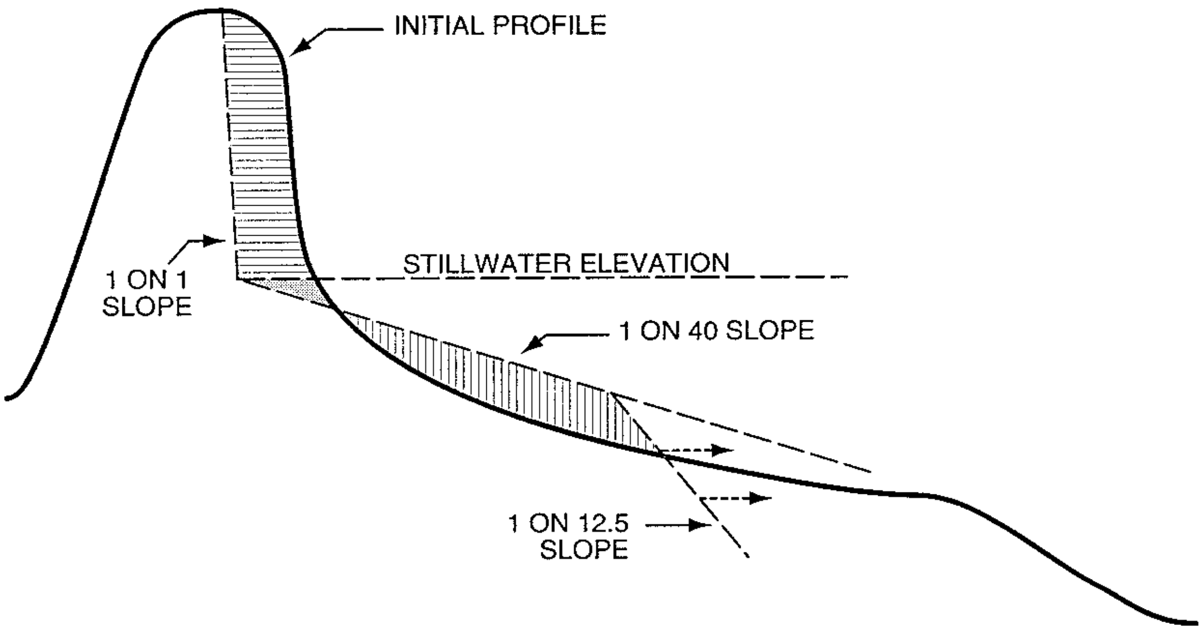
4.3 Treatment of Duneface Retreat

The procedure described here yields an eroded profile for duneface retreat in the 100-year flood, for cases where the frontal dune reservoir is at least 540 square feet. During such retreat, the frontal dune barrier remains basically intact and eroded sand is transported in the seaward direction. The post-storm profile provides a balance between sand eroded from the duneface and sand deposited at lower elevations seaward of the dune.

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 (Reference 29). Erosion above 100-year stillwater elevation is fixed at 540 ft^2, to guarantee an appropriate amount for the U.S. Atlantic and Gulf coasts (References 14, 16). (In the DHL model, erosion is determined as the variable depending on specified storm and site conditions.)

This modification to 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 (References 15, 16). Other simplifications in this treatment are that the variation of sand size is ignored and that the curved segment of the DHL post-storm profile is approximated by a planar slope.

Figure 9 summarizes the simple procedure adopted to treat cases of duneface retreat. The eroded profile consists of three planar slopes: uppermost is a retreated duneface slope of 1 on 1, joining an extensive middle slope of 1 on 40, which is terminated by a brief segment with a slope of 1 on 12.5 at the limit to storm deposition. Upper dune erosion is specified to be 540 ft^2 above the 100-year stillwater elevation and in front of the 1 on 1 slope. Geometrical construction balances the nearshore deposition with the total dune erosion of somewhat more than 540 ft^2 by an appropriate seaward



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 9. Procedure giving eroded profile in cases of duneface retreat. This is simplification of dune retreat model developed by Delft Hydraulics Laboratory of the Netherlands.

extension of the 1 on 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.

Figure 10 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. For this example, estimated erosion and deposition do not match well with those recorded, because there is a net sand loss shown on this profile and the event appears somewhat less extreme than a 100-year flood (judging from reported characteristics of Hurricane Eloise). 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 very large variations in natural situations, and this procedure presumes a generally representative value for 100-year flood conditions.

The basic procedure outlined in Figure 9 should also be applied in estimating erosion of high open-coast headlands or bluffs of sandy material. In such cases, parallel retreat of the existing face slope should be presumed, rather than using the typical 1 on 1 slope for the escarpment on an eroded sand dune, because that existing

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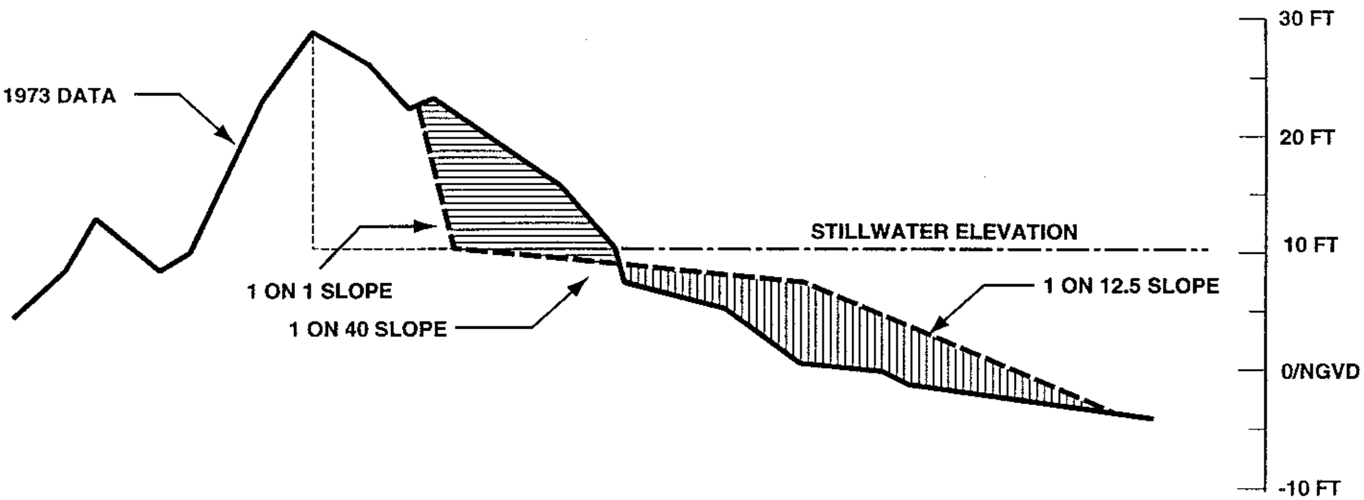

B - ANALYSIS

1 - FRONTAL DUNE RESERVOIR, OUTLINED BY--- , GREATLY EXCEEDS 540 FT² SO DUNEFACE WILL RETREAT.
2 - SPECIFYING EROSION ABOVE STILLWATER ELEVATION AS 540 FT², TOTAL DUNE EROSION ABOVE TWO UPPERMOST SLOPES IS FOUND TO BE APPROXIMATELY 620 FT².
3 - EROSION [≡] IS MADE TO EQUAL DEPOSITION [|||||||] BY EXTENDING MIDDLE (1 ON 40) SLOPE.

C - ERODED TRANSECT

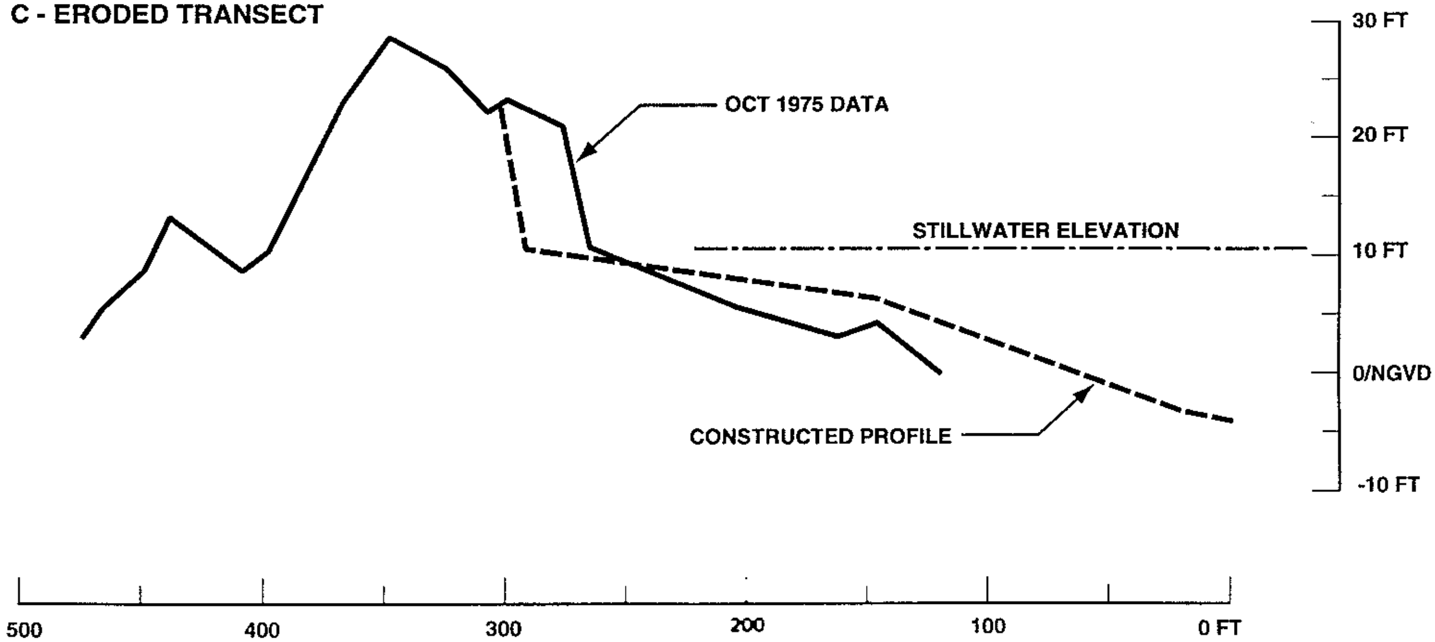

Figure 10. Example for duneface retreat treated by simplified version of D.H.L model, with erosion above stillwater elevation fixed at 540 ft^2. Situation is profile R-105 in Walton County, Florida, surveyed before and after 1975 Hurricane Eloise.

slope reflects actual consolidation properties of the headland or bluff material.

4.4 Finalizing Erosion Assessment

Based on measured erosion along the U.S. Atlantic and Gulf coasts, the demarcation between duneface retreat and dune removal in a 100-year flood has been set at a frontal dune reservoir of 540 square feet (References 14, 16). 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, so that validating the present erosion assessment by means of available evidence for a specific site is advisable.

At many sites, some historical evidence may be available regarding the extent of flooding, erosion, and damage in an extreme event comparable to the local 100-year flood. Then 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 FISs.

4.5 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 base flood. However, storm wave action can result in occasional extreme runups overtopping that barrier, yielding floodwaters running off or ponding landward of the dune. The mean overtopping rate with storm waves incident on a typical duneface retreat geometry has been determined to be (Reference 30).

$$\overline{Q} = 5.26 \exp [-0.253 F] \tag{1}$$

Here the overtopping rate $\overline{Q}$ has units of cubic feet per second, per foot alongshore (cfs/ft), and F is maximum height (in feet) of the dune remnant above stillwater elevation. 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 base flood along U.S. seacoasts, although expected wave characteristics will differ between hurricanes and extratropical storms at various sites. Note that recorded rates of overtopping can show sizable departures from the expected mean even with steady flood conditions (References 26, 31).

Despite uncertainties about actual overtopping rates for a dune remnant, Equation 1 gives a useful basis for outlining expected effects. The order of magnitude for severe overtopping may be taken as 1 cfs/ft, past allowable thresholds for structural integrity with bare soil behind steep barriers exposed to storm waves (Reference 26). From Equation 1, $\overline{Q}$ of about 1 cfs/ft corresponds to F of about 7 feet, so retreated remnants with less relief above the 100-year stillwater elevation certainly require consideration of possible flood hazards landward of the dune. Appropriate treatments for ponding or runoff behind barriers are outlined in the next chapter.

5.0 WAVE RUNUP, SETUP, AND OVERTOPPING

Wave runup is the uprush of water from wave action on a shore barrier intercepting stillwater level. 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 present purposes is the wave runup elevation, the vertical height above stillwater level ultimately attained by the extremity of uprushing water. Wave runup at a shore barrier can provide flood hazards above and beyond those from stillwater inundation and incident wave geometry, as sketched in Figure 11.

Two additional phenomena, wave setup and wave overtopping, may require explicit consideration for adequate treatment of the coastal flood hazards linked to wave runup. Wave setup generates a mean water surface elevated above the stillwater level, due to accumulation of water against a barrier exposed to wave heights attenuating in shallow water. Wave overtopping consists of any wave-induced flow passing over the barrier crest, so that flood water can provide wave-like impacts, sheet flow, and/or quiet ponding over an inland area. These phenomena and their quantitative evaluation will be addressed in later subsections, after describing the more basic assessment of wave runup for a coastal FIS.

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. Current policy for the NFIP

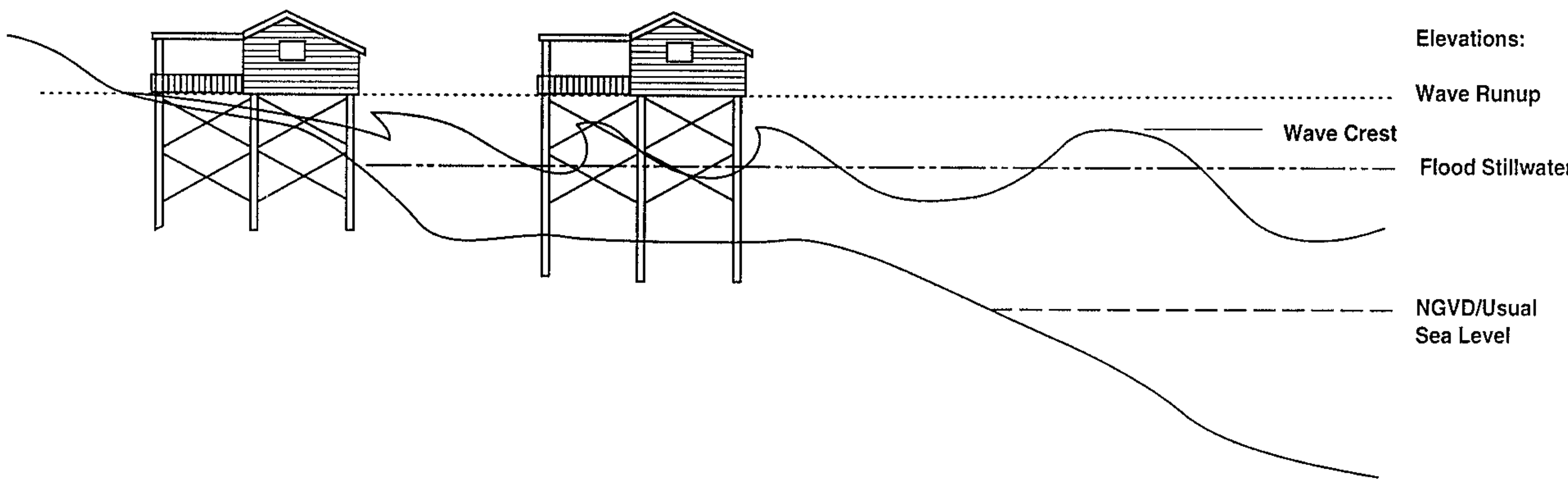

Figure 11. Schematic Illustration of Wave Effects Extending Above and landward of Stillwater Intercept on Transect.

is that the mean runup elevation (rather than some occasional extreme) for a situation is appropriate in mapping coastal hazards of the base flood. The following material describes content and usage of the Wave Runup Model, a FEMA computer program that determines mean runup elevation once the coastal flood situation is specified.

5.1 Wave Runup Model Description

The current version of the FEMA Wave Runup Model, called RUNUP 2.0, may be run either on a minicomputer (e.g., DEC VAX 11/750) or on an IBM-compatible personal computer (PC or PC/AT). Given the flood level, shore profile and roughness, and incident wave condition described in deep water, the program computes by iteration a wave runup elevation fully consistent with the most detailed guidance available (Reference 32). 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.

Some additional description of the workings of the Wave Runup Model 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 of Reference 33; this assumes the input shore profile (composite slope) is equivalent to a hypothetical uniform slope, as shown in Figure 12. 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 13 presents an overview of the basic computation procedure within RUNUP 2.0.

Basic empirical guidance incorporated within this computer model generally does not extend to vertical or nearly vertical flood barriers. For such configurations, RUNUP 2.0 usually will 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, it is preferable to develop a runup estimate using specific guidance in Figure 14, from the Shore Protection Manual (Reference 12). As within RUNUP 2.0, these empirical results for uniform waves should be utilized by specifying mean wave height and mean wave period for entry, and taking the indicated runup as a mean value in storm wave action. Shore configurations with a vertical wall are also addressed separately by detailed wave overtopping guidance presented in Section 5.7.

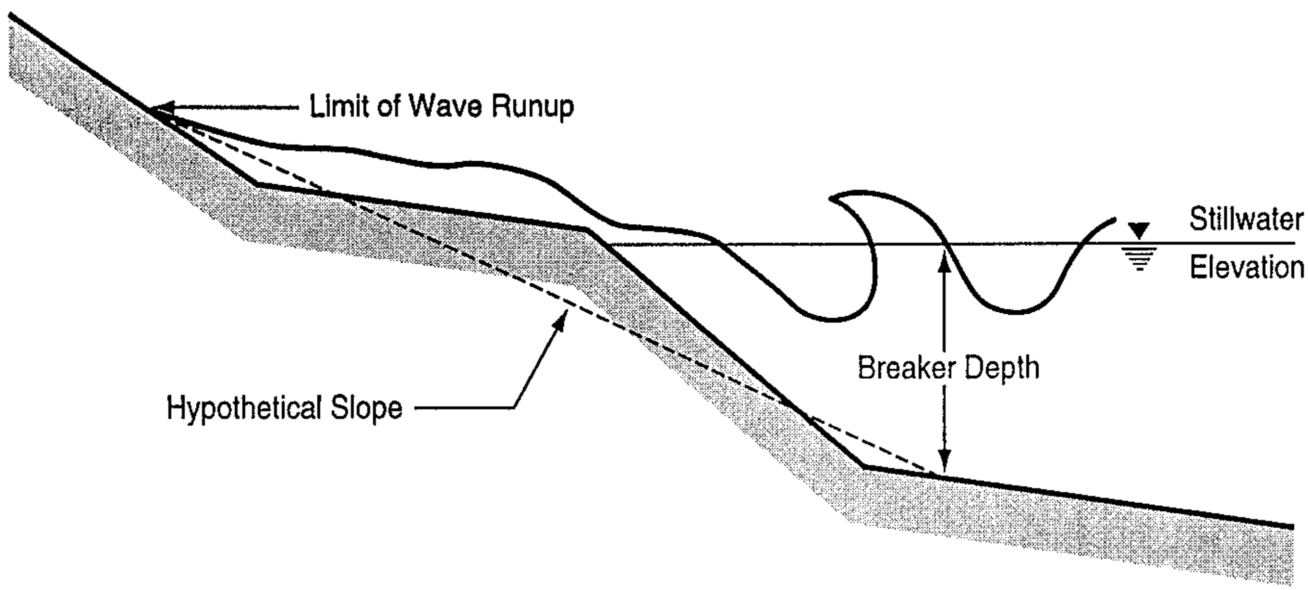

Figure 12. Hypothetical Slope for Determining Wave Runup on Composite Profiles.

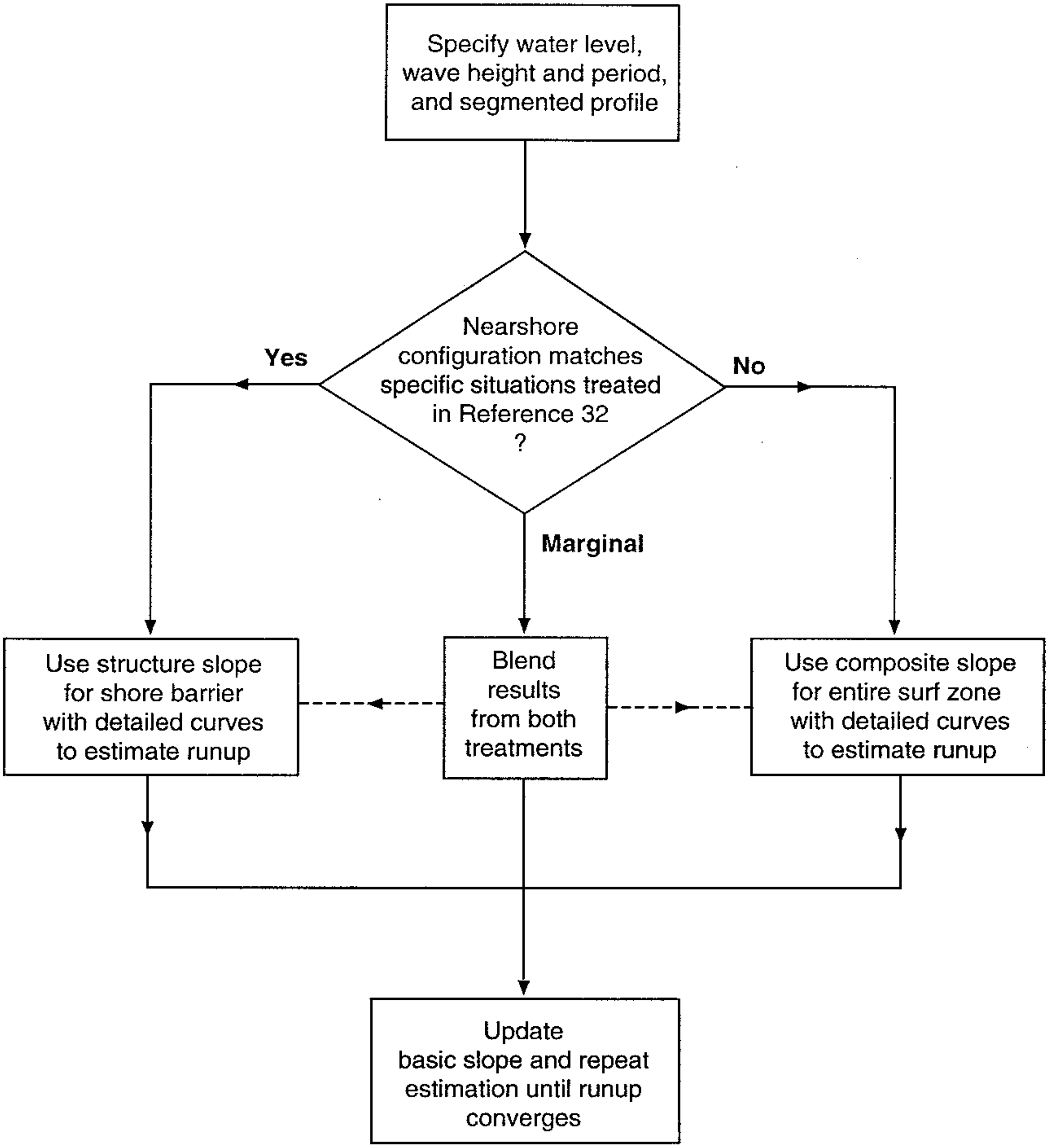

Figure 13. Overview of computation procedure implemented in modified FEMA Wave Runup Model.

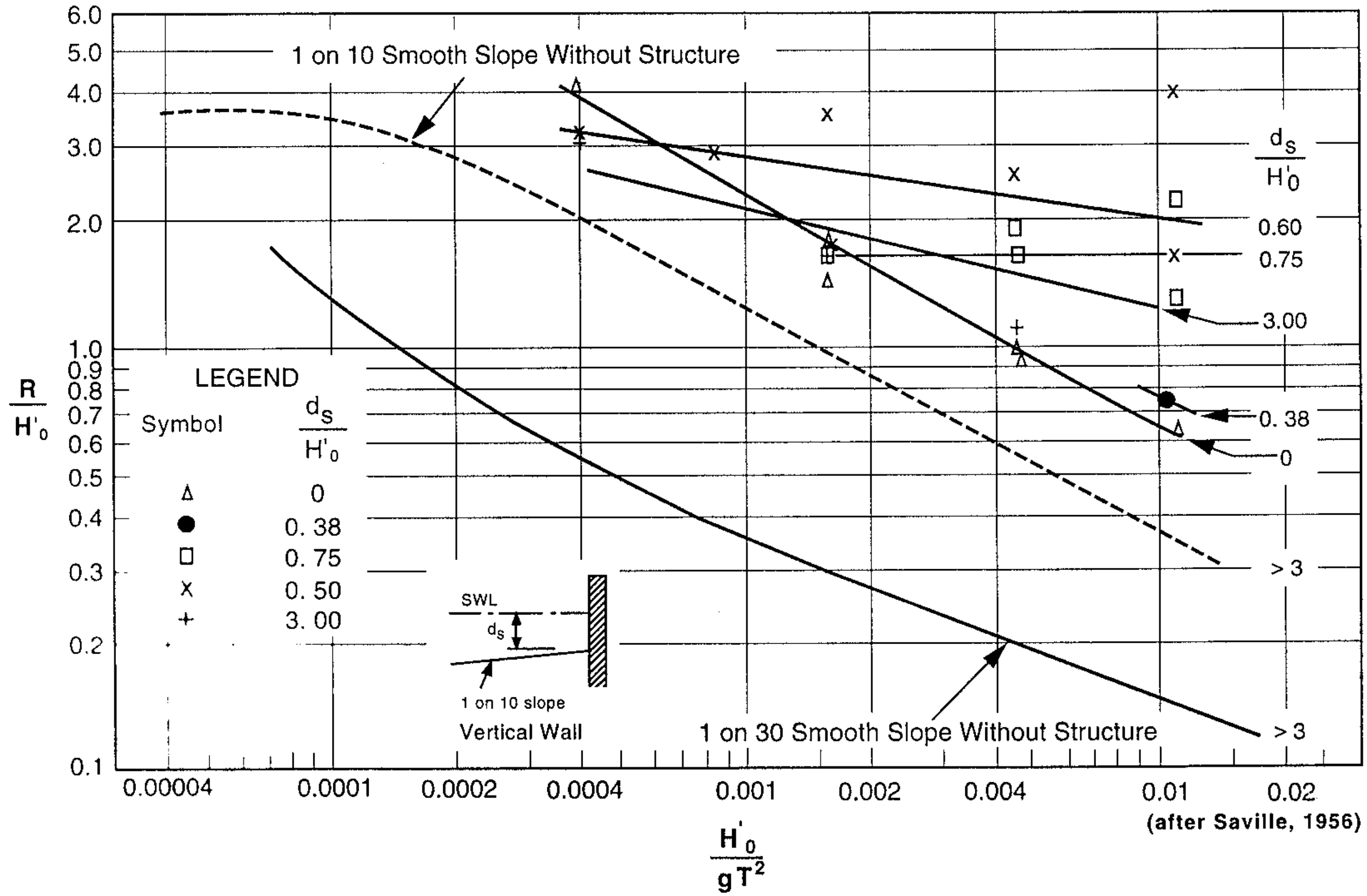

Figure 14. Wave Runup Guidance from Vertical Wall, From Reference 12.

5.2 Wave Runup Model Input Preparation

The input to the Wave Runup Model is done by transects. Transects should be located along the shoreline as previously specified. 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 the areas that are similar. 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 should be 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 after the data have been referenced to the National Geodetic Vertical Datum (NGVD). The profile should 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 stillwater elevation. If the landward profile does not extend above the computed runup (30 feet NGVD is commonly a maximum), it will be assumed that the last positive slope segment continues indefinitely. This is very common with low barriers, so the last slope should be carefully chosen to be representative. To complete the description, each slope segment of the profile will need a roughness coefficient, with some common values presented in Table 3. Roughness coefficient must be between zero (maximum roughness) and one (hydraulically smooth), and values for slope segments above stillwater level 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.

Transects are approximated by the minimum adequate number of linear segments, up to 20 as a limit. Segments may be horizontal, or higher at the landward end; portions with opposite inclination should be represented as horizontal when developing the transect approximation. Using many linear segments to represent a transect can be wasted effort, since the Wave Runup Model may combine adjacent segments in defining the appropriate approach and barrier extents. Bearing in mind the runup computation procedure, engineering judgment applied to transect representation can assist in obtaining the most valid estimate of wave runup elevation.

The input transect should reflect wave-induced modifications expected during the 100-year event, including erosion on sandy shores with dunes. Only coastal structures expected to remain intact throughout the 100-year event should be represented on a specific transect. Besides the transect specification, other required input data for the Wave Runup Model are the 100-year

Table 3. 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
0.50	Cast-concrete armor units: cubes, dolos, quadripods, tetrapods, tribars, etc.

stillwater elevation and the incident mean wave condition described in deep water. The specified stillwater elevation should exclude any contributions from wind-wave effects. If available elevations include wave setup, that component should be removed prior to using this model so that calculated runup elevations do not indicate a doubled wave setup. Basic empirical guidance refers 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 deep water generally conform to a Rayleigh probability distribution, so that 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. There is no exact correspondence between period measures, but mean wave period usually can be approximated as 0.85 times the significant wave period or the period of peak energy in the wave spectrum.

Table 4 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% of a fully arisen sea for extratropical storms. Commonly, there may be some difficulty in specifying a

Table 4. Appropriate Wave Conditions for Runup Computations Pertaining to 100-Year Event in Coastal Flood Insurance Studies

Mean Wave Period (sec)	Mean Deep-Water Wave Height (ft)
Hurricanes	
8	12
9	15½
10	19
11	23
12	27½
Extratropical Storms	
11	18
12	21½
13	25
14	29
15	33½

precise wave condition as accompanying the 100-year flood. In that case, it is appropriate to consider also wave heights and periods both 5% higher and lower than that selected (or whatever percentages suit the level of uncertainty), and to 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. A wide range in computed runups signals the need for more detailed analysis of expected wave conditions or for reconsideration of the transect representation.

5.3 Wave Runup Model 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 5. 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

Table 5. Description of the 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. At least one profile point with a ground elevation greater than the stillwater elevation must be specified. The number of Profile Lines cannot exceed 20.

Table 5 (continued)

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 with respect to NGVD, 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 hydrodynamic conditions for runup calculations on each profile. Namely, 100-year stillwater elevation along with mean wave height and period for deep water. Typically, stillwater elevation remains constant for a given profile, while the selected wave conditions closely bracket that expected to accompany the 100-year 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 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	Stillwater elevation with respect to NGVD, in feet.
7	Blank
8-12	Deepwater mean wave height, H_0, in feet, greater than 1 foot
13	Blank
14-18	Mean wave period, T, in seconds
19-80	Blank

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 stillwater elevation along with a mean wave height in deep water and a mean wave period.

The output as shown in Table 6 has two parts. The first page is a printout of the transect listed as a numbered set of profile points, 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: the values of runup elevation and breaker depth, each with respect to the specified stillwater elevation, along with an identification of the segment numbers giving the seaward limit to wave breaking and the landward limit to mean wave runup.

5.4 Wave Runup Model Output Messages

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

```
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 6. Output Example for the FEMA Wave Runup Model

****

OUTPUT TABLE

INPUT PARAMETERS			RUNUP RESULTS			
WATER LEVEL ABOVE DATUM (FT.)	DEEP WATER WAVE HEIGHT (FT.)	WAVE PERIOD (SEC.)	BREAKING SLOPE NUMBER	RUNUP SLOPE NUMBER	RUNUP ABOVE WATER LEVEL (FT.)	BREAKER DEPTH (FT.)
10.40	16.60	10.30	2	8	1.66	26.46
10.40	16.60	10.80	2	8	1.83	26.83
10.40	16.60	11.30	2	8	1.99	27.20
10.40	17.50	10.30	2	8	1.75	27.69
10.40	17.50	10.80	2	8	1.92	28.07
10.40	17.50	11.30	2	8	1.92	28.45
10.40	18.40	10.30	2	8	1.84	28.91
10.40	18.40	10.80	2	8	1.84	29.30
10.40	18.40	11.30	2	8	2.02	29.69
					Avg. 1.89	

- "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, since each transect is to represent an extensive area.

- "**** H_o/L_o LESS THAN 0.002 ****"
- "**** H_o/L_o 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 ten 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.

5.5 Wave Runup in Special Situations

Output of the Wave Runup Model should be examined carefully for each distinct situation, to assist proper interpretation and application of calculated results. One important consideration is that a mean runup elevation below the crest of a given barrier does not necessarily imply the barrier will not occasionally be overtopped by flood waters; the necessary supplementary examination of wave overtopping is addressed in Section 5.7 below. Other cases may yield results of more immediate concern, in that the Wave Runup Model may calculate a runup elevation exceeding maximum barrier elevation; this outcome can occur because the program assumes the last positive slope to continue indefinitely. The following material provides guidance on proper assessment of flood hazards beyond relatively low shore barriers, where wave runup surpasses the maximum ground elevation but falls off before it reaches the computed runup elevation.

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. When the runup overtops a barrier such as a partially eroded bluff or a structure, the flood water 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 Zones AO with a depth of flooding given (1, 2, or 3 feet). Ponding areas are designated as Zone AH (depth of flooding equal to 3 feet or less) with a flood elevation given. Standardized NFIP procedures have been developed for the treatment of sizable runoff and ponding, but are beyond the scope of this presentation; see Reference 1.

A fairly typical situation on 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 needs to be determined. Simplified procedures have been used to determine an approximate depth of flooding in the runoff area (Reference 34). These procedures are illustrated on Figure 15 and discussed below.

When the runup computed on the imaginary extension of the last positive slope is equal to or greater than 3 feet above the maximum ground elevation, the maximum runup is taken to be 3 feet above the ground crest elevation. This elevation decays to 2 feet above the ground profile at 50 feet behind the crest, and continues at this depth until it encounters other flooding. Computed runup is not adjusted if it is less than 3 feet above the ground crest. In the same initial 50 feet, this elevation decays to one foot above the ground and continues at this depth until it encounters other flooding. The runoff area from the ground crest to the limit of the other flooding is designated Zone AO with the appropriate depth of flooding specified.

A distinct type of overflow situation can arise 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. Reference 35 provides a simple procedure to determine realistic runup elevations for such situations, as illustrated in Figure 16. An extension to the bluff face slope permits computation of a hypothetical runup elevation for the barrier, with the imaginary

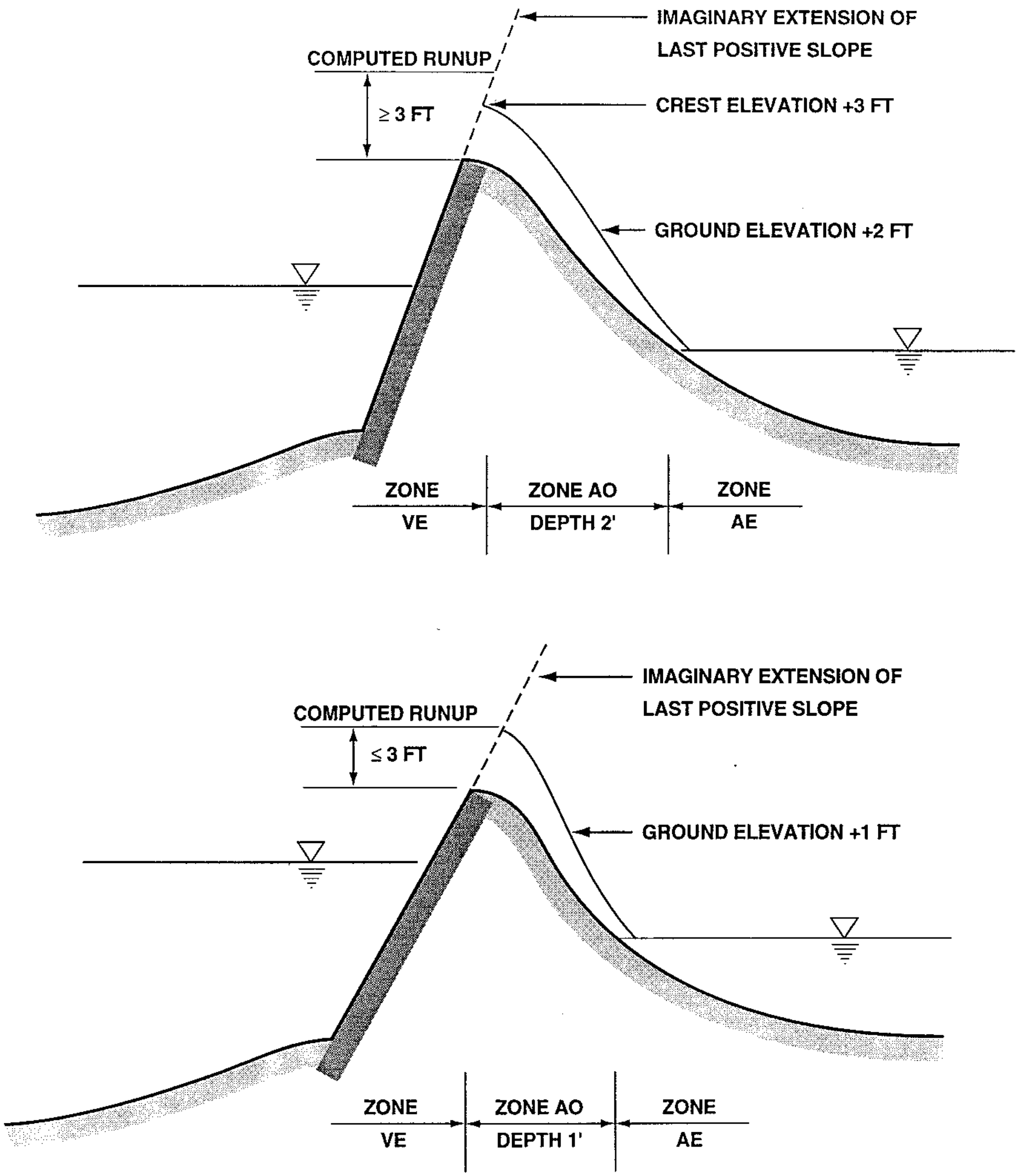

Figure 15. Simplified Run-off Procedures (Zone AO).

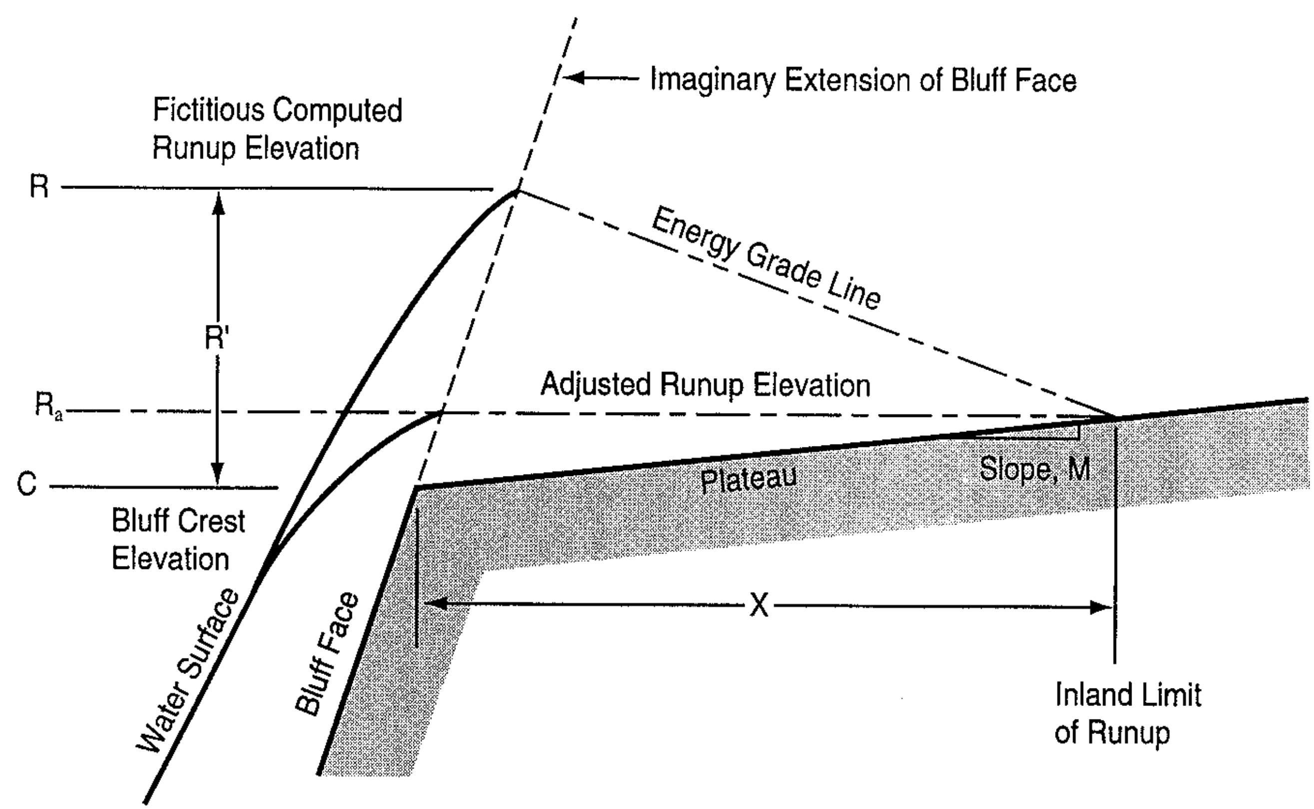

Figure 16. Treatment of Runup onto Plateau above Low Bluff.

portion given by the excess height $R' = (R\text{-}C)$ between calculated runup and the bluff crest. Using that height R' and the plateau slope m, Figure 17 defines the inland limit to wave runup, X, corresponding to runup above the bluff crest of $(m\,X)$ or an adjusted runup elevation of $R_a = (C + mX)$. This procedure is based on a Manning's "n" of 0.04 along with some simplifications in the energy grade line, and is meant for application only with positive slopes landward of the bluff crest. Reference 36 provides a different treatment of wave overflow onto a level plateau, for possible FIS usage.

These runup assessment procedures are given for general guidance, but situations may exist where they are not entirely applicable. For example, runup elevations need to be fully consistent with wave setup and wave overtopping assessments described in the following sections. In problematic cases, good judgment and reliance on the historical data should be used to reach a solution about realistic flood hazards associated with a shore barrier. Chapter 7 considers the integration of separately calculated wave effects into coherent hazard zonations for the base flood. When a unique situation is encountered, a Special Problem Report should be prepared and discussed with the Project Officer.

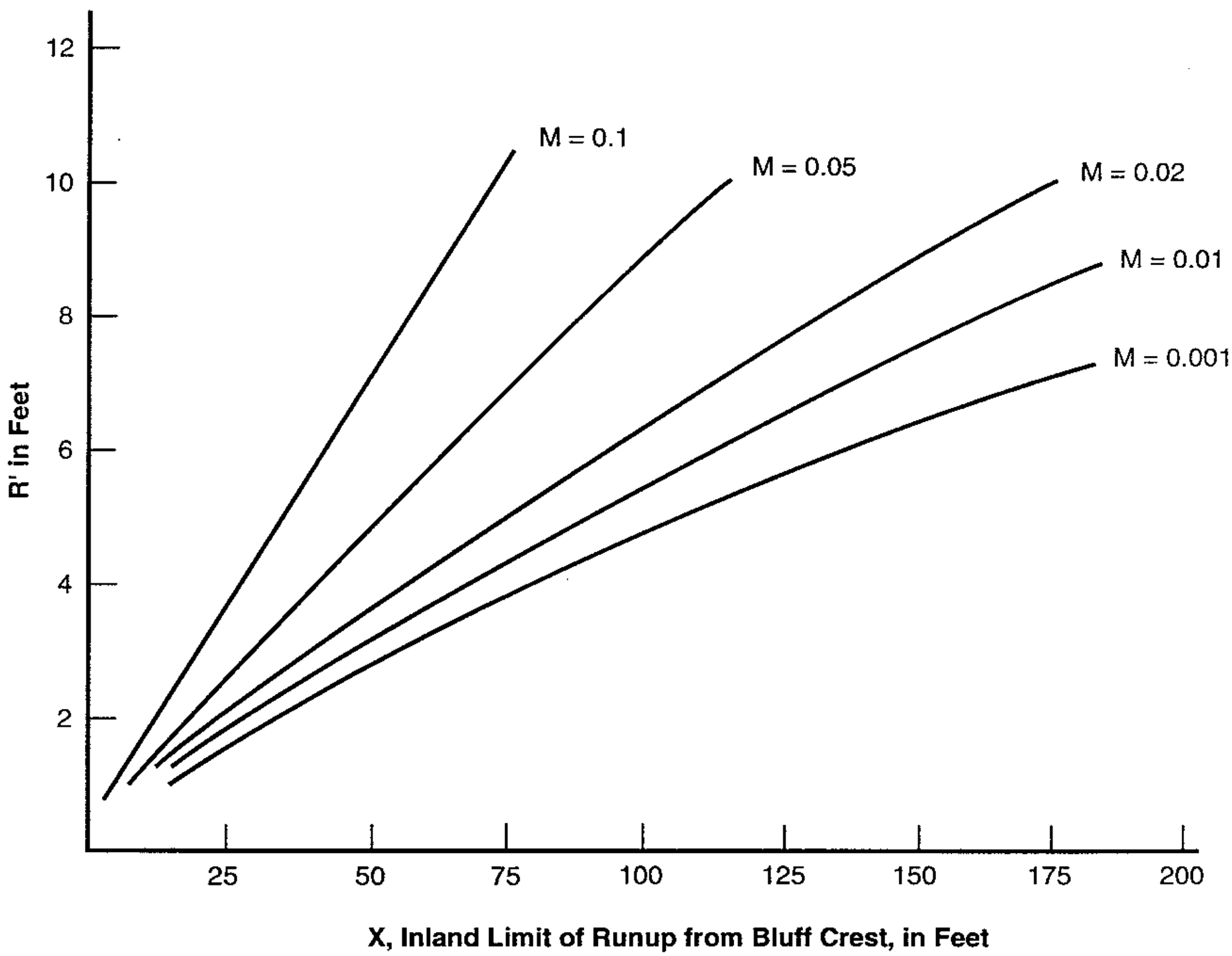

Figure 17. Curves for Computation of Runup Inland of Low Bluffs.

5.6 Wave Setup

Nearshore wave action can increase mean water elevation in front of a shore barrier by the phenomenon called wave setup, which is related to wave attenuation by breaking in shallow water. In treating the 100-year flood, focus may be restricted to the cumulative setup effect in the immediate vicinity of the shore barrier. Laboratory measurements of wave runup generally include the contribution due to wave setup, because runup elevations are defined relative to stillwater level in the absence of wave action.

A separate calculation for wave setup can be appropriate even if a wave runup elevation has already been determined, in part because the changed mean water depth can increase wave heights and crest elevations to be expected near the shore. In addition, empirical guidance within the Wave Runup Model is based on uniform laboratory wave action, so that incorporated setup might pertain to the field situation of swell waves from distant storms; setup effects may be much different in the local storm waves accompanying the 100-year coastal flood. If storm wave setup is found to exceed the wave runup calculated for a particular situation, the setup estimate must be applied as a lower bound for actual wave runup in further analysis of wave effects and base flood elevations.

Reference 12 provides straightforward empirical guidance on wave setup for various storm wave conditions and plane bottom slopes, as reproduced in Figure 18. Setup magnitude here is given in dimensionless form, as normalized by incident significant wave height. This guidance with typical significant storm-wave steepnesses about 0.03 to 0.04 indicates shore setups amounting to 7% or 8% of incident wave height. Incident wave conditions are specified in deep water as the significant wave height and the wave steepness, $H_{os}/L_{op})$, where $L_{op} = gT_p^2/2\pi$ is wavelength in deep water. Bottom slope may be taken as an overall average over the breaker zone between $d = 2H_o$ and d=0, if the bottom geometry is relatively simple. For other geometries, e.g., with a berm or reef in front of the shore barrier, the wave setup can be larger than given by Figure 18 and a more detailed examination may be required.

Wave setup also appears appreciably larger according to an independent treatment of storm waves on plane slopes, as outlined in Reference 26 for a relatively narrow spectrum describing incident wave energy. If historical evidence indicates greater setup increases of mean water depth in extreme floods than Figure 18 gives for the study site, a wave setup estimate based on that independent guidance may be conveniently developed through an ACES computer program provided in Reference 27. The program does not permit direct calculation of wave effects at d=0, but setup results from about d=H_o to the shallow limit of computations may be linearly extrapolated to the stillwater shoreline.

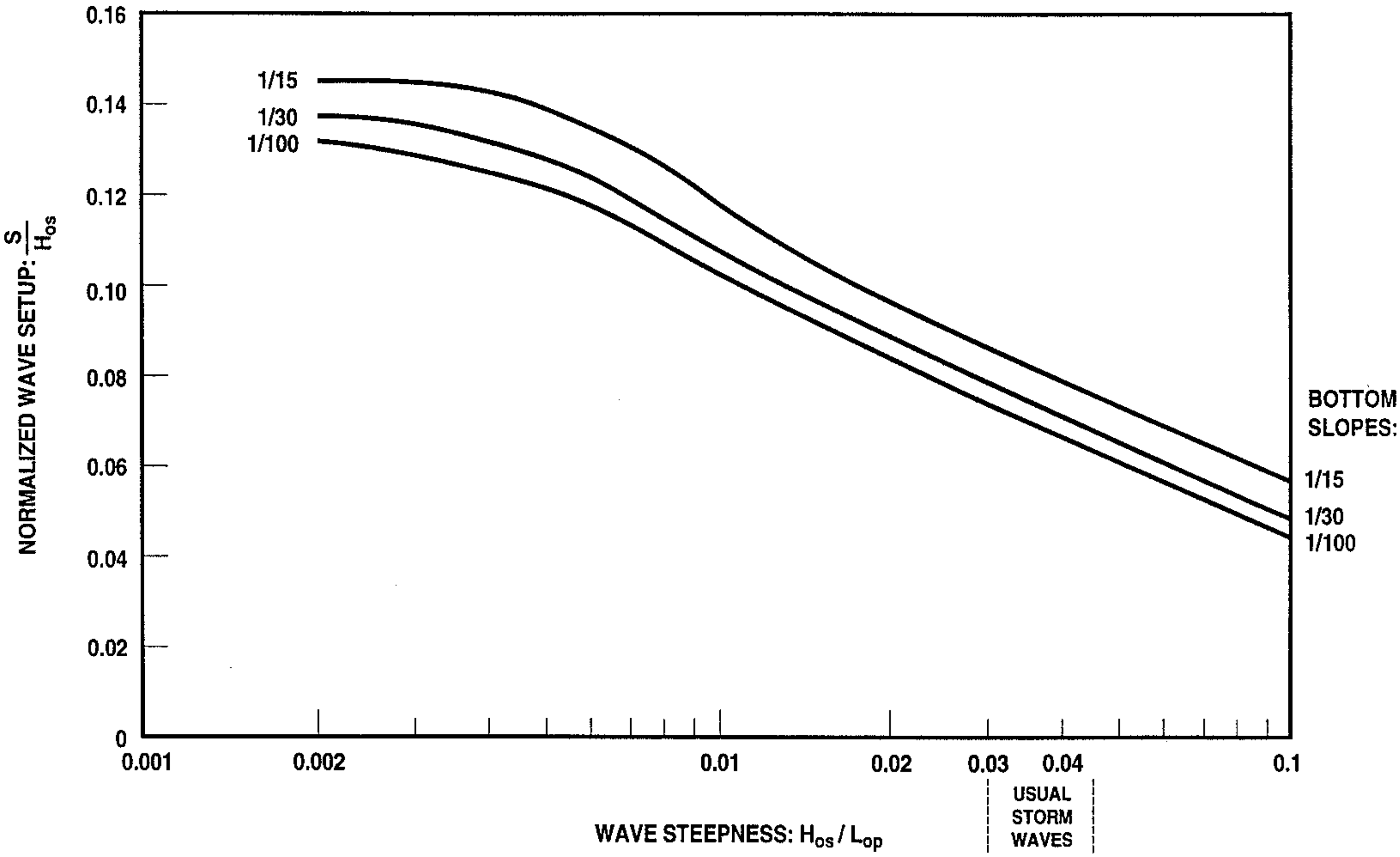

Figure 18. Guidance on Wave Setup in Irregular Wave Action, From Shore Protection Manual (Reference 12).

5.7 Wave Overtopping

Wave overtopping results when a shore barrier does not contain incident wave action, so that flood water penetrates to the protected area landward. This process of a partial halt and dissipation to storm waves is more difficult to treat than wave runup or wave setup. Important rates of wave overtopping can vary over several orders of magnitude, and can depend strongly on the detailed geometry of the barrier. That complicates the development of empirical guidance on wave overtopping, but there apparently is little demand for such guidance in coastal engineering practice. According to Reference 17, the design process for any major coastal flood-protection structure relies on site-specific model testing, rather than generalized overtopping guidance.

Of course, the assessment of potential wave overtopping for present purposes must rely on readily available empirical guidance, historical effects, and engineering judgment. Except for very heavy overtopping, useful guidance must be derived from tests with irregular waves, because the intermittently large overtopping discharges in storm situations could not be reproduced otherwise. Adding to the formal complexity of an adequate treatment for flood hazard assessment, overtopping effects may be cumulative so that the entire course of a flood event could require consideration, not just the peak conditions. Fortunately, only the order of magnitude of overtopping rates commonly needs to be estimated because there are clearly documented thresholds below which wave overtopping may be classified as negligible. On the other hand, it must be noted that if a preliminary estimate indicates severe overtopping which threatens the stability of a given structure, then that structure might be removed from the transect for analyses of the base flood, so no further overtopping consideration is required.

References 26 and 31 appear to provide the most trustworthy and wide-ranging summaries of mean overtopping rates with storm waves. Reference 31 addresses smooth plane or bermed slopes, and Reference 26 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 detailed assessment is needed for base flood conditions at a specific shore barrier.

The initial consideration should be an interpretation of 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 (Reference 5), a controlling runup magnitude may be defined as 1.6 times significant runup, or 2.5 times mean runup according to the Rayleigh distribution. If elevation of the barrier crest above 100-year stillwater elevation, or the barrier freeboard F, equals or exceeds (2.5 $\bar{R}$), then the landward area is not subject to wave-induced discharges in the base flood. That requirement might be supplemented by consideration of F near $(2\ \overline{R})$, corresponding to 4.5% of runups reaching the barrier crest according to the Rayleigh distribution. If $F \leq (2\overline{R})$, wave overtopping can certainly be appreciable during the base flood, and ponding or runoff behind the barrier should be assessed. Note that extreme runups introduced here, $(2\overline{R})$ and $(2.5\overline{R})$, bracket the elevation exceeded by the extreme 2% of wave runups, a value commonly considered in structure design.

Once the need for quantitative overtopping assessment is established, wave runup considerations become inapplicable because a runup elevation generally cannot be converted to an overtopping estimate. Also, the composite-slope method used in determining wave runup does not appear applicable for overtopping of barriers with composite geometry, because details of the wave transformation on a barrier influence the resultant overtopping rates. Wave overtopping estimates for a specified situation generally must 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 (References 26, 37).

Wave overtopping is specified as a mean discharge: water volume per unit time and per unit alongshore length of the barrier, commonly cfs/ft. Interpreting or visualizing a given overtopping rate should take into account that the actual discharges generally are 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. Water discharges corresponding to those regimes naturally depend on the incident wave size, but certain overtopping rates have been identified with various impacts (Reference 26). Among those rates, 0.01 cfs/ft seems to correspond to flooding that generally should be considered appreciable, and 1 cfs/ft appears to define an approximate threshold where structural stability of the shore barrier commonly becomes threatened by severe overtopping.

Once mean overtopping rate has been estimated for the base flood, determining resultant flooding may require a representative duration for the interval of overtopping. That duration can vary widely depending on the coastal flood cause, from a fast-moving hurricane to a nearly stationary extratropical storm (Figure 2). A minimum assumption for the duration of flood-peak overtopping would generally be one to two hours. Durations on the order of ten hours or more could be appropriate for cumulative effects in an extratropical storm causing flooding over multiple high tides.

Figure 19 summarizes some empirical overtopping guidance for storm waves, in a schematic form meant to assist deciding the likely significance of flooding behind a coastal structure. Variables describing the basic situation are cotangent of the front slope for

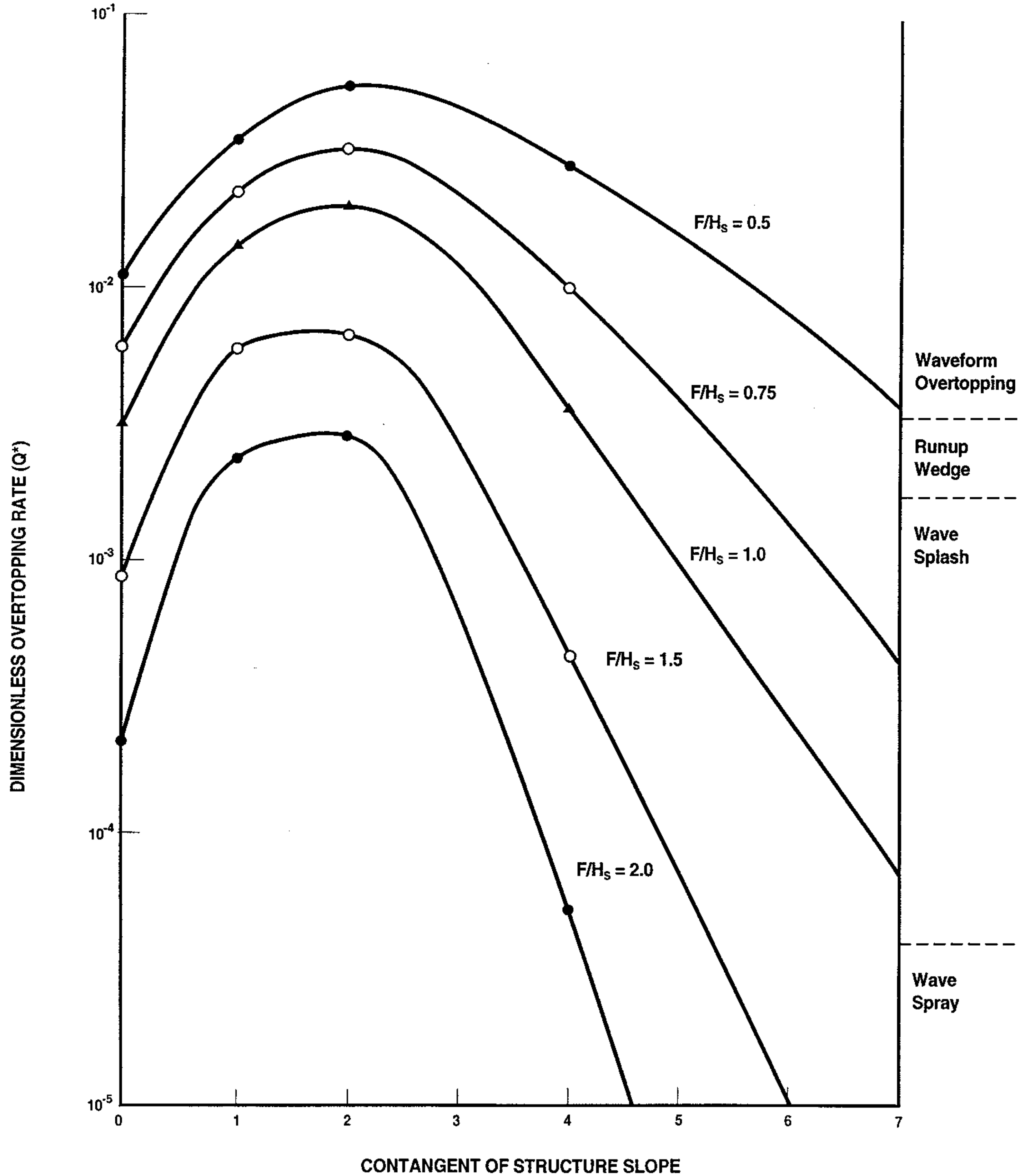

Figure 19. Schematic Summary of Storm-Wave Overtopping at Structures of Various Slopes and Freeboards, Based on References 26 and 31.

a smooth structure with ideally simple geometry, and freeboard of the structure crest above stillwater level, as normalized by incident significant wave height, F/H_s. The mean overtopping rate, $\overline{Q}$, is provided in dimensionless form as

$$Q* = \overline{Q}/(gH_s^3)^{0.5}, \tag{2}$$

with test results shown for structure slopes of 1 on 1, 1 on 2, and 1 on 4 (Reference 31), and for a smooth vertical wall (Reference 26). These results pertain to: significant wave steepness of about $2\pi H_s/gT_p^2 = 0.035$, fairly appropriate for extreme extratropical storms or hurricanes; water depth near the structure toe of about $d_t = 2H_s$, so that incident waves are not appreciably attenuated; and moderate approach slopes, of 1 on 30 for a vertical wall, or 1 on 20 for other structures. The major feature of interpolated curves is fixed as a maximum in overtopping rate for structure slope of 1 on 2, corresponding to the gentlest incline producing (at this wave steepness) total reflection rather than breaking, and thus peak waveform elevations (Reference 38).

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 about 0 and 4). Also, for freeboards considered here, a vertical wall (cotangent 0) permits less overtopping than common sloping structures with cotangent less than about 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. Basic effects of those differences can be outlined for use in simplified overtopping assessments.

For sloping structures sited within the surf zone ($d_t < 2H_s$), Reference 31 indicates that basic overtopping guidance in Figure 19 can be used with attenuated rather than incoming wave height. 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 19, and the indicated Q* value is then converted to $\overline{Q}$ using H_s'. Note that the presumed wave attenuation ignores any wave setup as a small effect with the partial barrier, and that d_t should always correspond to the scour condition expected in wave action accompanying the base flood.

Figure 19 might also be made applicable to rough slopes, using a roughness coefficient (r) from Table 3 to describe the effectively increased freeboard with greater wave dissipation on the structure. Reference 31 proposed that effect of structure roughness be formulated as F/r, and Reference 29 confirmed a similar dependence of overtopping on roughness in measured results for irregular waves. The overtopping relation reported as reliable in Reference 39 is

$$Q* = 8 \cdot 10^{-5} \exp[3.1(rR* - F/H_s)] \quad (3)$$

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 H_s, for a barrier slope given as the tangent m. Equation 3 is meant to pertain to very wide ranges of test situations with moderate overtopping, but appears very approximate in comparison with specific results for $r=1$ shown in Figure 19. It may be advisable to evaluate Equation 3 for both smooth and rough barriers, then use the ratio to adapt a Figure 19 value for the case with roughness. Note that References 31 and 39 provide further overtopping guidance on the effects of composite profiles, oblique waves, and shallow water with sloping structures.

For overtopping of vertical walls, effects of wave attenuation appear relatively complex, but Reference 26 provides extensive empirical guidance on various structure situations with incident waves specified for deep water. Figure 20 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. Reference 26 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 about 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),

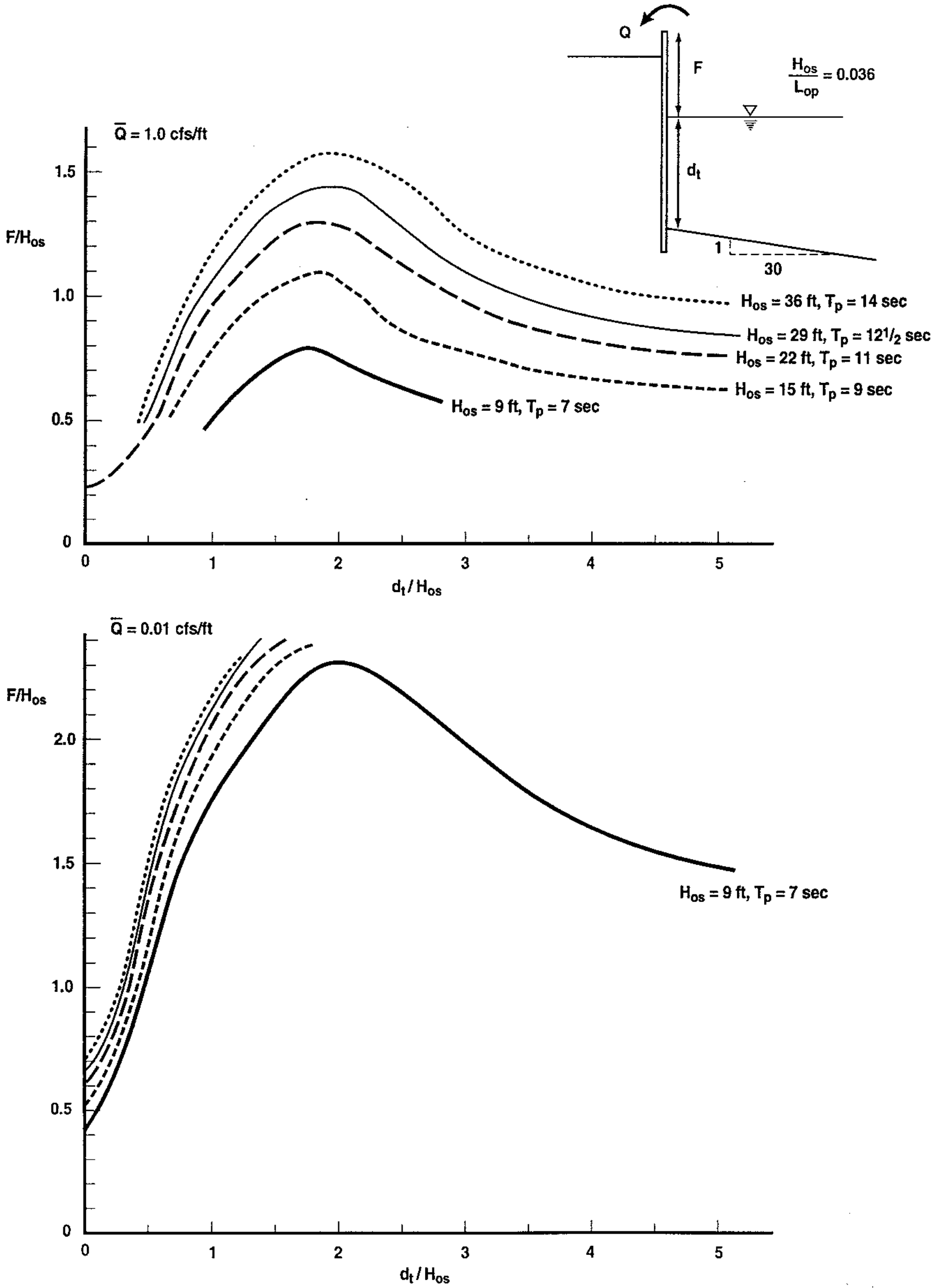

Figure 20. Required Freeboard of Vertical Wall to Limit Mean Overtopping Rate to Certain Values, Based on Design Curves of Reference 26.

intermediate overtopping, or severe overtopping ($\overline{Q} > 1$ cfs/ft) expected for the base flood. Likely runoff or ponding behind the wall then needs to be identified, and severe overtopping requires delineation of the landward area having wave impacts and velocity hazard. Chapter 7 outlines some common zonations of flood hazards near shore barriers in describing the integration of computed wave effects.

Considering Figure 20 along with common wall and wave heights, wave overtopping dangerous to structural stability appears the usual case in the base flood. An assessment of failure during the base flood for typical walls would be fully consistent with one recommendation of Reference 17, namely that "FEMA not consider anchored bulkheads for flood-protection credit because of extensive failures..."

Interpretation of estimated overtopping rate in terms of flood hazards is complicated by the projected duration of wave effects, by the increased discharge possible under storm winds, by the varying inland extent of water impacts, and by the specific topography/ drainage landward of the barrier. However, guidance in Table 7 is provided as potentially applicable to typical coastal situations.

For each coastal structure experiencing sizable wave runup in the base flood (say, $\overline{R} > 2$ ft), a brief report to the Project Officer should outline overtopping assessments, and document conclusions consistent with historical evidence for the site.

Table 7. Suggestions for Interpretation of Mean Wave Overtopping Rates

$\overline{Q}$ Order of Magnitude	Flood Hazard 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 structure width, incident wave period or wavelength, and other factors.

6.0 ANALYSIS OF OVERLAND WAVE DIMENSIONS

As water waves propagate near the shore and over flooded land, they can undergo marked transformations due to local winds, interaction with the bottom, and physical features such as buildings, trees, or marsh grass. Figure 21 illustrates schematic effects on the wave crest elevations and on the type of flood zone. The fundamental analysis of wave effects for an FIS is provided by a computer program (Reference 13) entitled "Wave Height Analysis for Flood Insurance Studies" (WHAFIS 3.0). This program or model calculates wave heights, wave crest elevations, flood hazard zone designations, and the location of zone boundaries along a transect.

Wave description for an FIS addresses the controlling wave height, equal to 1.6 times the significant wave height common as a representative wave description. Significant wave height is the average height of the highest one-third of waves, and controlling wave height is approximately the average height of the highest one percent of waves in storm conditions. The original basis for FIS wave treatment was the NAS methodology which accounted for varying fetch lengths, barriers to wave transmission, and the regeneration of waves over flooded land areas (Reference 4). Since the introduction of the NAS methodology there have been periodic upgrades to incorporate improved or additional wave considerations.

Technical details of the current model are fully documented in Reference 13, but a brief overview indicates the level of wave treatment in WHAFIS 3.0. A wave action conservation equation governs wave regeneration due to

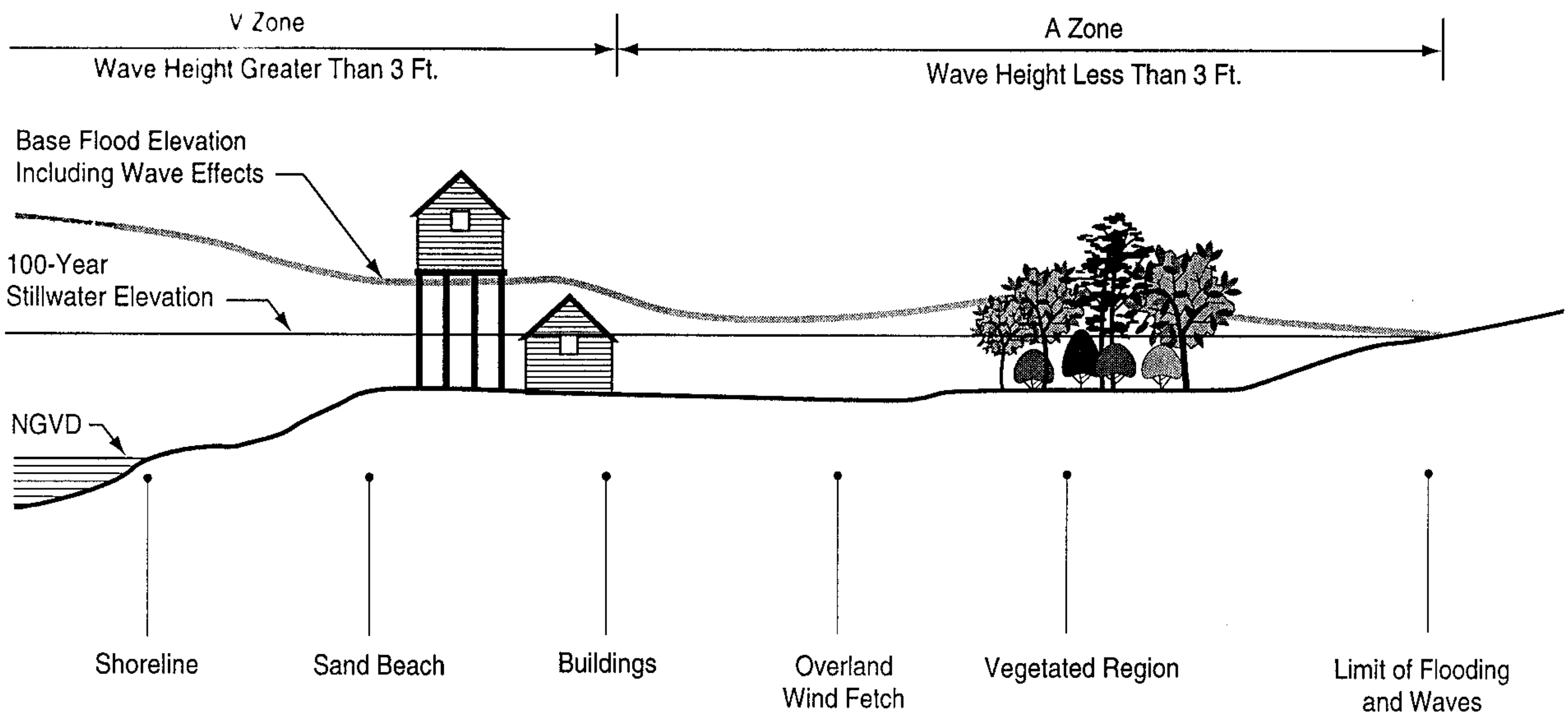

Figure 21. Schematic Wave Effects on A Transect.

wind and wave dissipation by marsh plants. 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 transect. A predominant element in this wave treatment remains unchanged from the NAS methodology: controlling wave height is limited to 78 percent of the local mean water depth.

6.1 Use of WHAFIS 3.0 Model

This computer program usually resides on the hard-disk drive of a PC. Careful preparation and input of required site data are necessary in using WHAFIS. Like the other coastal treatments, the WHAFIS model considers the study area by representative transects. For WHAFIS, transects should be selected considering major topographic, vegetative, and cultural features. The ground profile is defined by elevations referenced to NGVD, usually begins at elevation 0.0, and proceeds landward until either the ground elevation exceeds the meanwater elevation for the base flood, or another flooding source is encountered.

Other fundamental specifications among WHAFIS input include the 100-year mean water elevation and a description of waves existing at the transect start. As the wave description, provision is made for an overwater fetch length, an initial significant wave height, or an initial period of dominant waves. In most applications, the wave period should be the input description, since that parameter is readily available from information about offshore storm waves and the period does not change during most wave transformations. WHAFIS will then compute an appropriate depth-limited wave height at the transect start. The only check necessary is to confirm that incident waves likely exceed that height and a wave condition limited by water depth occurs.

Different wave specifications can be appropriate for sites not on an open, straight coast. Where land shelter or wave refraction may result in reduced incident waves, it is appropriate to specify an initial significant wave height for the transect. Also, at sites on restricted water bodies, the overwater fetch length should be specified for likely wind direction at the flood peak. WHAFIS will then compute an appropriate incident wave condition for the transect, but note that such waves are limited and any fetch length exceeding 24 miles will yield the same results.

In preparing WHAFIS input, transects should be located on the work maps and the transect ground profile plotted from the topographic data, adjusted for erosion. Each transect should have all the input data identified on the profile plot for ease of input coding. The location, height, and extent of elongated manmade structures should also be identified and shown as part of the ground profile, after confirming the structure's stability under forces of the base flood (Chapter 3). When locating transects across barrier islands or sand spits, common practice is to continue the transect across the back bay 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 parallel the transects from the open coast, and they may cross one another. Crossing transects should be kept to a minimum, but where it is not possible to avoid this, the transect determining greatest flood hazards should control in mapping the flood hazards.

Once representative transects are located, the local 100-year mean water levels can be defined for WHAFIS input. Reference 13 specifies that wave setup should be included in this water elevation, as a part of the appropriate mean depth controlling wave dimensions. If wave setup was not calculated separately for the site, 100-year stillwater elevation is the appropriate specification. WHAFIS also has an input field for a 10-year stillwater elevation, although it is only employed to determine flood hazard factors which are no longer used. Still, this input should be provided if it is readily available, since it could help in distinguishing between transects.

When a transect covers two or more flooding sources, an area of transition between the different stillwater elevations must be identified. This is a common situation for barrier islands with ocean elevations on one side and bay elevations on the other side. It is usually assumed that the higher ocean elevations extend inland to the highest point of the reduced ground profile. WHAFIS performs a linear interpolation within a transect segment where elevations differ at the end stations. The interpolated elevations are compared to the ground elevations and adjusted, if necessary, to be above the ground elevations. A stillwater elevation may have to be input a second time to identify areas of constant elevation and elevation transition.

The proper transect representation of some land features, particularly buildings and vegetation, merits further discussion. Buildings are specified on the transect as rows perpendicular to the transect. Since buildings are not always situated in perfect rows, judgment must be exercised 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. It should be examined whether the first row or two of buildings along the shoreline should be considered as obstructions. During a 100-year event, it is sometimes appropriate to assume that these buildings will be destroyed before the peak of the flood occurs if they are not elevated on pilings. If they are elevated, the waves should propagate under the structure with minimal reduction in height. It is useful to contact local officials to obtain typical construction methods and the lowest elevations of structures.

The WHAFIS program has two separate routines for vegetation: one for rigid vegetation that can be represented by an equivalent "stand" of equally spaced circular cylinders (Reference 5), and one for marsh vegetation that is flexible and oscillates with wave action (Reference 40). For either type, considerable care is required in selecting representative parameters and in ruling out that the vegetation will be intentionally removed or that effects would be markedly reduced during a storm through erosion, uprooting, or breakage.

For the areas of rigid vegetation located on the transect, the required input values are the drag coefficient, C_D; mean wetted height, h; mean effective diameter, D; and mean horizontal spacing, b. The value of C_D 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, a value of 0.35 should be used. Representative values for h, D, and b can be obtained from stereoscopic aerial photographs or by field surveys. Various guides for terrain analysis can provide advice on estimating values from aerial photographs. Table 8 provides a useful process developed from Terrain Analysis Procedural Guide for Vegetation (Reference 41).

Table 8.

Procedures for Vegetation Analysis

Using Stereoscopic Aerial Photographs

1. Using the parallax bar or wedge, determine the height of three representative trees and compute the average height, h.

2. Locate three representative tree crowns, measure the diameters, and compute the average crown diameter, CD.

3. Determine the type of vegetation and calculate the stem diameter, D, using the following formulae:

Southern Pines	D (inches) = 5 + 0.5 CD (feet)
Eastern Hardwoods,	
Northern Pines and Others	D (inches) = 0.75 CD (feet)

4. Based on the scale of the aerial photographs, determine the diameter of a circle containing 0.08 hectares using Table 8a. Place the circle on the photograph, over a representative area of trees, and count the number of trees, n, in the circle. A magnifier may be needed. More than one area can be counted and an average used for n. Calculate the number of trees per hectare, N, using the following formula:

$$N = \frac{n}{0.08}$$

5. Determine the horizontal spacing between trees using the following formula:

$$b\ (feet) = (\frac{12732}{N} - \frac{D\ (inches)}{12}$$

Table 8a.

CIRCLE DIAMETERS
.08 HECTARE AREA, (1/5 ACRE)
(800 Square Meters, 8712 Square Feet)

PHOTO SCALE	.08 HECTARE CIRCLE	CIRCLE DIAMETER	
		INCHES	MILLIMETERS
1:5,000		.253	6.38
1:6,000		.211	5.32
1:7,000		.1805	4.56
1:8,000		.158	3.99
1:9,000		.140	3.55
1:10,000		.126	3.192
1:11,000		.115	2.90
1:12,000		.105	2.66
1:13,000		.092	2.46
1:14,000		.090	2.28
1:15,000		.084	2.13
1:16,000		.079	1.99
1:17,000		.074	1.88
1:18,000		.070	1.77
1:19,000		.067	1.68
1:20,000		.063	1.60
1:21,000		.060	1.52
1:22,000		.057	1.45
1:23,000		.055	1.39
1:24,000		.053	1.33
1:25,000		.051	1.28

For marsh vegetation, a more complicated specification is required for completeness, and the eight parameters used to describe dissipational properties of a specific type are explained in Table 9. However, WHAFIS incorporates considerable basic information on the eight common types of seacoast marsh plants listed in Table 10 (Reference 40). That information can be utilized either by specifying the Table 10 abbreviation, or a geographical region as indicated in Figure 22. Figure 22 shows the coastal wetland regions of the Atlantic and Gulf coasts, along with the identifying number used in WHAFIS. If the site is near a region 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 usage in spatial interpolation.

Climate affects the geographic range of each marsh plant type, so that some plant types are not found in all regions. Table 11 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 12 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 percent of coverage may be specified. Given this information, WHAFIS will supply default values for the other marsh plant parameters appropriate to the site (see Reference 40).

Table 9. 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	Mid stem 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 10. Abbreviations of Marsh Plant Types used in WHAFIS

SPECIES OR SUB-SPECIES	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

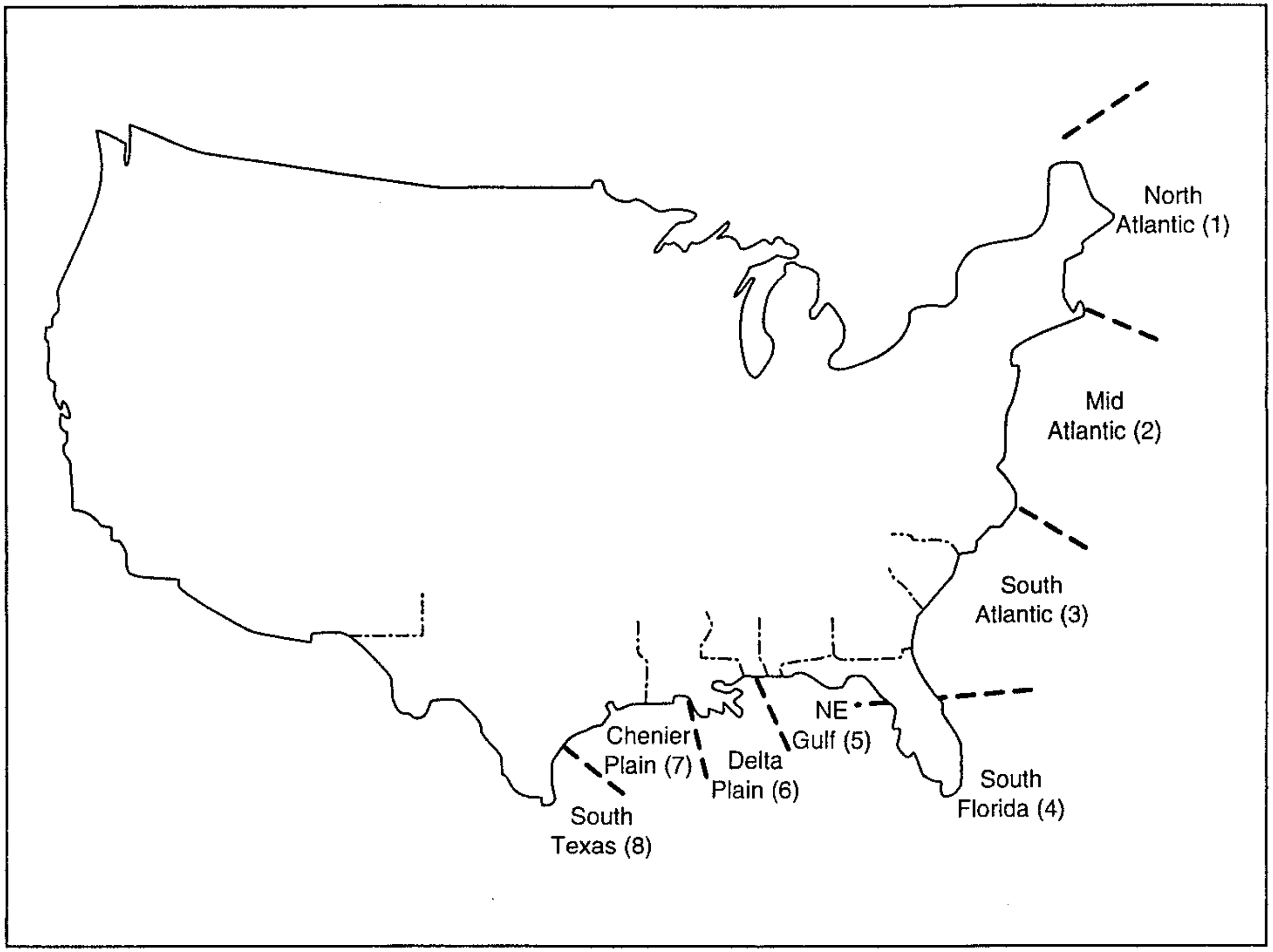

Figure 22. Coastal wetland regions of Atlantic and Gulf coasts having enough marsh grass to significantly affect wave heights. Region numbers are indicated in parentheses.

Table 11. 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	--- ---

[1]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).

---Indicates that there are insignificant amounts of marsh plants within the given habitat in the region.

Table 12. "Significant" Marsh Plant Types in Each Seacoast Region and Default Regional Plant Parameter Data Used in WHAFIS

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

Symbols in parentheses indicate source of data in vertical triplet:

1 = Reference 42
2 = Reference 43
3 = Reference 44
4 = Reference 45, Diameters extrapolated
5 = Reference 46, Diameters extrapolated
\+ = Extrapolated Data
--- = There are insignificant amounts of this plant type in the region

Where detailed specification of marsh plants is required, some basic considerations can aid in the determination of likely types. Each marsh plant type may be associated with certain climatic conditions and certain ranges of salinity, so consultation of Figure 22 and Table 11 can be helpful. In addition, it is useful to realize that salinity levels are related to elevations above mean tide level. Figure 23 gives the salinity tolerance of marsh plants, and Figure 24 gives the preferred tidal elevation range. Care should be taken in interpreting Figure 24 because where the local tide range is less than one foot, the boundaries between species can deviate somewhat from those shown. Furthermore, in regions such as the Northeastern Gulf, where S. alterniflora does not occur in significant amounts, both Juncus species and Distichlis spicata can grow down to the mean tide level. Some plant types are usually found together, for example, tall and medium varieties of S. alterniflora. Typically, 20 to 25 percent of S. alterniflora can be characterized as tall, with the tall variety usually found adjacent to tidal creeks.

Following the identification of the marsh plant types present, the area and fraction of coverage, F_{cov}, for each plant type must be calculated. For each transect, the total area of marsh vegetation coverage is determined. 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

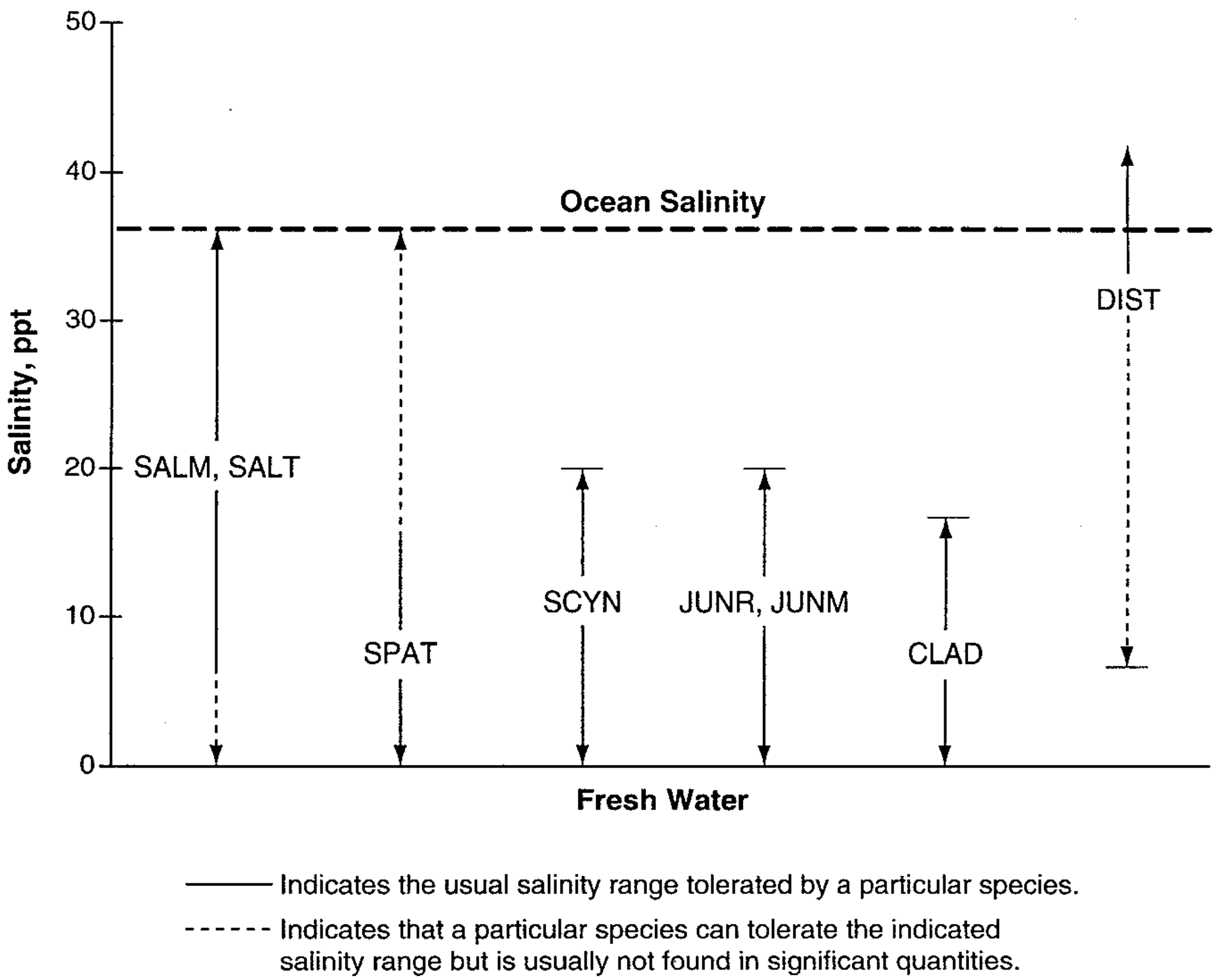

Figure 23. Salinity Tolerance of Marsh Plants, from Reference 47

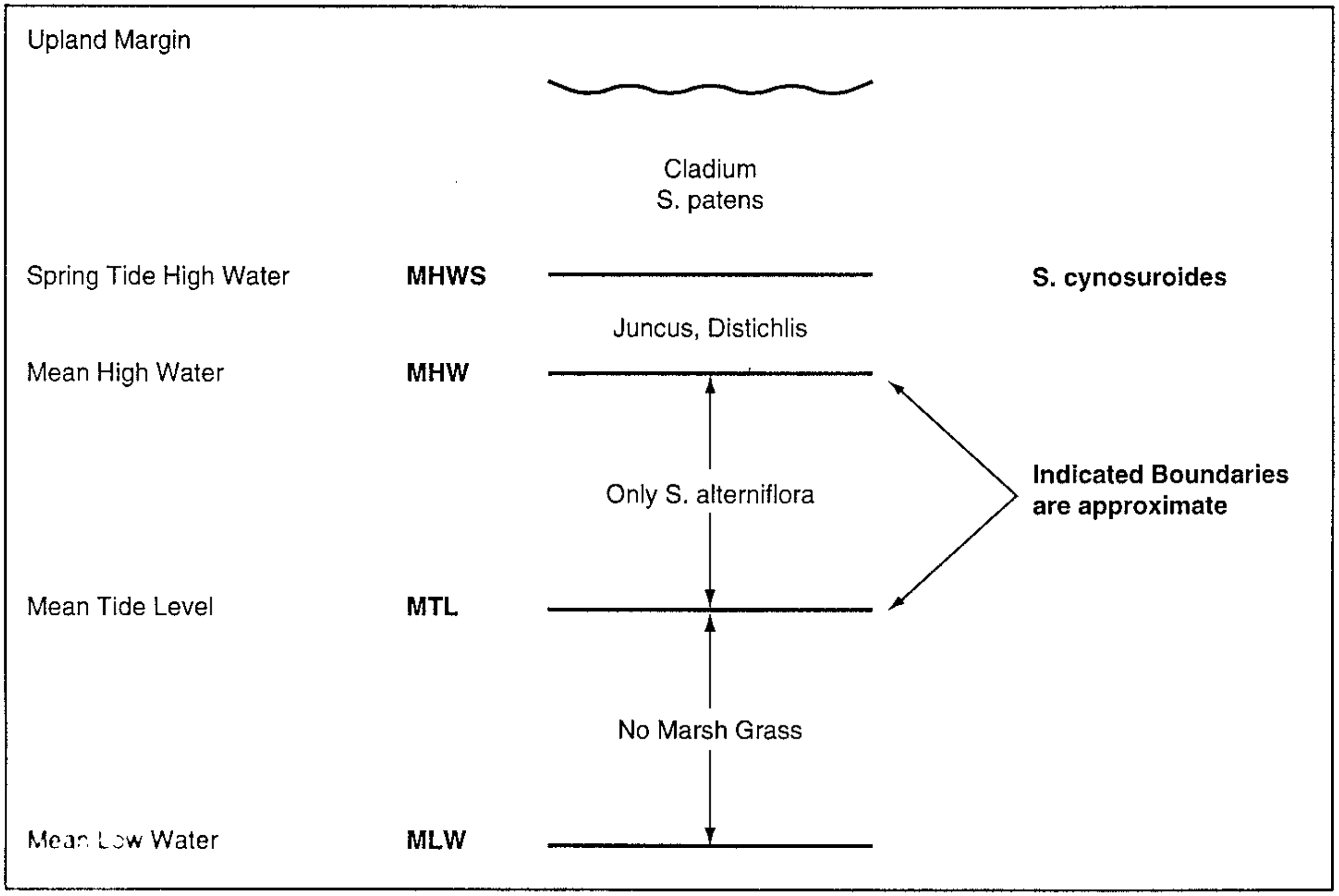

Figure 24. Tidal Control on Salt Marsh Plant Viability

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.

6.2 Input Coding for WHAFIS

After all the necessary input data have been identified on the transect, the transect should be divided 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. Fetches should be subdivided at points where the ground elevation abruptly changes and in the transition area of changing stillwater elevations. Obstructions should be subdivided into smaller segments at the transect's seaward edge to more accurately model the wave dissipation. Rigid vegetation should have two to three seaward segments extending 10 to 50 feet, and the first two or three rows of buildings should have a segment for each row. Marsh vegetation will be subdivided within WHAFIS so segmented input is not necessary.

The necessary data are entered via an input file created using 10 line types. Each line describes a certain type of fetch or obstruction. The IE (Initial Elevation) line describes the initial overwater fetch and the initial stillwater elevations. The IF (Inland Fetch) and OF (Overwater Fetch) lines define the end point stationing and elevation of inland and overwater fetches, respectively. Obstructions are categorized as either buildings (BU line), rigid vegetation (VE line), marsh vegetation (VH and MG lines), dunes and other natural or man-made elongated barriers (DU line), or areas where the ground elevation is greater than the 100-year stillwater elevation (AS line). The tenth line type, the ET (End of Transect) line, enters 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 ten numeric fields describing the parameters.

To ensure proper modeling, all segments of each transect must be entered as either fetches or obstructions, with one input line required 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 1 field of 6 columns followed by 9 fields of 8 columns. The numbers in any data field need to be right-justified only if no decimal point is used, and decimal points are permitted but not required. The end point 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 end point. The last two fields used on each line are for entering new stillwater elevations. An interpolation is performed within a transect segment starting at the closest station with an input stillwater elevation. This interpolation uses the new stillwater elevation input at the end point of the segment and the stillwater elevation input at a previous segment. If these fields are blank or zero, the stillwater elevations 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 input line. This line is used to begin a transect at the shoreline and 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 end point of initial overwater fetch in feet (zero at beginning of transect)
2	9-16	Ground elevation at end point 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-year stillwater elevation
5	33-40	100-year stillwater elevation
6	41-48	Initial wave height; 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 end point of an area with ground elevation greater than the 100-year stillwater elevation (such as a high dune or other land mass). This is used when the ground surface temporarily rises above the 100-year stillwater elevation. The line immediately preceding the AS line must enter the stationing and elevation of the point at which the ground elevation first equals the 100-year stillwater elevation. Stillwater elevation on the inland side may differ from stillwater elevation on the seaward side. The ground elevation entered on the AS line must equal the stillwater elevation which applies to the inland side of the land mass. Computer calculations will be terminated if a ground elevation greater than the 100-year stillwater elevation is encountered.

Data Field	Columns	Contents of Data Fields
0	1-2	AS
1	3-8	Stationing at end point of area above 100-year stillwater elevation
2	9-16	Ground elevation at end point
3	17-24	A blank or zero indicates no change to the 10-year stillwater elevation; otherwise new 10-year stillwater elevation
4	25-32	A blank or zero indicates no change to the 100-year stillwater elevation; otherwise new 100-year stillwater elevation
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 end point of group of buildings
2	9-16	Ground elevation at end point
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-year stillwater elevation; otherwise new 10-year stillwater elevation
6	41-48	A blank or zero indicates no change to 100-year stillwater elevation; otherwise new 100-year stillwater elevation
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 (e.g., levees, seawalls).

Data Field	Columns	Contents of Data Fields
0	1-2	DU
1	3-8	Stationing at top of dune or barrier
2	9-16	Elevation at top of dune or barrier
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-year stillwater elevation; otherwise new 10-year stillwater elevation
5	33-40	A blank or zero indicates no change to 100-year stillwater elevation; otherwise new 100-year stillwater elevation
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 wind speed of 60 mph.

Data Field	Columns	Contents of Data Fields
0	1-2	IF
1	3-8	Stationing at end point of fetch
2	9-16	Ground elevation at end point
3	17-24	A blank or zero indicates no change to 10-year stillwater elevation; otherwise new 10-year stillwater elevation
4	25-32	A blank or zero indicates no change to 100-year stillwater elevation; otherwise new 100-year stillwater elevation
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 (i.e., large lakes, bays) using a sustained wind speed of 80 mph. If an inland waterbody is sheltered and has a depth of ten feet or less, the IF line calling for reduced wind speed should be used.

Data Field	Columns	Contents of Data Fields
0	1-2	OF
1	3-8	Stationing at end point of fetch
2	9-16	Ground elevation at end point
3	17-24	A blank or zero indicates no change to the 10-year stillwater elevation; otherwise new 10-year stillwater elevation
4	25-32	A blank or zero indicates no change to 100-year stillwater elevation; otherwise new 100-year stillwater elevation
5-10	33-80	Not used

VE Line (Vegetation)

This line enters parameters necessary to compute wave dissipation due to rigid vegetation stands.

Data Field	Columns	Contents of Data Fields
0	1-2	VE
1	3-8	Stationing at end point of vegetation
2	9-16	Ground elevation at end point
3	17-24	Mean effective diameter of equivalent circular cylinder
4	25-32	Average actual height of vegetation
5	33-40	Average horizontal spacing between plants
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-year stillwater elevation; otherwise new 10-year stillwater elevation
8	57-64	A blank or zero indicates no change to 100-year stillwater elevation; otherwise new 100-year stillwater elevation
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.

Data Field	Columns	Contents of Data Fields
0	1-2	VH
1	3-8	Stationing at end point of marsh vegetation segment
2	9-16	Ground elevation at end point
3	17-24	Reg_p, number of the primary seacoast region for default plant parameters. See Figure 22.
4	25-32	Wt_p, weighting factor for the primary seacoast region.
5	33-40	Reg_s, number of secondary seacoast region. See Figure 22.
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-year stillwater elevation; otherwise new 10-year stillwater elevation
8	57-64	A blank or zero indicates no change to the 100-year stillwater elevation; otherwise new 100-year stillwater elevation
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 10, some potentially useful default values are supplied in Table 12, and program can provide additional default values (Reference 40). If a plant type not listed in the table is used, then appropriate data must be developed for Fields 2-9.

Data Field	Columns	Contents of Data Fields
0	1-2	MG
1	5-8	Marsh plant type abbreviation (see Table 10)
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, mid stem 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 input 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

6.3 Error Messages

- "AS card ground elevation less than stillwater elevation, should use other type card, job dumped."

  Only use AS (above surge) line when the ground elevation is above the stillwater elevation. Can 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, 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 stillwater elevation and the ground elevation if 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 within 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."

  The transect profile should be coded up to the inland limit where ground elevation intersects the stillwater elevation so that wave height should decrease to zero. If the scope of work ends at the corporate limits before the ground elevation meets the stillwater elevation, this message can be ignored.

- "Array dimensions exceeded. Job dumped."

  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," etc.

  Check input data to make sure that data are in 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."

- "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."

  A 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 interpolated 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 100-year 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 stillwater elevation 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.

6.4 Output Description

The output of the program provides all the data necessary for plotting the BFEs and flood hazard zones along the transect. Examples are presented within Appendix A. The output is in six parts:

PART 1 - INPUT: This part 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 showing any default values supplied by the computer.

PART 2 - CONTROLLING WAVE HEIGHTS, SPECTRAL PEAK WAVE PERIOD, AND WAVE CREST ELEVATIONS: This is a listing of the calculated controlling wave heights, spectral wave peak periods, and wave crest elevations at the end point of each fetch and obstruction of the input, and at calculation points generated between the input stations.

PART 3 - LOCATION OF AREAS ABOVE 100-YEAR SURGE: This is a listing of the locations of areas where the ground elevation is greater than the 100-year stillwater (surge) elevation. Only areas identified by AS lines are listed.

PART 4 - LOCATION OF SURGE ELEVATIONS: This is a listing of the 10- and 100-year stillwater (surge) elevations and the stationing of the points where each set of stillwater elevations first becomes <u>fully</u> effective.

PART 5 - LOCATION OF V ZONES: This is a listing of the locations of the V/A zone boundary and 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 listing 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 given 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 (Reference 1). When the same zone and elevation are repeated in the listing, they should be treated as a single zone.

7.0 MAPPING OF FLOOD ELEVATIONS AND ZONES

7.1 Review and Evaluation of Basic Results

Prior to mapping the flood elevations and zones, the results from the models and assessments should be reviewed from a common-sense viewpoint and compared to available historical data. When utilizing these models there is the potential to forget that the transects represent real shorelines of sandy beaches, rocky or cohesive bluffs, wetlands, etc., being subjected to extremely high water, waves, and winds. Familiarity and experience with the coastal area being modeled or similar areas should provide an idea of what is a "reasonable" result.

Use of the historical data is also very important in evaluating whether the results are reasonable. It would be very convenient if data from a storm closely approximating the 100-year event were available, but this is seldom the case. Although most historical flood data are for storms less intense than a 100-year event, these data will still indicate, at a minimum, what areas should be in flood zones. For instance, if a storm that produced an extreme flood below the 100-year stillwater elevation generally caused structural damage to houses 100 feet from the shoreline, a "reasonable" Zone VE width must be at least 100 feet. Similarly, houses that collected flood insurance claims for the same storm should be at least in a Zone AE, AH, or AO. If the analyses of the 100-year 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 can be that a new coastal structure acts to reduce flood hazards locally.

If there are indications that a reevaluation is needed, it should be determined whether the results of the erosion assessment are appropriate. An attempt should be made to compare the eroded profile to past effects, whether in the form of profiles, photographs, or simply descriptions. A general idea of what happened previously can be sufficient. Judgment and experience must be used to project previous storm effects to the 100-year conditions, and to ensure that the eroded profile is consistent with previous events.

The other data input to the assessments of wave effects should also be examined. This includes checking that the stillwater elevations, wave heights, wave periods, and fetch lengths were used correctly and are consistent with the historical data. Further consideration might be given to examining if the buildings or structures modeled would be destroyed by the storm or if the buildings are on pilings above the flooding.

The main point to be emphasized here 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.

7.2 Identification of Flood Hazard Zones

The flood zones and base flood elevations (BFEs) including wave heights should be identified on each transect plot before delineating zones on the work maps, because of additional wave effects along with the 1988 redefinition of Coastal High Hazard Area to include the primary frontal dune. The existing topography, the eroded transect, the combination of shore effects in the wave envelope, and other results from wave overtopping assessment are all important to the proper identification of flood hazard zones.

Specifically, the existing ground profile defines an appropriate extent of the primary frontal dune, as a ridge of sand bounded by relatively steep slopes (Section 1.2). The eroded transect for cases of duneface retreat may imply flood hazards due to wave overtopping into an area landward of WHAFIS results (Section 4.5). In addition, wave overtopping of stable shore barriers can result in flooding to areas above the mean elevation of wave runup (Section 5.7). However, the main consideration for integrated treatment of wave-controlled flood elevations is to define the wave envelope joining height and runup effects.

This wave envelope is a combination of representative wave runup elevation with the controlling wave crest profile determined by WHAFIS. The wave crest profile is plotted on the transect from the data in Part 2 of the WHAFIS output. A horizontal line is extended seaward from the wave runup elevation to its intersection with the wave crest profile to obtain the wave envelope, as shown in Figure 25. If the runup elevation is greater than the maximum wave crest elevation, the wave envelope will be a horizontal line at the runup elevation. Conversely, if the wave runup is negligible or was not modeled, the wave crest profile becomes the wave envelope.

Flood hazard zones are defined basically by the wave envelope along with the general zone descriptions in Table 13. Those results are supplemented by runup and overtopping considerations, as introduced previously. The following material outlines the process of zone identification, with specific examples presented in the next section to illustrate some usual results.

The first step in identifying the flood zones on the transect is locating the inland extent of the VE zone, also known as the VE/AE boundary. The VE zone limit for each of the three criteria is identified, and the VE/AE boundary placed at the one furthest landward, as shown in Figure 26. That boundary may need to be moved further inland in the vicinity of a wave barrier where severe overtopping is indicated for the base flood, so high velocity impacts occur over a limited landward area.

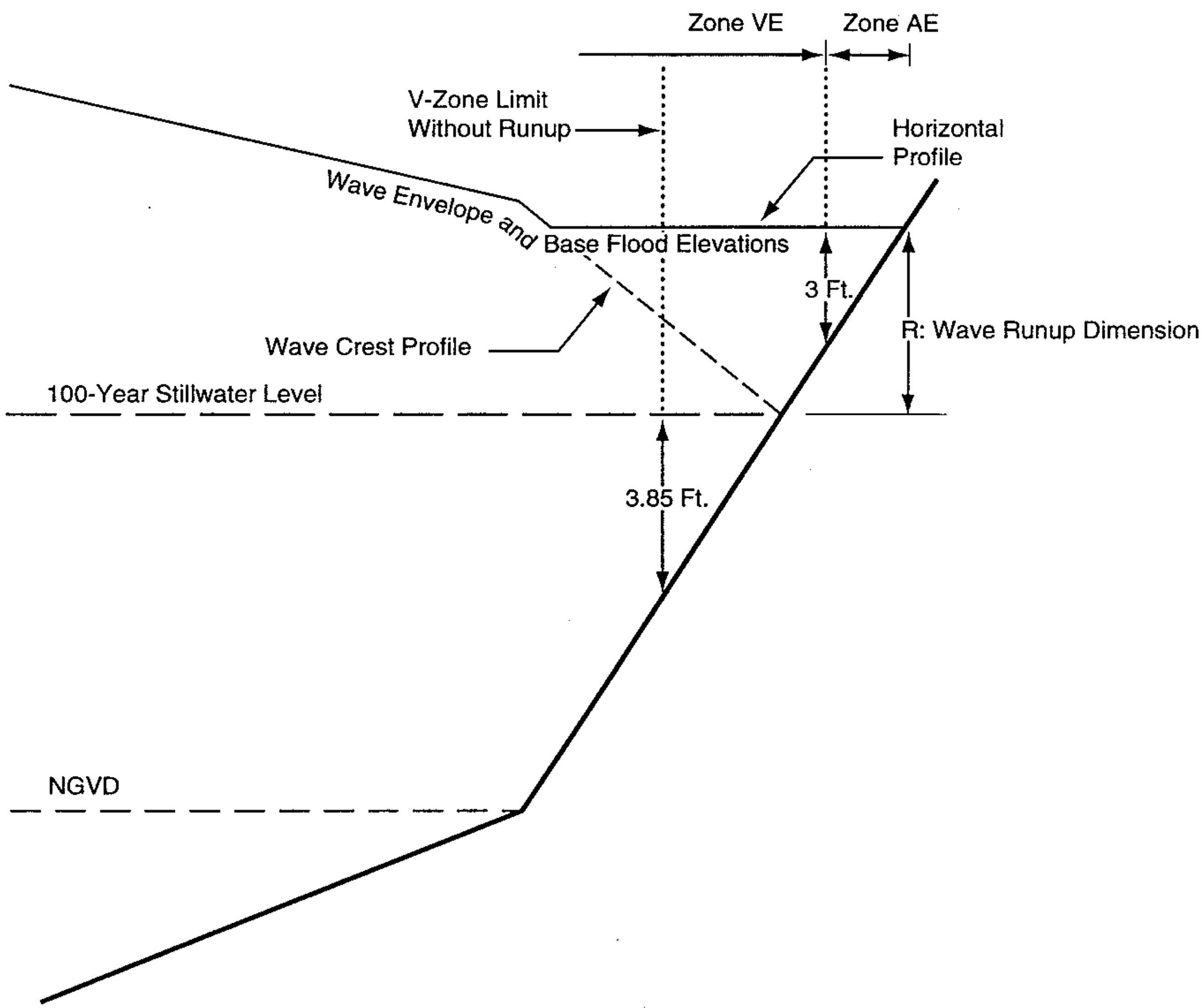

Figure 25. Wave Envelope Resulting from Combination of Nearshore Crest Elevations and Shore Runup Elevation.

Table 13. Description of Coastal Flood Zones.

Zone VE	Coastal High Hazard Areas where wave action and/or high velocity water can cause structural damage in the 100-year flood. Primarily identified by: (1) the area where 3 foot or greater wave height could occur (this is the area where the WHAFIS wave crest profile is 2.1 feet or more above the stillwater elevation), (2) the area where the eroded ground profile is 3 feet or more below the representative runup elevation, and (3) the entire primary frontal dune, by definition. Subdivided into elevation zones with BFEs assigned.
Zone AE	Areas of inundation by the 100-year flood, including wave heights less than 3 feet and runup elevations less than 3 feet above the ground. Also subdivided into elevation zones with BFEs assigned.
Zone AH	Areas of shallow flooding or ponding, with water depth equal to 3 feet or less. Usually not subdivided, but a BFE is assigned.
Zone AO	Areas of "sheet-flow" shallow flooding where overtopping water flows into another flooding source. Assigned with 1-, 2-, or 3-foot depth of flooding.
Zone X	Areas above 100-year flood inundation. On the FIRM, shaded Zone X is inundated by the 500-year flood, unshaded Zone X is above 500-year flood.

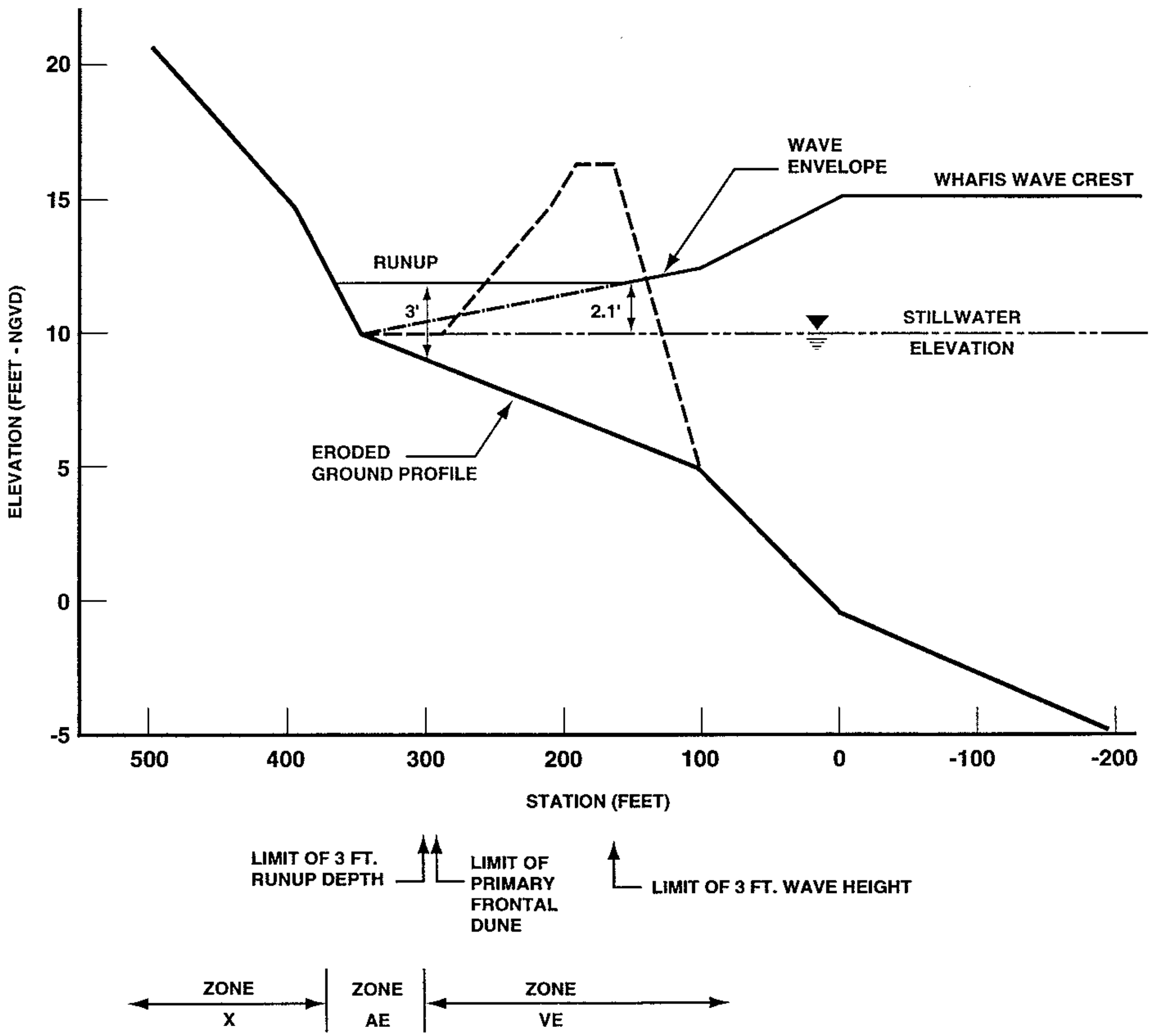

Figure 26. Possible V-Zone Limits at Eroded Dune.

The AE zone will extend from the VE/AE boundary to the inland limit of 100-year inundation, which is a ground elevation equal to the representative runup elevation, or the 100-year stillwater elevation if runup is negligible. Additional areas of shallow flooding or ponding for the 100-year event may be designated as Zone AH or Zone AO. In cases of severe wave overtopping impacts, a Zone VE may abut areas designated as Zone AH or Zone AO. All areas above the 100-year inundation are Zone X.

The AE and VE zones are then subdivided into elevation zones with whole-foot BFEs assigned according to the wave envelope. Ideally, there would be an elevation zone for every BFE in the wave envelope, but because these zones are mapped on the FIRM so that buildings or property can be located in a flood zone, a minimum width must be used for the mapped zone to provide a usable FIRM. For coastal areas, the minimum zone width is 0.2 inch on the FIRM. For identifying elevation zones on the transect, the minimum width is 0.2 times the final FIRM scale; for example, a width of 80 feet for a FIRM at 1 inch equals 400 feet, or a width of 100 feet for a FIRM at 1 inch equals 500 feet.

The horizontal runup portion of the wave envelope, if any, does not need to be subdivided; the runup elevation, rounded to the nearest whole foot, is the BFE. It is the WHAFIS wave crest profile that requires subdivision. Generally, the VE zone is subdivided first. Initially mark on the transect the location of all the elevation zone boundaries. Since whole-foot BFEs are being used, these should always be at the location of the half-foot elevation on the wave envelope.

The elevation zones that do not meet the minimum width should be combined with an adjacent zone or zones to yield an elevation zone wider than the minimum. The BFE for this combined zone is a weighted average of the combined zones. Often in subdividing VE zones, the maximum BFE zone is located just inside the mapped shoreline, and the remainder of the VE zone is then subdivided into elevation zones of the minimum width.

The AE zone, if wide enough, is subdivided in the same manner. If the total AE zone is less than the minimum width, the lowest elevation VE zone 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 to extend the AE zone slightly into the next zone seaward in order to satisfy the minimum width requirement.

Relatively low areas inland of zones assigned wave elevations may be subject to shallow flooding or ponding of flood water and designated as AH or AO Zone. Such designations can be relatively common landward of coastal structures and dunes, where wave overtopping occurs. Identifying appropriate zones and elevations may require particular care for dunes, given that the entire primary frontal dune is defined as Coastal High Hazard Area. Although the analyses may have determined a dune will not completely erode and wave action should stop at the retreated duneface with only overtopping possibly propagating inland, the entire dune is still designated as a VE zone. The BFE at the duneface is assigned for the remainder of the dune.

It may seem unusual to use a BFE that is lower than the ground elevation, although this is actually 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; under current regulations, it requires structures to be built on pilings and prohibits alterations to the dune.

7.3 Transect Examples

Figure 26 provided a schematic summary for the three criteria potentially defining the landward limit to the Coastal High Hazard Area. The following examples depict idealized transects of typical types in order to illustrate common flood hazard zonations in a quantitative way. Coastal erosion is a dominant consideration for the first set of cases, and the second set addresses some usual effects at stable shore barriers exposed to extreme wave action.

Figure 27 presents an example of dune removal with appreciable runup occurring on the eroded profile. For this transect, the VE zones with BFEs of 13, 14, and 15 feet are too narrow to be mapped, so they are averaged to a BFE of 14 feet. The Zone VE, elevation 12 feet, is enlarged slightly to include some of the elevation 13-foot area so that the boundary would be located at the dune toe or 5-foot contour line, a feature easily identified on the work maps. The boundary between the Zones VE, elevation 14 feet and elevation 16 feet, is located just landward of the shoreline. The Zone AE, elevation 12 feet, in Figure 27 is only 70 feet wide, slightly less than the minimum mapping width. In this case, the work maps should be examined to determine if this zone might be wider or narrower in the contiguous area. If wider, the Zone AE should be used; if narrower, the designation extended through this area should be Zone VE, elevation 12 feet.

Figure 28 is an example of a relatively high retreated duneface. A mean runup elevation of 13 feet is calculated for the eroded duneface. This elevation is assigned through the dune, all of which is designated as a VE zone. Because the dune remnant extends more than 7 feet above stillwater elevation, no flooding landward of the dune is indicated by designating a Zone X. Note that the retreated dune profile shifts the 0.0 foot elevation shoreline 65 feet seaward. Because the work maps use the existing 0.0 foot elevation shoreline, the Zone VE, elevation 16 feet, is located just landward of the existing shoreline.

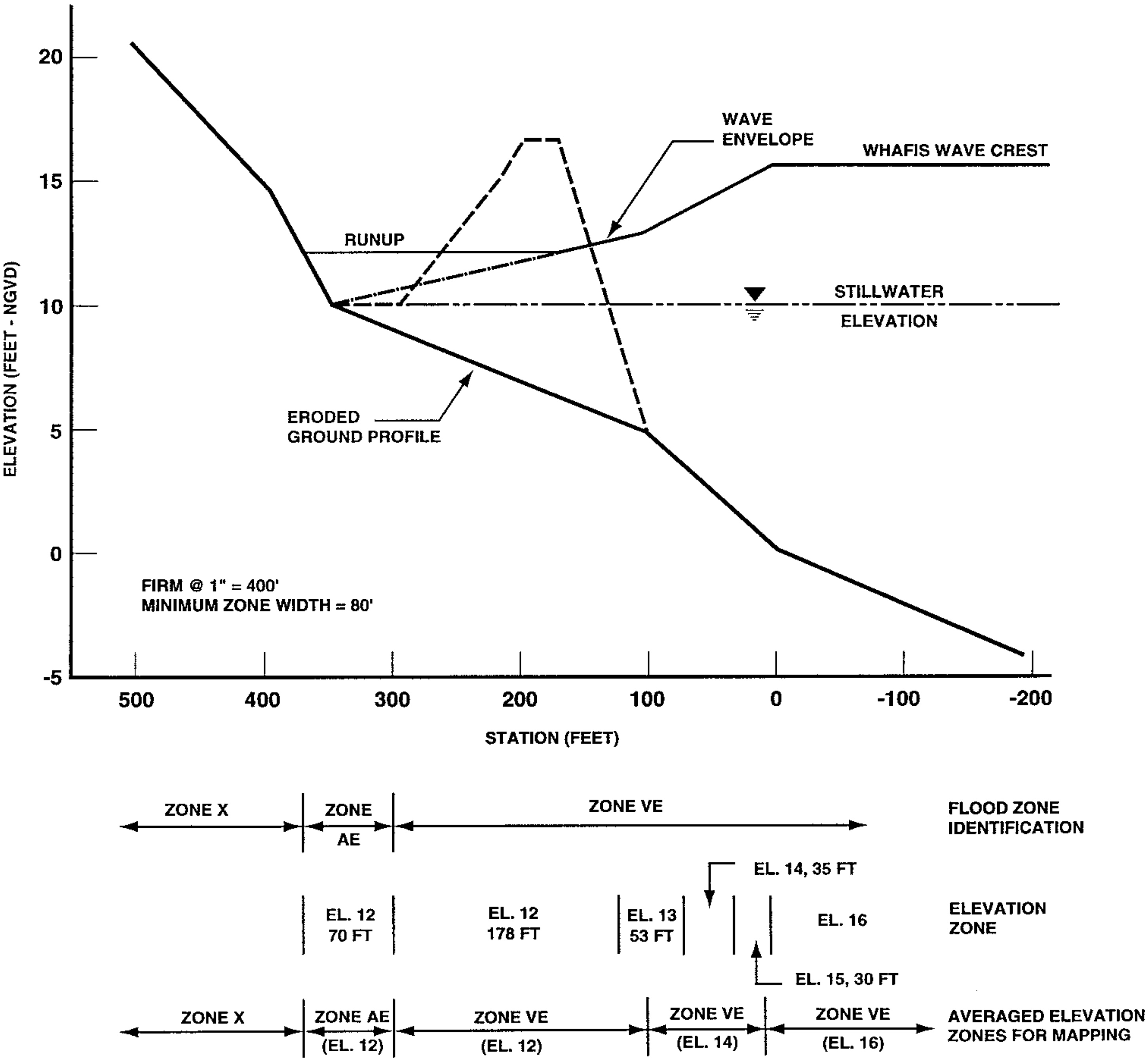

**Figure 27. Identification of Flood Zones, Example 1:
Dune Removal with Wave Runup Landward.**

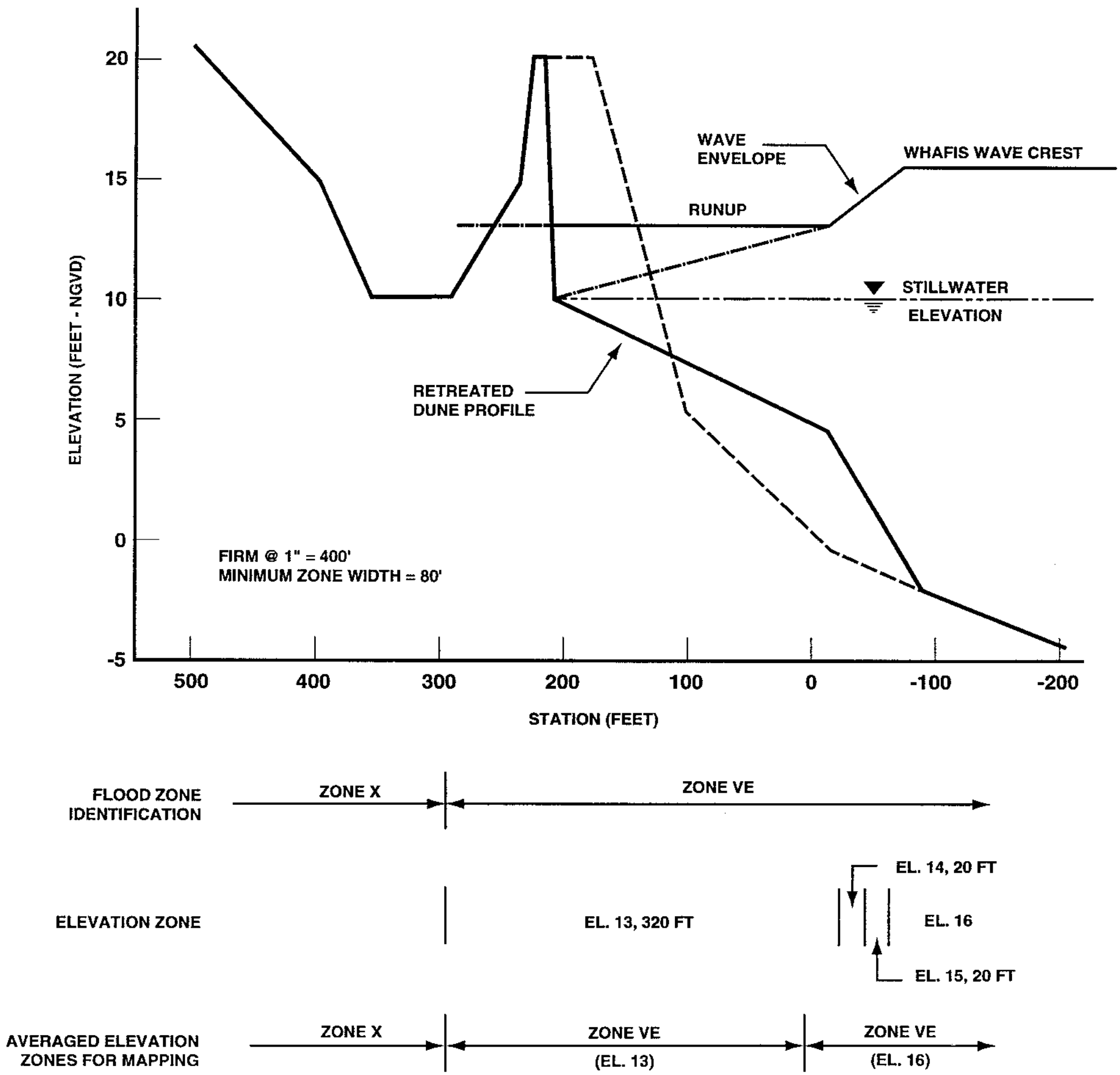

Figure 28. Identification of Flood Zones, Example 2: Duneface Retreat with Relatively High Remnant.

Figure 29 provides an example of a retreated duneface with a relatively small remnant having low relief. A mean runup elevation of 12 feet is calculated for the eroded profile and this flood elevation is assigned through the dune, all of which is designated as a VE zone. The division into separate map zones is similar to Figure 28. Because the dune remnant extends less than 7 feet above stillwater elevation, appreciable wave overtopping is expected during the base flood. An area landward of the dune of about the minimum mapping width is designated as a Zone AO, depth 1 foot.

Figure 30 is an example of dune removal where there is some runup and overtopping of the remaining stub. As in Figure 27, the VE zone with the runup elevation of 11 feet is extended to the dune toe and the Zone VE, elevation 16 feet, is located just landward of the shoreline. Although elevation 14 feet is shown on Figure 30 for the intermediate VE zone, elevation 13 feet could also be used; adjacent transects should be examined and a compatible BFE selected. Also note that the boundary between the Zone AO, depth 1 foot, and the Zone AE, elevation 7 feet, is at the intersection of the stillwater elevation and ground profile.

An eroded bluff is shown in Figure 31. The angle of the bluff face remains the same while the seaward extension from the toe is a 1 on 40 slope. The computed runup elevation slightly exceeds the bluff crest and is higher than the maximum wave crest elevation. The area is designated Zone VE, elevation 18 feet, until the difference

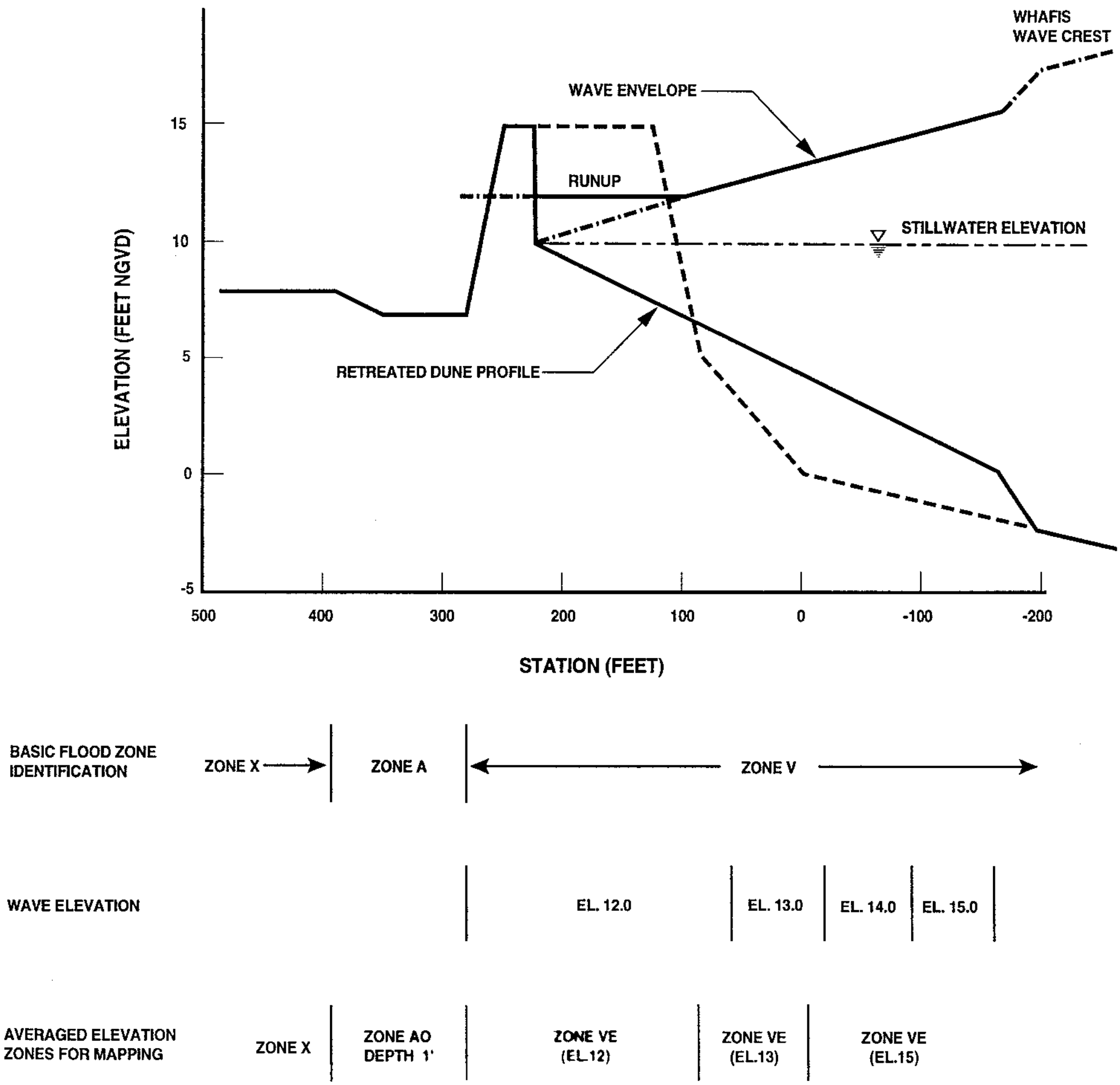

Figure 29. Identification of Elevation Zones, Example 3: Low Retreated Dune with Wave Overtopping.

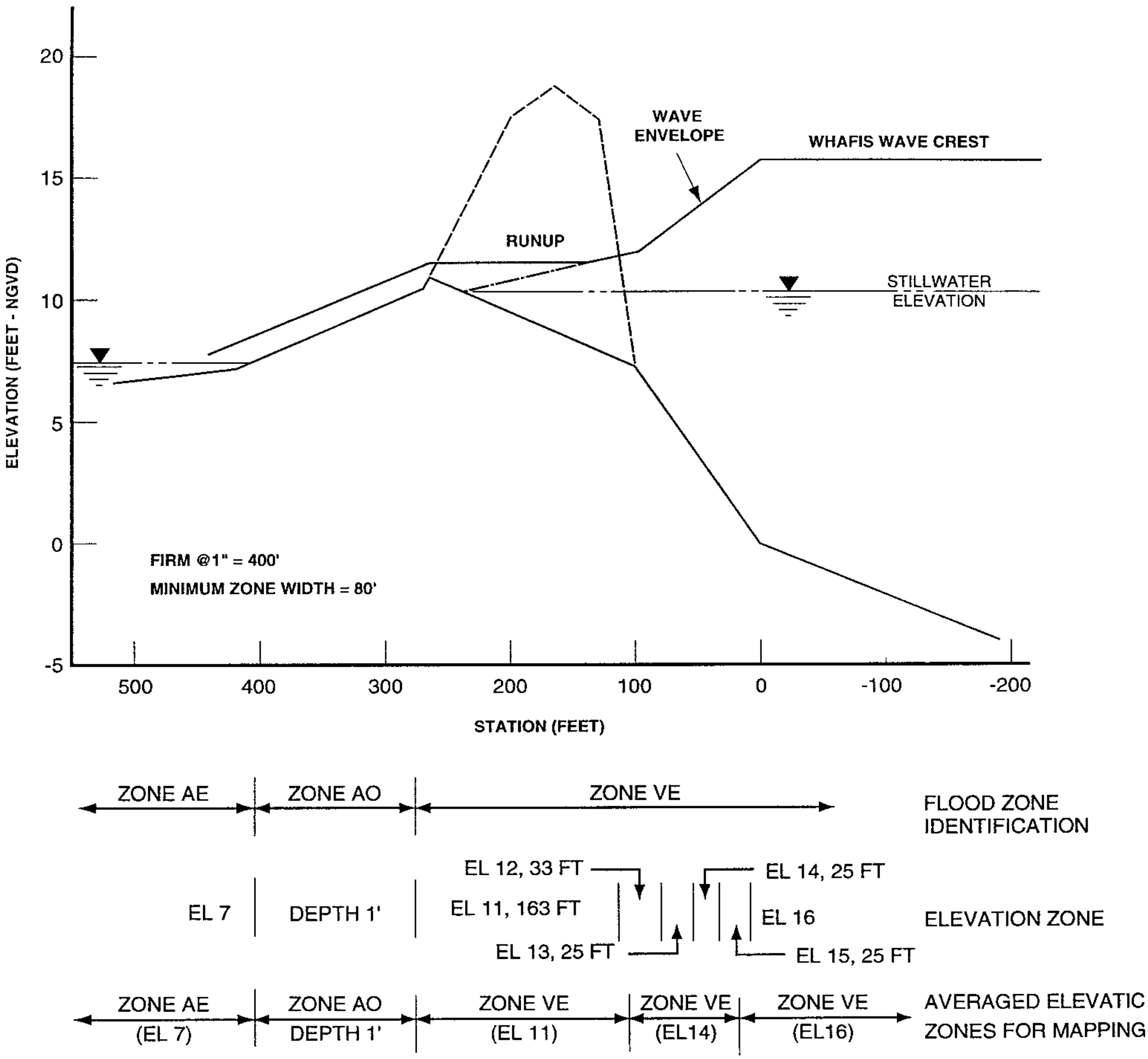

Figure 30. Identification of Elevation zones, Example 4: Dune Removal with Wave Runup and Runoff

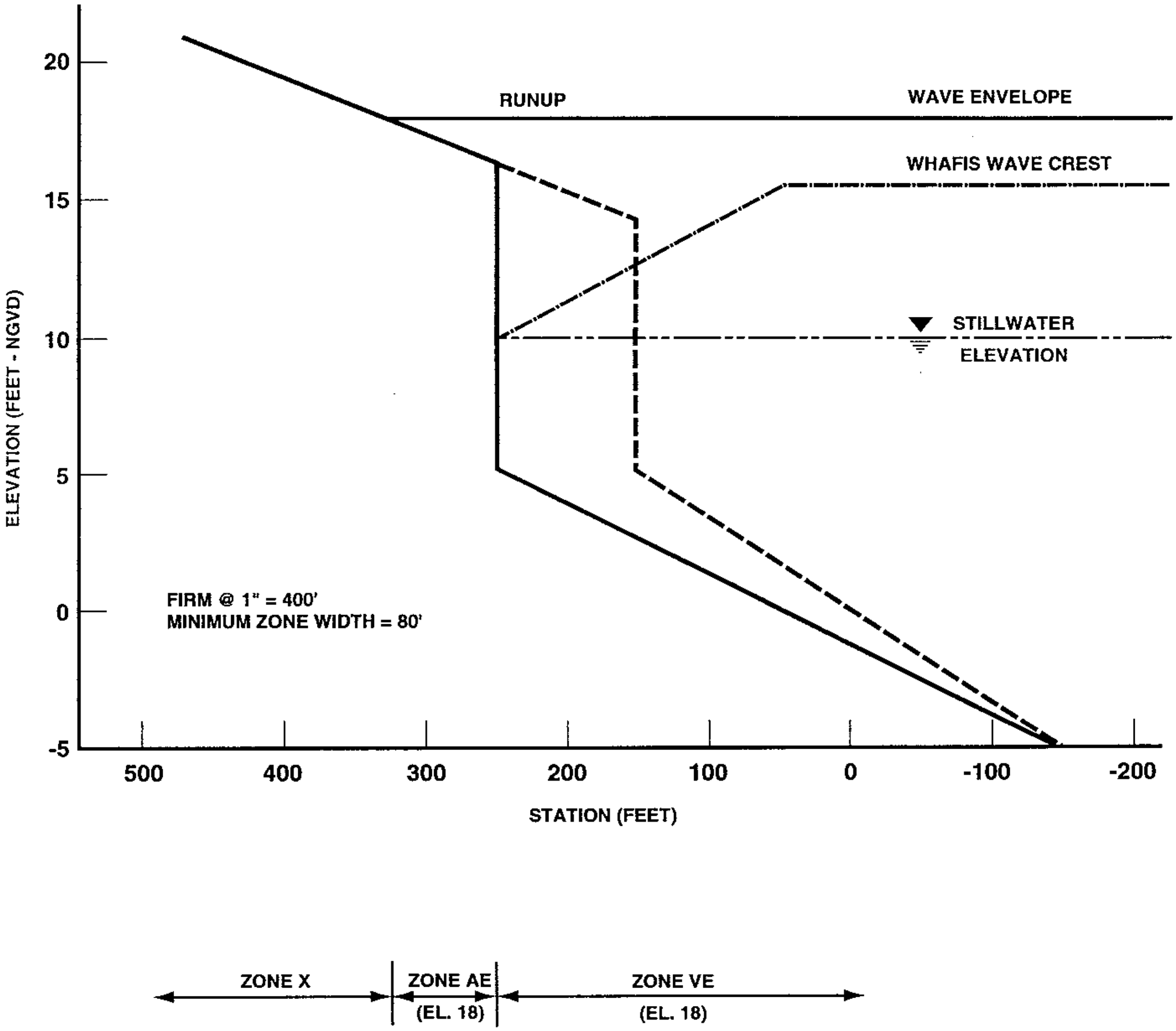

Figure 31. Identification of Flood Zones, Example 5: Eroded Bluff with Wave Runup.

between the runup elevation and the ground is less than 3 feet. In this figure, the Zone AE, elevation 18 feet, is slightly less than the minimum mapping width. As recommended for Figure 27, the neighboring area on the work map should be examined to determine if this zone should be mapped. AE zones are usually not mapped for bluffs unless computed runup exceeds the bluff crest, as shown in Figure 31. (Note that Figures 16 and 17 outline another flooding treatment of bluffs where computed runup is well above the crest).

On sandy shores, it is usual for transects to extend across barrier islands, marshes, inland water bodies, etc., such that two or more areas of VE zones can be identified. Procedures in these cases are the same with elevation averaging also very common. With a little practice, identification of the flood zones and elevations becomes fairly routine using the wave envelope and transect profile.

With shore structures having 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 appropriate assessment of flood hazards. Following examples provide limited discussion of schematic effects for a few basic configurations, presuming the structures remain intact through the base flood and no appreciable shore erosion occurs.

Figure 32 presents an example with moderate structure overtopping expected for waves accompanying the base flood. The structure crest has sufficient freeboard above 100-year stillwater elevation to contain calculated mean runup of 6 feet, but extreme wave runups are likely to overtop the structure intermittently. The entire extent of shore structure is treated as a unit and designated as a VE Zone, assigned the mean runup elevation of 16 feet. Landward of the structure, an area with at least the minimum mapping width is appropriate for designation as Zone AO, depth 1 foot, with extent depending on ground profile.

Figure 33 is an example for a structure extending above 100-year stillwater elevation but heavily overtopped by wave action. Calculated mean runup elevation is 5 feet above the seaward face, but that is reduced to the maximum excess runup of 3 feet in assigning a flood elevation of 16 feet for the shorefront VE Zone. That zone extends through the entire structure and over an additional 30 feet landward, because likely wave impact area is judged to reach beyond the structure during the base flood. Cumulative wave overtopping yields ponding within an additional landward area 100 feet in width, designated as Zone AO, depth 2 feet.

Figure 34 provides an example with a structure covered by 3 feet of water during the base flood. Flood depth is not sufficient for waves 3 feet in height to propagate inland of the structure, but the V Zone must extend to 30 feet landward of the structure, in view of

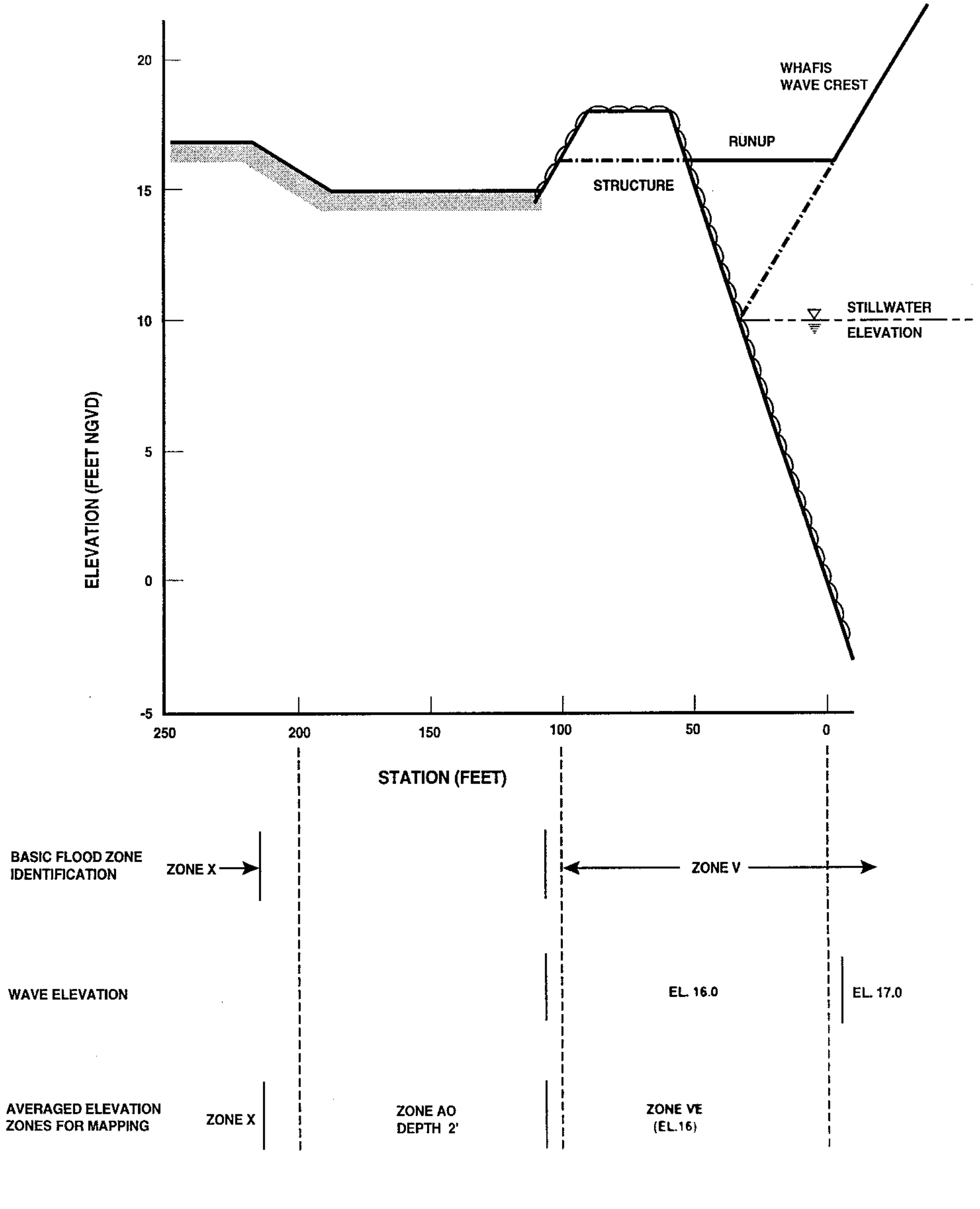

Figure 32. Identification of Elevation Zones, Example 6: Coastal Structure with Moderate Wave Overtopping.

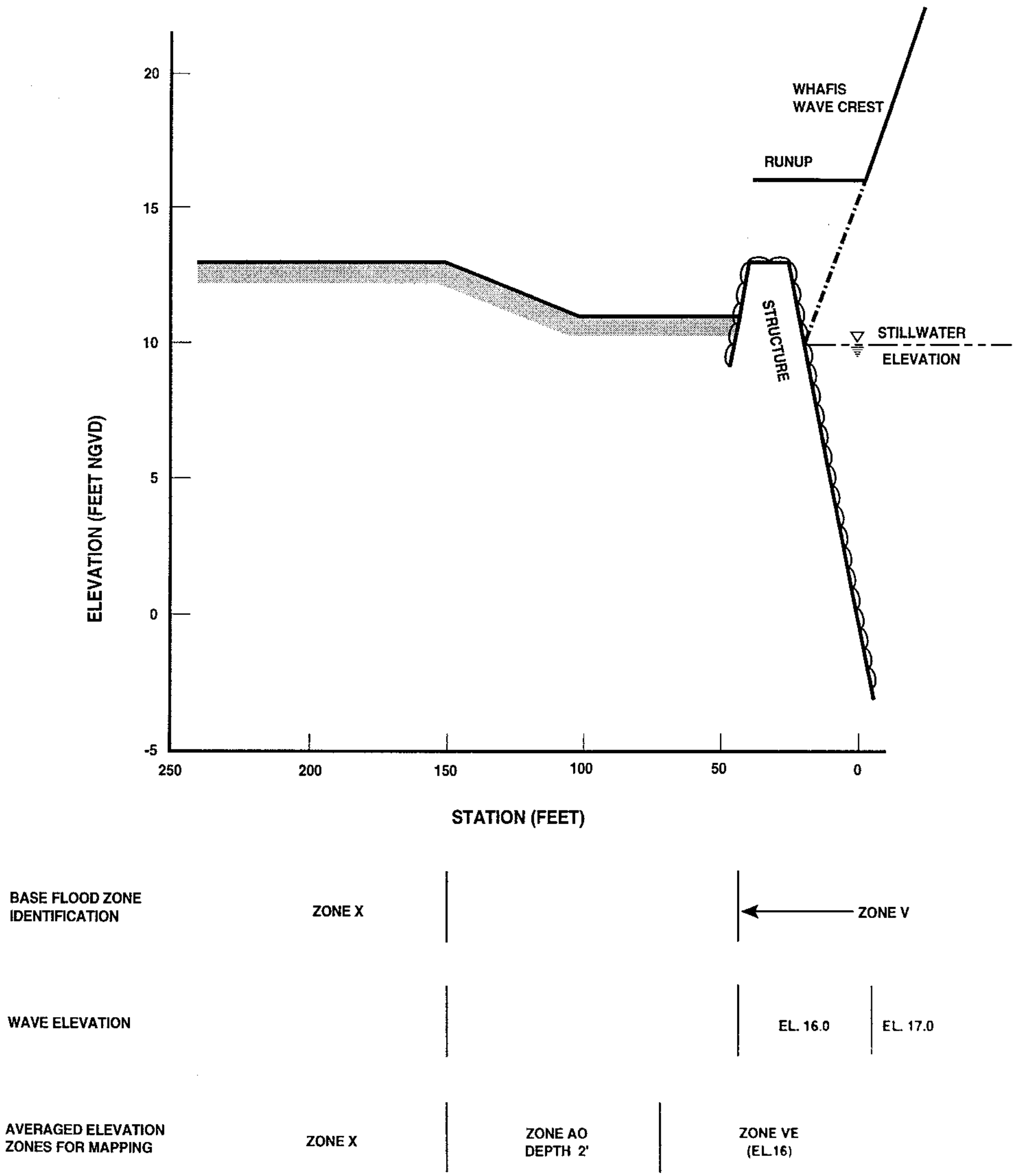

Figure 33. Identification of Elevation Zones, Example 7: Coastal Structure with Severe Wave Overtopping.

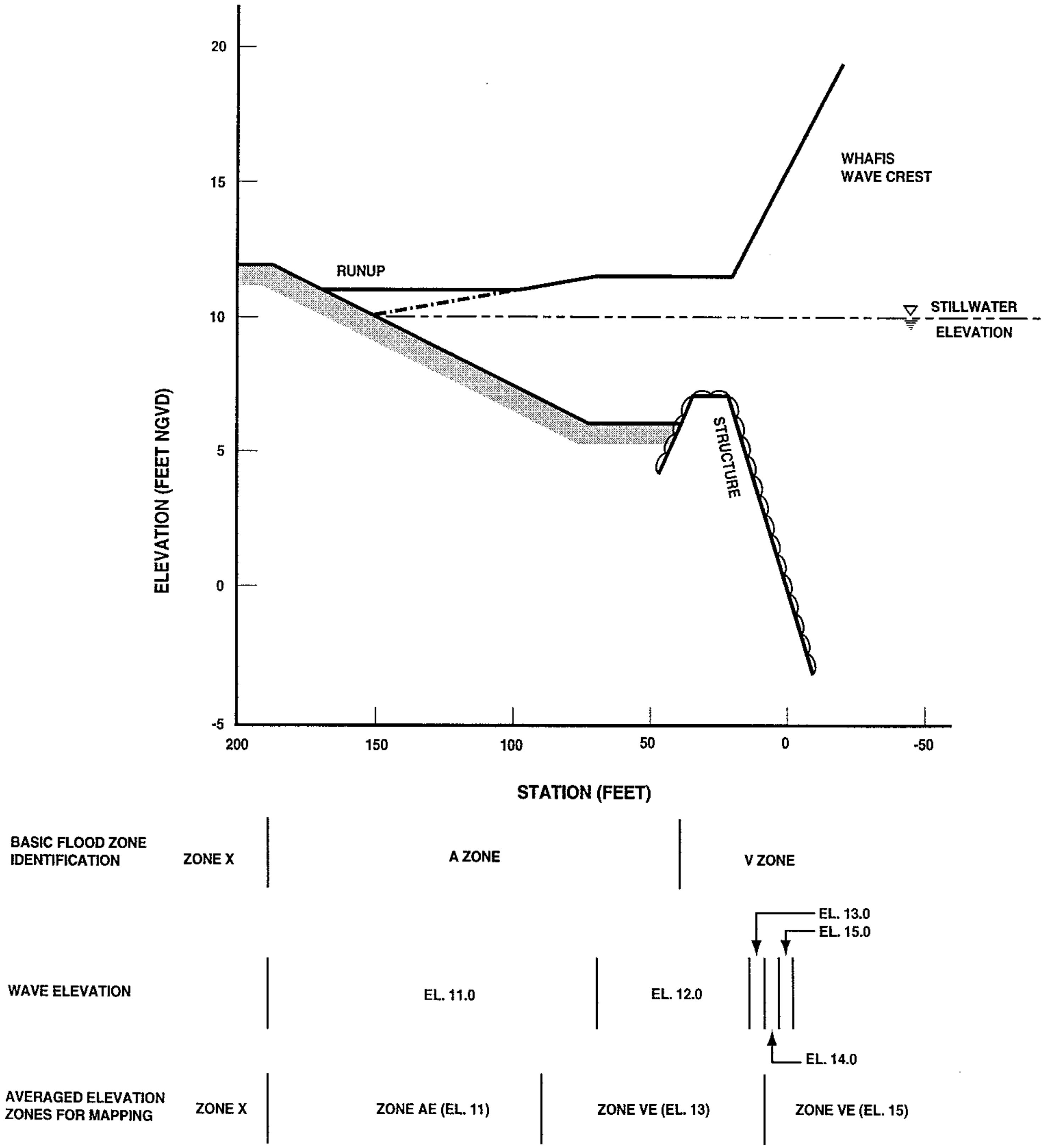

Figure 34. Identification of Elevation Zones, Example 8: Coastal Structure with Inundation.

likely wave impacts through the flood's course. The shore structure is too narrow for multiple V zones to be delineated, so there is one designation of Zone VE, elevation 13 feet. Landward of that, further wave hazards occur in the Zone AE, elevation 11 feet.

In examining Figures 32-34, it may seem surprising that relatively high structures can result in higher flood elevations, compared to an inundated structure. However, a structure with more freeboard can deflect incident wave action to greater elevations during the base flood, so the present zonations are physically appropriate. The hazard zonations landward of coastal structures generally have more importance, and those reflect the greater protection provided by higher but durable structures.

7.4 Mapping Procedures

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 extents will be negated if the results of these analyses are not properly delineated on the FIRM. The FIRM is usually produced from the work maps described in Chapter 2, so the flood zones and elevations identified on the transects need to be transferred to the work maps, and boundaries interpolated between transects. The work maps should be set up with contour lines, buildings, structures, vegetation, and transect lines clearly located. Because roads are often the only fixed physical features shown on the FIRM, it is important 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.

For each transect, the identified elevation zones are transferred from the transect to the work maps. The location of the boundaries are marked along the transect line so that boundary lines can be interpolated between transects. Care should be taken to assure that boundaries are marked at the correct location. Because of erosion assumptions, the location of the elevation 0.0 shoreline changes on the transect but not the work maps. Using the transect profile, determine the location of the zone change in relation to a physical feature such as a ground contour, the back side of a row of houses, 50 feet into a vegetated area, etc. Delineate the boundary line along this feature for the area represented by that transect.

Carefully watch the widths of the zones being delineated; if they narrow to less than 0.2 inch, they should be tapered to an end. Likewise, if the zone becomes much wider, it may be possible to break an averaged elevation zone into two mapped elevation zones.

One of the more difficult steps in delineating coastal flood zones and elevations is the transition between transect areas. Good judgment and an understanding of typical flooding patterns are the best tools for this job. Initially, locate on the work maps that area of transition: an area not exactly represented by either transect. Delineate the flood boundaries for each transect up to this area. Examine how a transition can be made across this area to connect matching zones, and still have the boundaries follow logical physical features. See if there are other transects that are similar to this area and could give an indication of flooding. Sometimes the elevation zones for the two contiguous transects are not the same; thus, some zones may have to be tapered to an end, or enlarged and divided in the transition area.

Communities with significant flooding hazards from wave runup may have one transect representing more than one area because the areas have similar shore slopes. In this case, the different areas are identified and the results of the typical transect delineated 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 boundaries. These zones should be fairly short and cover the shore segment with a slope not exactly typical of either area. The transition elevation is determined using judgment in examining runup transects with similar slopes. Transition zones should not be used if there is a very abrupt change in topography, such as the end of a structure.

Lastly, shaded Zone X's are mapped. Areas below the 500-year stillwater elevation and not covered by any other flood zone are designated Zone X and shown shaded on the FIRM. Often the maximum runup elevation is higher than the 500-year elevation; thus, there will be no shaded Zone X in that area. All other areas are designated Zone X without any shading.

Because flood elevations are rounded to the nearest whole foot, there is no reason to spend hours resolving a minor elevation difference. Also, because structures or proposed structures must be located on the FIRM, an attempt should be made whenever possible to smooth the boundary lines and to follow a fixed feature such as a road. In preparing the FIS, not only must the mapped results be technically correct, but the FIRM must be easy to use by the local insurance agent, building inspector, or permit officer.

These <u>Guidelines</u> have been compiled to provide guidance in the preparation of coastal FISs. The initial collection of accurate and representative data, the correct application of the models, the careful evaluation of results and comparison to historical data, and the proper delineation of flood elevations and zones will produce a FIRM that is both technically correct and directly useful. During all steps of the study, especially the mapping, the final product and its purposes should be remembered: the FIRM is used to determine flood insurance premiums and to regulate building standards.

8.0 STUDY DOCUMENTATION

The coastal flood hazard determination for each particular community shall be fully documented. Because FISs form the basis of Federal, State, and local regulatory and statutory enforcement mechanisms and are subject to administrative appeal and litigation, it is extremely important that all technical processes and decisions be fully recorded and documented. The FIS text has not been designed to contain all the documentation that would be needed for a response in the event that the study results are questioned; therefore, an engineering report is required for each study. This report will provide detailed data needed by FEMA, or the community, to reconstruct or defend on technical grounds the study results. At a minimum, the following information must be included:

a. Basic Data. This section will include all contacts made to obtain data for the study. All basic data used must be fully referenced and, if possible, reproduced in the report. It is very important that all historical flood information be documented in this section, even if it was not used in quantitative analyses.

b. Transects. All transects used must be shown on a map. Each transect must be plotted separately and show the erosion assessment, input data for wave models, wave envelope and zone determination.

c. Model Input and Output. Computer printout listings for input and output data for both the Wave Runup and Wave Height Models must be provided for all the transects. These must be keyed to the transect location map and transect plots.

d. Study File. During the course of the study, a file should be maintained that records all coordination, activities, and decisions. This is especially important where nonstandard approaches were used and engineering judgment played a significant role. This file should be in chronological order and include all written correspondence, interoffice memorandums, records of conversations, and working notes pertinent to the study.

9.0 REFERENCES

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APPENDIX A

Example Coastal Flood Insurance Study
For A Site Along the Atlantic Ocean

Guidelines and Specifications
for
Wave Elevation Determination and V Zone Mapping
Example Study

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Introduction

This example study is being prepared as an appendix to the Guidelines and Specification for Wave Elevation Determination and V Zone Mapping in order to provide a realistic application of the described methodology. It does not claim to cover all cases and scenarios which may be found in the field, nor is it meant to be directly applied to all coastal Flood Insurance Studies.

It is the determination of the Coastal High Hazard Area (CHHA) or V-Zone, which is critical to any coastal analysis, and it is important to keep the V-Zone definitions in mind throughout a coastal study, including this example. The CHHA is defined as the most landward of three points:

1) the point where a three foot wave height may occur;
2) the point where the eroded ground profile (or non-eroded profile if applicable) is 3 feet below the calculated wave runup elevation; and
3) the inland limit of the Primary Frontal Dune (PFD) as defined in the National Flood Insurance Program (NFIP) regulations.

A good coastal study is performed to locate all of these points for each transect, where applicable, so that the most landward can be chosen.

The site chosen for this example is found approximately 62 miles southeast of Boston, Massachusetts, along the Atlantic Ocean within Cape Cod Bay. For the purposes of reference, this area will be referred to throughout this example as Smithville. Smithville's corporate limits have been fictitiously created; however, much of the ground data used to perform this analysis comes from real surveyed information.

Five transects have been analyzed in this example study. Transect A typifies a large dune experiencing dune retreat, transect B typifies a small dune experiencing dune removal, transect C can be characterized as a steep bluff with a revetment, transect D typifies a seawall, and transect E is characterized as a marsh.

Several methods were used to determine deepwater wave heights and periods for this study. Based on the geometry of the local study site, the Automated Coastal Engineering System (ACES), developed by the U.S. Army Corps of Engineers (USACE), was considered to be the most appropriate for determining wave conditions.

Runup elevations were determined using the Federal Emergency Management Agency's (FEMA's) current wave runup model (RUNUP 2.0) and guidance from the Shore Protection Manual (1984). Wave height and crest elevations were determined using FEMA's Wave Height Analysis for Flood Insurance Studies (WHAFIS 3.0) model.

Compile Necessary Data and Information

The first step in preparing a detailed coastal analysis is to compile the information needed to run the appropriate models. Keep in mind that the overall goal is to delineate, as accurately as possible, the hazards associated with major coastal storms, specifically the base (100-year) flood, which is the flood having a one percent chance of being equalled or exceeded in any given year. The following is a list of materials that may be helpful when preparing a coastal study:

- Effective Flood Insurance Rate Map (FIRM), and Flood Insurance Study (FIS)
- Local topography of a suitable contour interval and scale

- USGS quadrangle maps
- NOAA nautical charts/bathymetric data
- Aerial photographs/land cover data
- Ground surveys
- WIS wave hindcast reports number 19 and 30
- Buoy and gage data (if available)
- Shore Protection Manual (1984)
- Historical flooding information (if available)
- WHAFIS 3.0, RUNUP 2.0, and ACES 1.07 models
- NFIP regulations

Most of the above materials are easily obtained. The FIRM and FIS are available through the local map repository, and the other documents can be obtained from the various agencies. The WHAFIS, and RUNUP models can be obtained from FEMA, and the ACES model is available through the Federal Software Exchange Center in Springfield, Virginia.

Locate Transects

Each transect is used to represent a length of shoreline which contains similar physical features and cultural characteristics. Since the defined 100-year stillwater elevation does not vary along the shoreline for this example, one transect could be used to represent several different sections of the study site. Figure 7-1 in Part 7 of the Exhibit shows the length of beach represented by each transect, as well as the location of each transect. The Smithville site was found to be accurately described by locating the five transects described in Table 1.

Determine Stillwater Elevation

To determine the stillwater elevation for the Smithville site, two references were consulted. The first was the USACE's

September 1988 report entitled "Tidal Flood Profiles New England Coastline". This document reports a 100-year stillwater elevation (without the wave setup component) of 10.4 feet National Geodetic Vertical Datum (NGVD). Also reported in this document are the mean high water and mean low water elevations for the Smithville site.

The second document used to determine the 100-year stillwater elevation was the current Flood Insurance Study (FIS) for the Smithville site. It too reports a stillwater elevation of 10.4 feet NGVD for this site. The covers of these references, and pages used to determine the 100-year stillwater can be found in the Exhibit, Part 1. Table 2 summarizes the findings.

The stillwater elevation used for this example is 10.4 feet NGVD, without the wave setup component. In this example, it was assumed that this elevation does not vary along the beach within the study area; however, this condition may not be valid for every study. For larger studies it is not uncommon to find stillwater elevations varying significantly along the shoreline.

Determine Wave Characteristics and Wave Setup Magnitude

<u>Wave Characteristics</u>

There are several methods for determining deepwater wave characteristics H_{mo} (deepwater wave height), and T_p (wave period); however, these methods are dependent on the type of event being considered. In the northeastern United States, "Northeasters" can cause significant flooding in coastal areas due to their long duration and intensity. Along most of the Atlantic coast, and in the Gulf of Mexico, hurricanes are the dominant event. The Smithville site however, happens to be exposed to both types of events, therefore, a determination had to be made as to the most appropriate for flood insurance purposes. Since the site is sheltered from southerly exposure by a massive cape, and the shoreline is more exposed to the northeast, it was determined that a "Northeaster" would be the more critical event. This is also documented in the FIS on page 5 as shown in Part 1 of the Exhibit.

Using the USACE's Automated Coastal Engineering System (ACES), 19 fetches were delineated at 6 degree intervals from approximately 343° (compass heading) to approximately 91° as shown in Figure 2-1 in Part 2 of the Exhibit. The length of each fetch was also determined and this information is summarized in Table 2-2 in that same part. Using ACES with the restricted fetch option, 6 predominant wind directions were initially analyzed (5°, 15°, 25°, 35°, 45° and 55°). A fifty mph average overwater wind speed sustained for the duration of the event was assumed, and used as the observed wind speed. The results of these six computations showed that the mean wave direction at approximately 22°. Therefore, a seventh wind direction of 22° was used to determine another set of wave conditions. A sixty mph wind was also used to determine yet another set of wave conditions, and the output from these runs has been included in Part 2 of the Exhibit. The results have also been summarized in Table 2-1 of the Exhibit. As seen from Table 2-1, the waves computed using the restricted fetch option are quite steep (H_{mo}/L_o is approximately 0.043). Since Northeasters do not typically have wave steepness values of this magnitude, a ninth ACES run was made, this time using the open water fetch option, with a fetch length of 160 statue miles, and an overwater wind speed of fifty mph. This computation resulted in a wave steepness much lower than the restricted fetch computations, and is more typical of northeast storm waves.

As a check to the ACES computations, we referenced the USACE-WES Wave Information Study (WIS) Report 30, entitled "Hindcast Wave Information for the US Atlantic Coast" and dated March 1993. The maximum waves computed over a twenty year period, at sites 93, 94, and 95, resulted in wave steepnesses of approximately 0.023, 0.022, and 0.030, respectively. Computed wave heights ranged from approximately 20 feet to 31 feet, and wave periods ranged from approximately 13 seconds to 15 seconds. These values are summarized in Table 2-1 in the Exhibit, and the WIS report pages from which these values were taken have also been included in Part 2 of the Exhibit.

Station 93 in the WIS report is the most applicable to the Smithville site, and the ACES open water wave period computation (12.7 seconds) compares with the WIS 20-year maximum wave period (13 seconds); however, the wave height computed using ACES (28.0 feet) is slightly larger than the 20-year maximum reported in the WIS report (20.3 feet). Given the relatively complex geometry in the vicinity of the Smithville site, we considered the ACES open water computations to be the most appropriate. These conditions, along with the various water surface elevations used are summarized in Table 2, and these values will be used throughout this example for further computations. Since the study area is small, and not drastically convoluted, we assumed that the wave conditions would be constant for the entire length of shoreline; however, in large studies or in areas where there are more complex shoreline geometries, this assumption may not be appropriate.

<u>Setup Computation</u>

Along the open coast, when waves approach the shore, they attenuate and eventually break in shallow water. This wave action can significantly increase mean water surface elevations close to shore. Therefore, a setup component was computed using the methodology outlined in the Shore Protection Manual (SPM). A nearshore slope of approximately 1/85 was used for this computation, and this was assumed to be relatively constant throughout the study area (refer to Table 4-2 through 4-5 in Part 4 of the Exhibit). Since the wave conditions where also assumed to be constant, and the nearshore slope does not vary drastically, we assumed that the magnitude of the wave setup would remain approximately constant for the Smithville site. For other studies however, these assumptions may not be appropriate. Good engineering judgement should be used in making such assumptions, and if there is any doubt as to the validity of an assumption, it should be tested for appropriateness.

For the Smithville site, a wave setup magnitude of approximately 2.0 feet was computed. The steps used to arrive at this value are presented as Worksheet 2-1 in Part 2 of the Exhibit.

Perform Erosion / Scour Assessment

For this example an erosion assessment was performed at transects A and B. For transect D, a scour assessment was made. For transects C and E, it was assumed that erosion would not be significant given the relatively mild slopes and heavy land cover.

Transect A typifies a large dune whose crest rises to 25 feet NGVD. Transect B typifies a small dune whose crest only reaches 15 feet NGVD. The FEMA erosion methodology was used to compute an eroded profile for each of these two cross sections, and the results are presented as Worksheets 3-1 and 3-2 in Part 3 of the Exhibit. The pre-storm ground elevations were determined using field data and USGS quadrangle maps.

Transect A was found to have a reservoir area greater than 540 ft^2, therefore the dune face was retreated rather than removed. Transect B was found to have a reservoir area less than 540 ft^2, therefore, the dune was removed. Figures 3-1 and 3-2 in Part 3 of the Exhibit show the pre-storm and eroded profiles for transects A and B. No historical information could be obtained documenting the effects of storm induced erosion in this area.

Transect D was used to represent the seawall along the shoreline at the northern corporate limit of Smithville. Since the toe of this structure was not protected, the effects of scour needed to be addressed. It was assumed that the amount of scour would be approximately equal to the significant wave height at the seawall.
The scour depth at the toe of the seawall for this example was found to be approximately 2.2 feet. The scour elevation at the toe of the seawall for this example was found to be approximately 3.8 feet NGVD. The procedure for this analysis is presented as Worksheet 3-3 in part 3 of the Exhibit. The approach outlined in Worksheet 3-3 is being presented as an attempt at quantifying toe scour. The results obtained appear to be reasonable in terms of qualitative guidance and general experience; however, this procedure may not be appropriate for every case. The results of this analysis are shown graphically in Figure 3-3 of Part 3 in the Exhibit.

Perform Runup / Overtopping Analysis

<u>Runup Computations</u>

Runup computations were made along transects A, B, and C, using the FEMA runup model. Mean wave conditions, local bathymetry, stillwater elevation (without the wave setup component), and eroded ground elevations were used as input. The mean deepwater wave characteristics (H_{bar} and T_{bar}) were computed using the following relationship:

$$H_{bar} = H_{mo} * 0.625$$
$$T_{bar} = T_p * 0.85$$

where H_{mo} is the deepwater significant wave height, and T_p is the dominant wave period, both of these values were computed previously. Nine wave conditions where input for each transect representing a variety of conditions from 1.05 H_{bar} to 0.95 H_{bar} and 1.05 T_{bar} to 0.95 T_{bar}. A stillwater elevation of 10.4 feet NGVD was used for each transect. Nearshore bathymetry was averaged using water depths obtained from the USGS quadrangle map, as shown in the Table 4-2 to 4-4 in the Exhibit, and are further summarized in Table 3. A roughness coefficient of 0.60 was assumed for the rock revetment at transect C, and a coefficient of 0.90 was assumed for the grass along that same transect.

For transect D (seawall), the methodology found in the SPM was used to compute a runup elevation. This elevation was found to be 27.9 feet NGVD; however, it was assumed that this elevation would not be appropriate for floodplain management purposes since it is more than 3 feet above the wall cap elevation. Nevertheless, this computation implies that overtopping should be considered.

Given the mild slope, and heavily vegetated ground surface along transect E, runup was assumed to be negligible. The runup results for the remaining transects are summarized in Table 4. It can be seen from Table 4 that the runup elevations for each transect are less than the stillwater elevation plus the wave setup component previously computed (12.4 feet NGVD). However, these results should not be dismissed because they will be needed to compute a possible location of the V-Zone (ie. that were the eroded ground profile is 3 feet below the mean runup elevation).

The input and output files used for the FEMA runup model, as well as transect D runup computations are presented in Part 4 of the Exhibit.

Overtopping

Overtopping was assessed at transect D, and reviewed for appropriateness at transect B. Since the eroded dune crest for transect A was much higher than the computed runup elevation, overtopping was not assessed at this location. Furthermore, since the bluff crest at transect C is much higher than the computed runup elevation for that transect, overtopping was not considered. For transect E overtopping was not applicable.

For transect D, the runup elevation was found to be quite substantial, therefore, overtopping was assessed at this location. Using the procedure outlined in the main text of the Guidelines and Specifications for Wave Elevation Determination and V-Zone Mapping, it was found that overtopping may exceed 1 cfs/ft. Therefore, the V-Zone should extend, at a minimum, 25 feet landward of the seawall cap, and an area of Zone AO (shallow water flooding) should be delineated landward of the V-Zone.

The details of the overtopping assessment for transect D are presented as Worksheet 4-2 at the end of Part 4 in the Exhibit.

As the water level rises in front of a structure or dune, it will reach a level where overtopping becomes excessive, this may cause high velocities landward of the structure crest. Therefore, in cases where structures or dunes are completely inundated during the 100-year event, it is good practice to extend the V-Zone, at a minimum, 25 feet landward of the crest to allow for energy dissipation when overtopping becomes critical. For transect B, this was found to be the case; therefore, the final location of the V-Zone should be extended at least 25 feet landward of the eroded dune crest.

Perform WHAFIS

The WHAFIS 3.0 model was used for transects A through E to determine appropriate wave crest elevations along each. Eroded ground elevations were used, and vegetation was modeled where appropriate. A printout of the input and output files for each transect is located in Part 5 of the Exhibit.

For transect A, marsh grass (Region 2 SPAT) was assumed to exist from approximately station 700 inland to approximately station 3500. A stillwater flood elevation of 12.4 feet NGVD was used from station 0 to approximately station 700, where the wave setup component was removed from the stillwater elevation. A 100-year stillwater elevation of 10.4 feet NGVD was then used from approximately station 700 inland to approximately station 3500. All buildings were assumed to be elevated above surge for this transect.

For transect B, marsh grass (Region 2 SPAT) was assumed to exist from station 400 inland, and the first and only row of buildings was assumed to be removed due to erosion.

For transect C, the rock revetment from station 120 to station 220 was not modeled; however, based on a field inspection by a certified professional engineer, the structure was assumed to be likely to withstand the 100-year event. The WHAFIS model input was terminated at approximately station 220, where the ground elevation rises and remains above 12.4 feet NGVD.

For transect D, the seawall cap was surveyed at 18.2 feet NGVD. Taking into consideration the overtopping and scour assessment, combined with the results of a field inspection by a certified professional engineer, it was assumed the structure would likely withstand the 100-year event, and was modeled in place.

For transect E, marsh grass (Region 2 SPAT) was assumed to exist from station 500 inland to station 4200, and the wave setup component was removed from the 100-year stillwater flood at approximately station 500.

Construct Wave Envelopes

The wave envelope combines the results of all modeling and analysis in a graphical manner, making it one of the most critical components of the coastal flood insurance study. Figures 6-1 through 6-5 of Part 6 of the Exhibit presents the information typically shown on a wave envelope. Stationing is shown along the X-axis (from the pre-storm zero NGVD), and elevation is shown along the Y-axis. In each figure, pre-storm and eroded ground elevations, wave crests, average zone wave heights, and the inland limit of the V-Zone are shown. The average zone wave heights shown on figures 6-1 through 6-5 where computed at a minimum 200 foot increment given the scale of the work maps (1"=1000'). It is assumed that a width less than 1/20th of an inch can not accurately be shown. Tables 6-1 through 6-5 in Part 6 of the exhibit are presented to summarize the final results for transects A through D, respectively. Table 5 is presented to summarize the various criteria used in determining the inland limit of the V-Zone for each transect based on the 3 criteria mentioned in the introduction to this example. Tables 6-6 through 6-23 of the Exhibit are presented for completeness, and are meant to supplement the wave envelope figures.

Mapping The Results

After creating the wave envelopes, the zero station was transferred to the base maps (USGS quadrangle in this example) to obtain a fixed reference point from which to delineate the flood zones. Using this point, in conjunction with summary tables 6-1 through 6-5, and the wave envelopes, the flood zone boundaries and elevations were delineated on the work maps.

Once that was done for each transect, the next step was to interpolate the location of these boundaries between transects. The wave crest envelopes and work maps are used together to find the location of zone changes in reference to physical features such as ground elevations, roads, houses, vegetation, seawalls, revetments, dunes, etc... Boundary lines are then delineated outward perpendicular to the transect, following the direction of identified physical features, and generally parallel to the shoreline. These zones are extended from the transect to a point where there is a significant change in physical features. Once this point is reached for each transect, the engineer must examine how a logical transition can be made across the areas between transects where zones have not been delineated. In some cases this area may be so dramatically different from the surrounding areas that an additional transect must be added to accurately describe it.

Figure 1 shows a general schematic of how results are interpolated between transects. The solid arrows extending outward from the transect lines terminate at the points where physical features were judged to have changed significantly. As shown in Figure 1 by the dashed lines, similar zones are then connected through areas where the physical features between the transects could be described as a combination of the two transects. Notice that transect 3 in Figure 1 contains an area of Zone AE (El. 11) and Zone X, and transect 2 does not; therefore, these two zones where tapered to an end using engineering judgement given the physical features of the area. Similarly, for transect 1, Zone VE (El. 17) differed from the Zone VE (El. 15) found along transects 2 and 3; therefore, zone VE (El. 17) was terminated at a location deemed to be the most logical based on physical features (end of a seawall, revetment, dune, etc...). Note that in some cases it can be assumed that a transect is applicable at several locations along the coast which are not necessarily physically connected; however, caution should be used when exercising this technique over long stretches of shoreline because stillwater elevations, wave characteristics, and nearshore bathymetry may vary significantly.

The above described methodology was used for delineating the flood zones at the Smithville site, and the results are presented in Part 7 of the Exhibit.

Review and Evaluate Results/Compare to Historical Flooding Information

Using the various summary tables and figures in Part 6 of the Exhibit, and the maps in Part 7, the results are reviewed for reasonableness. No historical flooding information was available for the Smithville site, so a direct comparison between the presented results and past flooding occurrences could not be made; however, a review of the major components of this study, and the assumptions made will help in determining the studies reasonableness.

The 100-year stillwater elevation of 10.4 NGVD was obtained from two different sources, and is believed to be the best available information. Given the two harbors at either end of the study area, this elevation alone (without wave setup, beach erosion, wave height analysis, or runup computations) would inundate the marsh areas at a minimum to an elevation of approximately 10 feet NGVD.

Deepwater wave conditions were determined using several methods. The ACES results seemed to generally agree with the information published for Station 93 in the WIS report 30, and the results appear to be reasonable based on past experience and the understanding of typical northeast storm waves.

The wave setup contribution to the stillwater elevation was computed using the methodology outlined in the SPM. According to this methodology, the setup component is a function of deepwater wave height, wave period, and nearshore beach slope. The nearshore slope was averaged for the site; however, the magnitude of the setup component is only slightly affected by changes in nearshore slope. Therefore, the averaging of this slope is not considered to have a major impact on the overall magnitude of the setup computation. The wave characteristics are more critical in the determination of the wave setup, and confidence in these values is crucial. As mentioned previously, we are fairly confident with these values, and the assumption that they will not vary significantly along the shoreline within the study area.

The erosion assessment for transects A and B was performed using standard FEMA methodology based on the 540 ft^2 rule. No historical information was available for a comparison of these results. Since scour is know to occur in front of seawalls exposed to wave action, the depth of scour at transect D was estimated assuming that the scour depth at the toe of the structure would be approximately equal to the significant wave height occurring directly in front of the wall. Once the scour assessment was performed, the integrity of the wall was reviewed for failure due to wave action and soil pressure. The wall was found to be stable under these conditions; however, no historical data was available for a comparison. No erosion assessment was performed for Transect E due to its relatively mild slope and heavy vegetation. The rock revetment at transect C was reported to have withstood several large historical events with little to no damage. The revetment was judged to be in good condition based on a field inspection, and it was assumed to withstand the 100-year event based on the recommendations from a professional engineer with experience in coastal structures. Furthermore, this structure was assumed to prevent excessive erosion.

Runup analysis was performed for transects A, B, and C using the FEMA runup model version 2.0. For transect D, the methodology outlined in the SPM was used. At transect E, wave runup was not applicable given the mild slope. According to the USGS quadrangle map, the nearshore bathymetry was fairly constant throughout the study area, and was therefore averaged for the entire length of the site. The computed runup elevations at transects A, B, and C, were lower than the computed setup component; therefore, they were not plotted on the wave envelopes. However, the point where the eroded ground profile was 3 feet below the computed runup elevation was still considered in locating the inland limit of the V-Zone, although this point was found to be seaward of the inland limit of the primary frontal dune. For transect D, the runup elevation was considered to be excessive since it rose more than 3 feet above the crest of the seawall. Therefore, it was recommended that the computed runup elevation not be used for floodplain management purposes. No historical data was available to compare the results of the runup analysis.

Overtopping analysis was performed only at transect D, the seawall, and was found to most likely be excessive. Therefore, the V-Zone was extended 25 feet landward of the crest of the structure to allow for adequate energy dissipation. Furthermore, a shallow water ponding area (Zone AO depth 2 feet) was delineated behind the seawall based on the overtopping assessment. For transect B, the dune remnant was completely inundated, thus overtopping was assumed to be extensive; however, the inland limit of the primary frontal dune was found to be further landward than the 25 feet required for excessive overtopping. Overtopping was not considered at transect C or A since the crest of the bluff and dune were much higher than the computed runup elevation. Overtopping was not applicable at transect E. No historical data was available to compare the results of the overtopping assessment.

After comparing the results of this study with those found on the effective FIRM, it was obvious that the inland limit of the primary frontal dune had not been considered in the previous analysis. Furthermore, the elevations along the shoreline proposed in this study were slightly larger than those shown on the effective FIRM. The WHAFIS model input used for the effective study was not available for review; however, the differences in these elevations may be due to differences in the magnitude of the wave setup component, or the wave conditions.

The floodplain boundaries matched well with those found on the effective FIRM, as did the zone elevations landward of the dunes and first row of structures. The results of this study were found to be generally consistent with the information shown on the effective FIRM, and were found to be reasonable.

Table 1 - Transect Descriptions

Transect	Description
A	Represents approximately 1.3 miles of shoreline along a LARGE DUNE.
B	Represents approximately 2.4 miles of shoreline along a SMALL DUNE.
C	Represents approximately 1.0 mile of shoreline along a REVETMENT backed by a steep BLUFF.
D	Represents approximately 0.2 miles of shoreline along a SEAWALL.
E	Represents approximately 0.3 miles of MARSH along the shoreline.

Table 2 - Summary of Water Elevations and Wave Characteristics

Parameter	Magnitude
Mean Low Water	-4.0 feet NGVD
Mean High Water	+5.0 feet NGVD
100-year Stillwater	+10.4 feet NGVD
Wave Setup Component	+2.0 feet
Deepwater Wave Height	+28.0 feet
Deepwater Wave Period	+12.7 seconds
Mean Deepwater Wave Height	17.5 feet
Mean Deepwater Wave Period	10.8 seconds
Significant Wave height near seawall	2.2 feet

Table 3 - Summary of Wave Runup Elevations

Transect	Mean Runup Elevation in feet NGVD
A	12.3
B	12.1
C	11.6
D	27.9[1]

[1]Not used for floodplain management purposes.

Table 4 - V-Zone Inland Limit Summary

Transect	WHAFIS		RUNUP		PFD		OTHER		REG	
	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev	Sta	Elev
A	314	15	278[3]	12	700	12[5]	N/A	N/A	700	12
B	200	13	303[3]	12	400	12[1]	328[6]	12[1]	400	12
C	170	15	172[3]	12	N/A	N/A	N/A	N/A	219[9]	16[8]
D	300	15	300[4]	21[2]	N/A	N/A	325[7]	21[2]	325	15[8]
E	704	13	N/A	N/A	N/A	N/A	N/A	N/A	704	13

Station zero is located at 0 NGVD

[1]Elevation based on runup analysis
[2]Three feet above crest of structure
[3]Eroded ground profile three feet below runup elevation
[4]Station located at the base of the seawall
[5]WHAFIS elevation at eroded dune face
[6]Station set 25 feet landward of eroded dune crest due to excessive overtopping
[7]Station set 25 feet landward of structure crest due to excessive overtopping
[8]Average WHAFIS zone elevation seaward of structure
[9]Zone boundary extended to this station given map scale

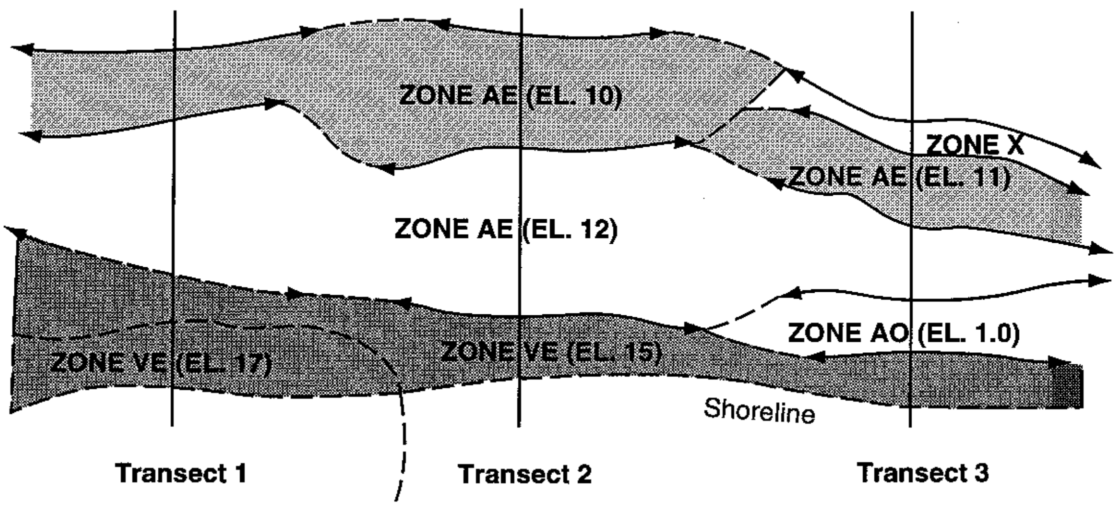


Figure 1 - Zone Mapping Schematic

EXHIBIT INDEX

Index to Exhibit

PART 1: Stillwater Elevation Determination

PART 2: Waves Characteristics and Wave Setup

PART 3: Erosion/Scour

PART 4: Wave Runup/Overtopping

PART 5: Wave Height Analysis, WHAFIS

PART 6: Wave Envelopes

List of Figures

List of Tables

List of Worksheets

PART 1: STILLWATER ELEVATION DETERMINATION

Table 1-1: Summary of Water Surface Elevations

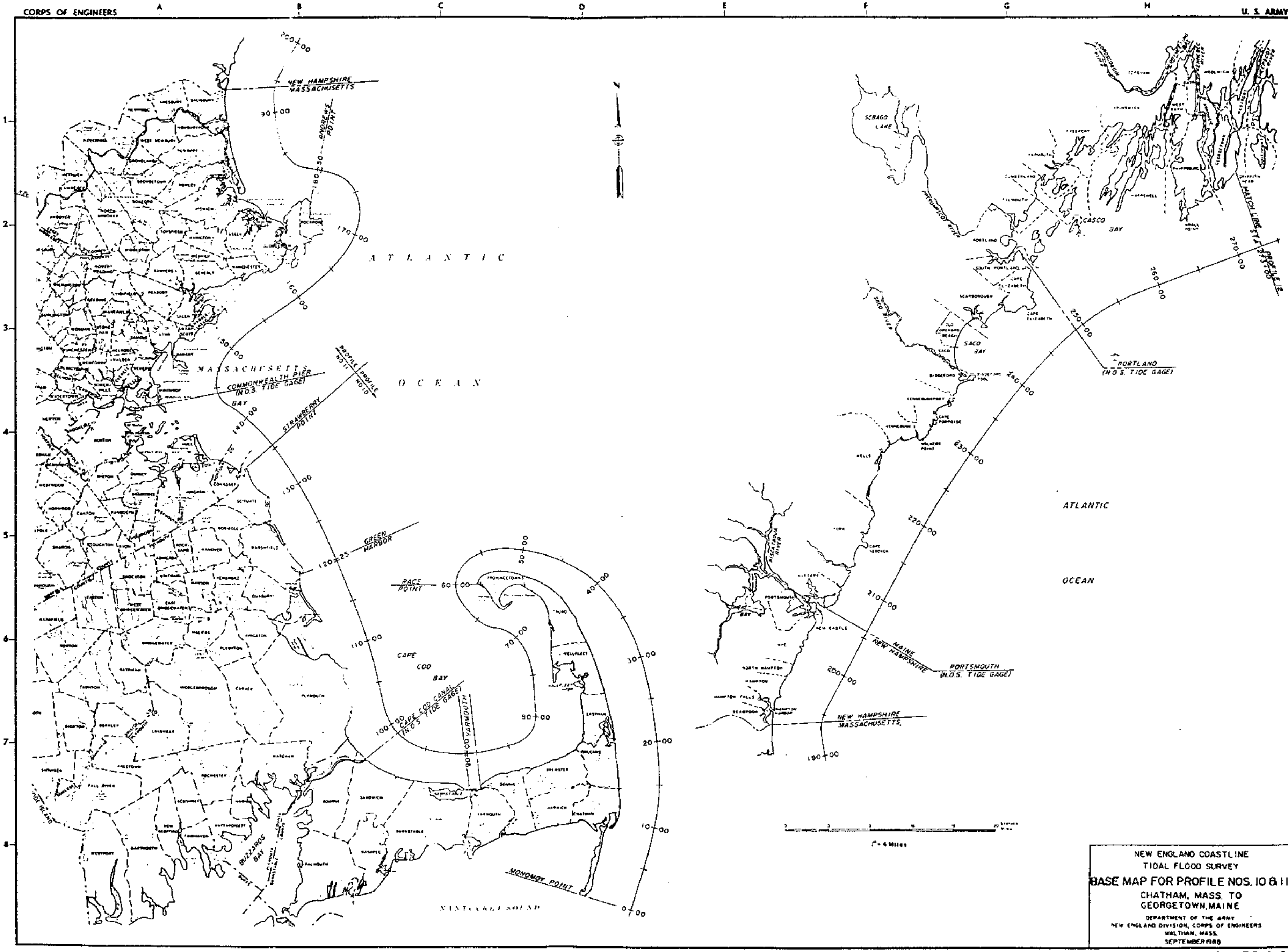
Water Level	Elevation (ft - NGVD)	Source
100-year SWFL (w/o setup)	10.4	Corps 1988 Report
Mean High Water	+5	Corps 1988 Report
Mean Low Water	-4	Corps 1988 Report
100-year SWFL (w/o setup)	10.4	FIS (1991)

Pages of Tidal Flood Profiles USACE Report (1988)

TIDAL FLOOD PROFILES
NEW ENGLAND COASTLINE

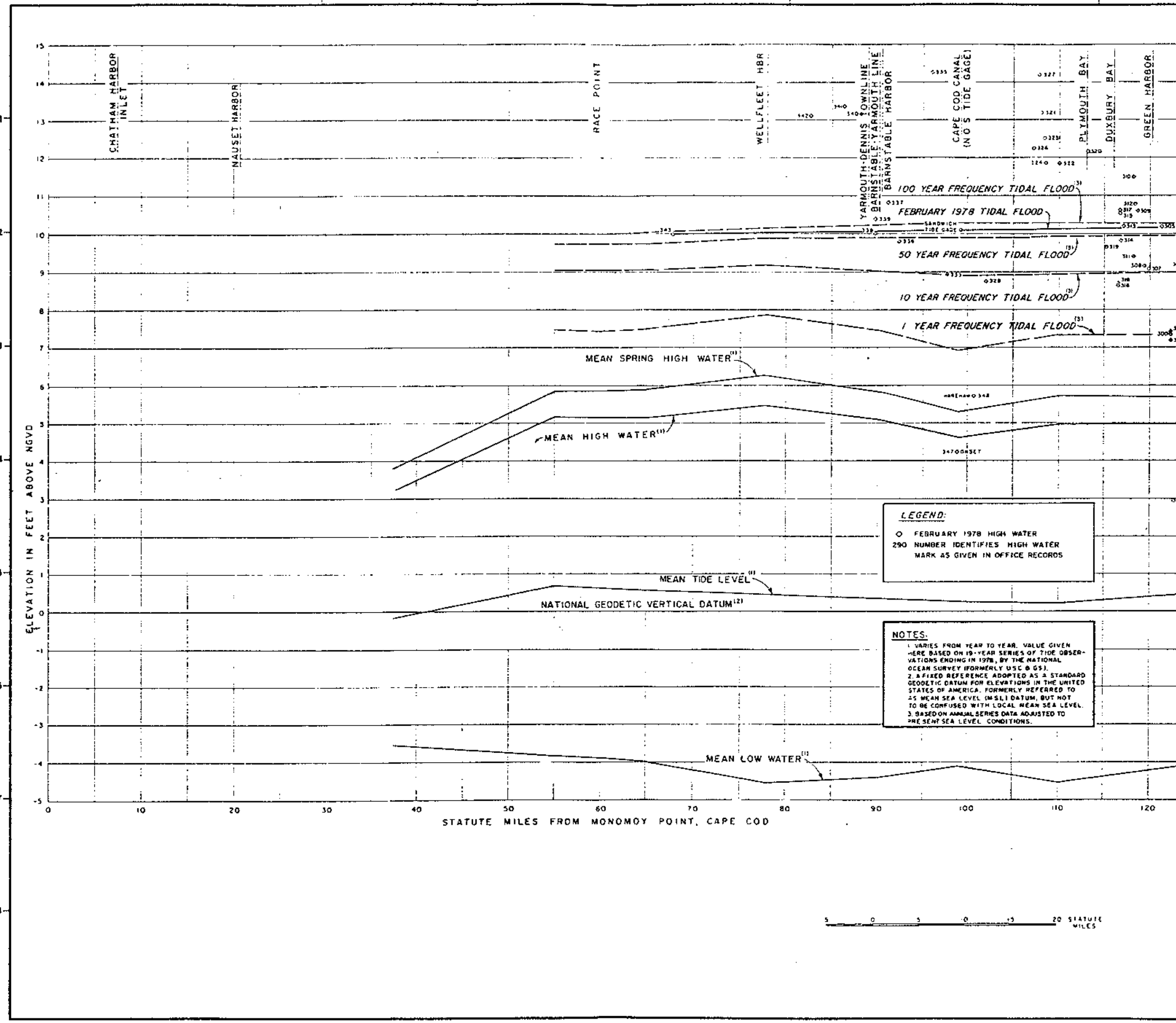
PREPARED BY THE
HYDRAULICS AND WATER QUALITY SECTION
NEW ENGLAND DIVISION
U.S. ARMY CORPS OF ENGINEERS

SEPTEMBER 1988



NEW ENGLAND COASTLINE
TIDAL FLOOD SURVEY
BASE MAP FOR PROFILE NOS. 10 & 11
CHATHAM, MASS. TO
GEORGETOWN, MAINE
DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.
SEPTEMBER 1988

PLATE C-24



NEW ENGLAND COASTLINE
TIDAL FLOOD SURVEY

TIDAL FLOOD PROFILE NO. 10

CHATHAM, MASS.
TO COHASSET, MASS.

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS.
SEPTEMBER 1988

Pages of FIS Report (1991)

FLOOD INSURANCE STUDY

TOWN OF
SMITHVILLE, MASSACHUSETTS
YOUR COUNTY

Federal Emergency Management Agency

COMMUNITY NUMBER - 123456

TABLE 1 - SUMMARY OF STILLWATER ELEVATIONS

FLOODING SOURCE AND LOCATION	ELEVATION (feet) 10-YEAR	50-YEAR	100-YEAR	500-YEAR
CAPE COD BAY				
At Smithville Corporate limits	9.1	10.0	10.4	11.4

The effects of wave action were also considered in the determination of flood hazard areas. Coastal structures that are located above stillwater flood elevations can still be severely damaged by wave runup, wave-induced erosion, and wave-borne debris. For example, during the northeasters of January and February 1978, considerable damage along the Massachusetts coast was caused by wave activity, even though most of the damaged structures were above the high-water level. The extent of wave runup past stillwater levels depends greatly on the wave conditions and local topography.

Wave heights and corresponding wave crest elevation were determined using the National Academy of Sciences (NAS) methodology (Reference 8). The wave runup was determined using the methodology developed by Stone and Webster Engineering Corporation for FEMA (Reference 9).

3.2 Hydraulic Analyses

Hydraulic analyses, considering storm characteristics and the shoreline and bathymetric characteristics of the flooding source studied, were carried out to provide estimates of the elevations of floods of the selected recurrence intervals along the shoreline.

Coastal high hazard areas are areas of special flood hazards along the open coast that, at a minimum, include primary frontal dunes. During major storms, these dunes receive the full impact of the wave attack and respond in a sacrificial manner; the wave energy spent eroding the dunes will often reduce the wave damage inland. Because of their vulnerability to wave attack and their role as the frontline defense against storms, primary frontal dunes are designated as coastal high hazard areas. Other areas of coastline subject to high velocity wave action are also coastal high hazard areas. The COE has established the 3-foot breaking wave as the criterion for identifying these coastal high hazard areas (Reference 10). The 3-foot wave was determined to be the minimum size wave capable of causing significant damage to conventional wood frame or brick veneer structures.

PART 2: WAVE CHARACTERISTICS AND WAVE SETUP

Table 2-1: Summary of Wave Computations

Method	Fetch (mi)	Uobs (mph)	Hmo (ft)	Tp (sec)	Lo (ft)	Hmo/Lo (-)	Wind dir.
ACES-Restricted	160.0	50	25.2	10.7	586.7	0.0430	5
ACES-Restricted	160.0	50	26.1	10.9	605.5	0.0431	15
ACES-Restricted	160.5	50	26.3	10.9	606.6	0.0434	25
ACES-Restricted	160.5	50	25.6	10.8	592.2	0.0432	35
ACES-Restricted	159.2	50	24.3	10.5	561.8	0.0433	45
ACES-Restricted	159.2	50	22.2	10.1	522.8	0.0425	55
ACES-Restricted	160.5	50	26.3	10.9	608.9	0.0432	22
ACES-Restricted	160.5	60	33.6	12.2	762.8	0.0440	22
ACES-Open Water	160.0[1]	50	28.0	12.7	826.6	0.0339	-
WIS 30, Site 93	-	-	20.3	13	866.1	0.0234	-
WIS 30, Site 94	-	-	24.9	15	1153.1	0.0216	-
WIS 30, Site 95	-	-	30.5	14	1004.5	0.0304	-

[1]Average fetch length of restricted fetch computations, input for ACES

Column 1: Method used to determine Hmo and Tp
Column 2: Fetch length computed by ACES in statue miles
Column 3: Observed overwater wind speed in mile per hour
Column 4: Deepwater wave height in feet
Column 5: Peak wave period in seconds
Column 6: Deepwater wave length in feet ($g*Tp^2/2\pi$)
Column 7: Wave steepness
Column 8: Wind direction from North (compass heading)

Table 2-2: ACES Fetch Values

Fetch Number	Direction	Length (miles)
0	334	19
1	340	58
2	346	58
3	352	59
4	358	109
5	4	124
6	10	140
7	16	147
8	22	162
9	28	178
10	34	22
11	40	25
12	46	25
13	52	24
14	58	22
15	64	20
16	70	22
17	76	23
18	82	22

Column 1: Fetch number corresponding to Figure 2-1
Column 2: Fetch direction from north (compass heading)
Column 3: Fetch length in statue miles

Figure 2-1: ACES Fetches

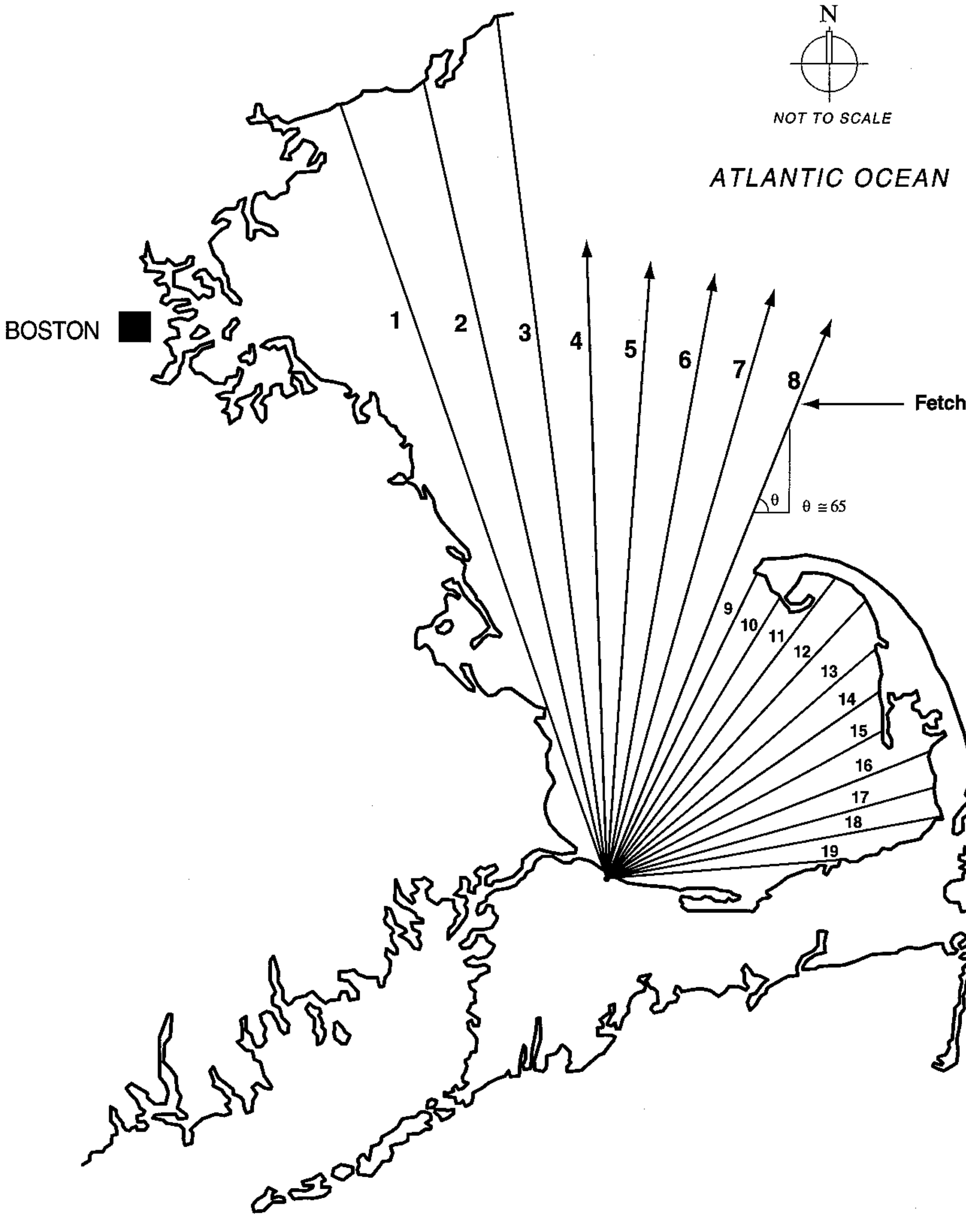

N
NOT TO SCALE
ATLANTIC OCEAN
BOSTON
1
2
3
4
5
6
7
8
Fetch
θ
θ ≅ 65
9
10
11
12
13
14
15
16
17
18
19

Figure 2-1

ACES Output

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	50.00 mph	---------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	160.00 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	49.35 mph	---------------------
Adjusted Wind Speed	Ua:	73.67 mph	Restricted Fetch
Wave Height	HmO:	25.24 ft	Deep-water
Wave Period	Tp:	10.71 sec	Fetch-limited
Wave Direction	Wdir:	5.00 deg	
Mean Wave Direction	Theta:	21.00 deg	

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	50.00 mph	---------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	160.00 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	49.35 mph	---------------------
Adjusted Wind Speed	Ua:	73.67 mph	Restricted Fetch
Wave Height	HmO:	26.11 ft	Deep-water
Wave Period	Tp:	10.87 sec	Fetch-limited
Wave Direction	Wdir:	15.00 deg	
Mean Wave Direction	Theta:	21.00 deg	

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	50.00 mph	---------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	160.49 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	49.35 mph	---------------------
Adjusted Wind Speed	Ua:	73.67 mph	Restricted Fetch
Wave Height	HmO:	26.26 ft	Deep-water
Wave Period	Tp:	10.90 sec	Fetch-limited
Wave Direction	Wdir:	25.00 deg	
Mean Wave Direction	Theta:	22.00 deg	

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	50.00 mph	--------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	160.49 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	49.35 mph	--------------------
Adjusted Wind Speed	Ua:	73.67 mph	Restricted Fetch
Wave Height	HmO:	25.62 ft	Deep-water
Wave Period	Tp:	10.78 sec	Fetch-limited
Wave Direction	Wdir:	35.00 deg	
Mean Wave Direction	Theta:	22.00 deg	

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	50.00 mph	--------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	159.17 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	49.35 mph	--------------------
Adjusted Wind Speed	Ua:	73.67 mph	Restricted Fetch
Wave Height	HmO:	24.28 ft	Deep-water
Wave Period	Tp:	10.53 sec	Fetch-limited
Wave Direction	Wdir:	45.00 deg	
Mean Wave Direction	Theta:	23.00 deg	

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	50.00 mph	--------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	159.17 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	49.35 mph	--------------------
Adjusted Wind Speed	Ua:	73.67 mph	Restricted Fetch
Wave Height	HmO:	22.21 ft	Deep-water
Wave Period	Tp:	10.12 sec	Fetch-limited
Wave Direction	Wdir:	55.00 deg	
Mean Wave Direction	Theta:	23.00 deg	

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	50.00 mph	--------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	160.49 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	49.35 mph	--------------------
Adjusted Wind Speed	Ua:	73.67 mph	Restricted Fetch
Wave Height	HmO:	26.30 ft	Deep-water
Wave Period	Tp:	10.91 sec	Fetch-limited
Wave Direction	Wdir:	22.00 deg	
Mean Wave Direction	Theta:	22.00 deg	

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	60.00 mph	--------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	160.49 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	59.26 mph	--------------------
Adjusted Wind Speed	Ua:	94.17 mph	Restricted Fetch
Wave Height	HmO:	33.62 ft	Deep-water
Wave Period	Tp:	12.15 sec	Fetch-limited
Wave Direction	Wdir:	22.00 deg	
Mean Wave Direction	Theta:	22.00 deg	

WIND ADJUSTMENT AND WAVE GROWTH

Elevation of Observed Wind	Zobs:	30.00 ft	Wind Observation Type
Observed Wind Speed	Uobs:	50.00 mph	--------------------
Air-Sea Temp. Difference	delT:	0.00 deg C	Overwater
Duration of Observed Wind	DurO:	15.00 hr	
Duration of Final Wind	DurF:	20.00 hr	
Latitude of Observation	LAT:	41.70 deg	
Length of Wind Fetch	F:	160.00 mi	Wave Growth Equations
Equiv. Neutral Wind Speed	Ue:	49.35 mph	--------------------
Adjusted Wind Speed	Ua:	73.67 mph	Open-Water Fetch
Wave Height	HmO:	28.01 ft	Deep-water
Wave Period	Tp:	12.71 sec	Fetch-limited

WIS Report 30
March 1993

US Army Corps
of Engineers
Waterways Experiment
Station

Wave Information Studies of US Coastlines

Hindcast Wave Information for the US Atlantic Coast

*by Jon M. Hubertz, Rebecca M. Brooks,
Willie A. Brandon, Barbara A. Tracy
Coastal Engineering Research Center*

Prepared for Headquarters, U.S. Army Corps of Engineers

Pages from WIS Report 30

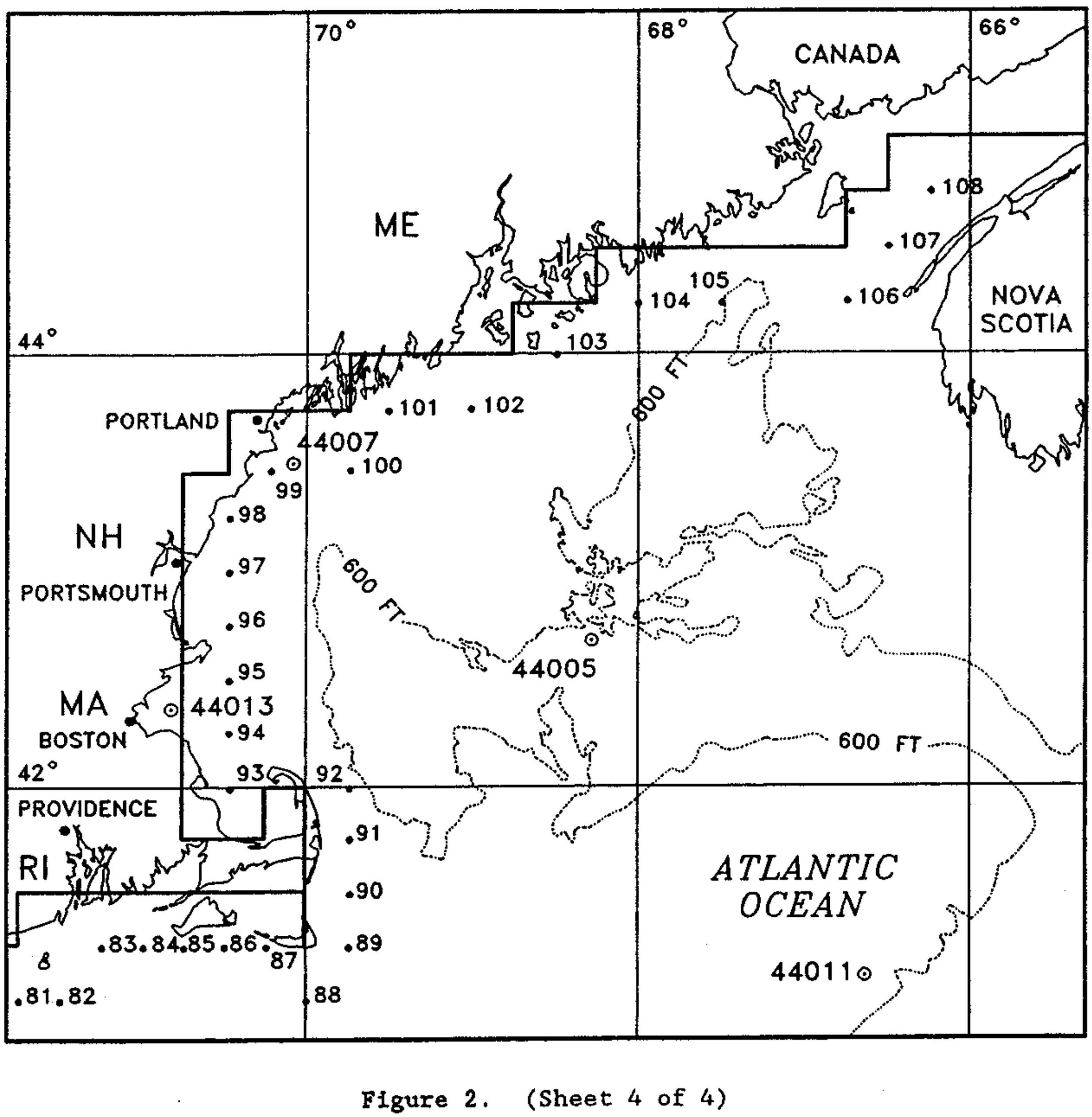


Figure 2. (Sheet 4 of 4)

WIS ATLANTIC REVISION 1956 - 1975
LAT: 42.00 N, LONG: 70.50 W, DEPTH: 18 M

STATION: 93

OCCURRENCES OF WIND DIRECTION BY MONTH FOR ALL YEARS

WD(deg) DIRECTION BAND & CENTER	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
337.50 - 22.49 (0.0)	673	564	616	572	402	276	201	383	522	520	590	657	5976
22.50 - 67.49 (45.0)	319	421	403	246	251	188	125	206	407	439	354	372	3731
67.50 - 112.49 (90.0)	251	301	372	256	268	162	85	139	383	313	298	391	3219
112.50 - 157.49 (135.0)	262	349	416	404	393	337	222	246	318	379	352	355	4033
157.50 - 202.49 (180.0)	386	302	421	542	763	781	786	618	550	554	637	432	6772
202.50 - 247.49 (225.0)	570	426	416	715	1111	1462	1674	1434	896	760	657	587	10708
247.50 - 292.49 (270.0)	1075	914	940	1104	1050	1073	1317	1227	953	1101	912	908	12574
292.50 - 337.49 (315.0)	1424	1243	1376	961	722	521	550	707	771	894	1000	1258	11427
TOTAL	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440

STATION: 93

SUMMARY OF MEAN Hmo(m) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1956	1.79	0.85	1.00	0.67	0.59	0.51	0.50	0.57	0.56	0.73	0.79	0.81	0.78
1957	0.97	0.91	0.90	0.71	0.56	0.43	0.50	0.40	0.51	0.73	0.82	1.11	0.71
1958	1.11	1.11	1.23	1.01	0.56	0.51	0.49	0.49	0.68	0.80	0.91	1.06	0.83
1959	1.15	0.94	1.05	0.63	0.53	0.44	0.49	0.47	0.49	0.72	0.93	1.07	0.74
1960	1.05	1.22	1.21	0.65	0.47	0.58	0.46	0.42	0.54	0.84	0.80	0.96	0.77
1961	1.13	0.85	0.92	0.85	0.66	0.55	0.41	0.43	0.66	1.11	1.01	1.01	0.80
1962	0.95	1.04	1.46	0.72	0.49	0.47	0.46	0.59	0.69	0.78	1.20	1.12	0.83
1963	1.01	1.02	0.97	0.88	0.63	0.52	0.47	0.46	0.80	0.83	1.08	1.07	0.81
1964	1.40	1.26	1.06	0.68	0.67	0.62	0.52	0.55	0.64	0.73	0.88	1.16	0.85
1965	1.33	1.03	0.89	0.71	0.52	0.64	0.47	0.58	0.57	0.82	0.91	0.83	0.77
1966	1.59	0.98	0.79	0.52	0.58	0.54	0.50	0.47	0.67	0.75	0.85	1.23	0.79
1967	0.97	1.22	1.14	1.27	0.93	0.58	0.45	0.47	0.80	0.72	0.98	1.39	0.91
1968	1.34	1.19	1.04	0.90	0.60	0.51	0.46	0.59	0.50	0.81	1.15	1.32	0.87
1969	1.14	1.98	1.27	0.79	0.70	0.50	0.58	0.59	0.57	0.82	1.02	1.46	0.95
1970	1.12	0.94	0.94	0.77	0.64	0.57	0.51	0.56	0.60	0.80	0.88	1.41	0.81
1971	1.13	0.91	1.08	0.89	0.64	0.53	0.56	0.62	0.48	0.64	0.99	1.17	0.80
1972	0.98	1.11	0.99	0.78	0.71	0.60	0.45	0.56	0.79	0.83	1.08	1.03	0.83
1973	1.17	1.36	1.06	0.88	0.62	0.60	0.58	0.47	0.63	0.83	1.01	1.17	0.86
1974	0.90	1.33	1.04	0.92	0.64	0.55	0.50	0.53	0.60	0.86	1.31	1.14	0.85
1975	1.23	0.96	1.35	1.19	0.61	0.68	0.69	0.52	0.66	0.85	1.07	1.52	0.94
MEAN	1.17	1.11	1.07	0.82	0.62	0.55	0.50	0.52	0.62	0.80	0.98	1.15	

STATION: 93

MAX Hmo(m)*10 WITH ASSOCIATED Tp(sec) AND Dp(deg/10) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MAX
1956	6015 4	21 527	4510 1	27 7 6	21 635	11 414	11 422	27 8 2	19 6 1	27 8 2	21 636	27 630	6015 4
1957	28 735	22 7 1	33 836	23 7 0	15 636	14 520	11 420	10 420	14 521	24 7 2	27 8 3	43 9 4	43 9 4
1958	28 735	28 735	4310 4	4710 2	20 6 3	13 5 1	12 429	11 419	30 735	20 7 5	21 632	32 8 1	4710 2
1959	32 732	21 631	33 8 1	17 6 4	11 425	11 5 1	14 512	11 422	24 736	21 613	35 8 2	38 8 1	38 8 1
1960	24 631	34 835	5612 3	28 7 1	11 413	24 7 2	12 515	10 5 3	15 512	3510 4	21 630	5410 1	5612 3
1961	4511 4	46 9 4	28 8 0	31 8 0	15 526	12 420	10 424	10 422	21 6 0	4911 3	34 9 0	32 8 0	4911 3
1962	25 7 1	36 9 4	5411 3	19 636	14 530	11 427	12 428	30 8 1	31 8 1	28 735	5210 0	45 936	5411 3
1963	34 836	38 936	29 735	23 736	17 6 1	15 6 0	12 531	13 429	27 7 0	45 935	27 8 1	32 735	45 935
1964	5011 3	36 836	28 7 3	17 531	12 426	14 534	19 6 0	17 6 1	20 635	20 634	30 8 1	5511 0	5511 0
1965	41 9 4	28 612	27 7 1	26 735	11 425	12 5 5	11 423	12 430	15 535	20 632	21 631	33 836	41 9 4
1966	4410 2	33 8 0	23 7 0	20 7 2	13 531	11 422	11 422	11 424	16 5 6	21 630	20 636	35 8 2	4410 2
1967	25 7 1	38 8 1	35 8 1	5110 1	4510 3	24 8 4	10 422	10 420	24 8 4	25 736	34 835	4810 1	5110 1
1968	4210 3	5211 2	3910 1	18 635	16 5 1	11 414	10 420	11 536	12 5 1	20 633	43 9 1	38 9 0	5211 2
1969	31 8 0	6213 3	4510 3	24 7 1	12 419	11 423	13 5 3	13 521	18 6 1	24 7 1	37 9 3	38 9 1	6213 3
1970	35 836	29 8 0	33 8 1	20 527	18 7 1	11 423	11 425	22 735	12 430	34 8 0	28 7 0	42 936	42 936
1971	34 836	22 6 0	40 9 2	25 735	16 535	10 423	11 419	20 6 0	12 531	20 6 6	25 736	32 8 1	40 9 2
1972	27 735	24 736	20 7 0	29 8 1	34 8 1	16 512	11 419	12 5 2	34 8 1	24 7 1	30 8 1	20 6 5	34 8 1
1973	27 736	38 9 2	36 9 1	19 512	13 5 3	13 536	13 520	11 423	16 523	25 735	19 527	31 9 4	38 9 2
1974	27 7 0	35 835	22 7 0	25 7 1	23 7 0	14 6 3	12 433	13 521	15 528	32 836	5210 3	35 8 1	5210 3
1975	44 9 1	30 8 1	39 9 1	4410 3	35 8 1	23 7 5	27 8 3	17 6 3	20 635	36 8 1	4710 2	5312 4	5312 4
MAX	6015 4	6213 3	5612 3	5110 1	4510 3	24 8 4	27 8 3	30 8 1	34 8 1	4911 3	5210 3	5511 0	

MAX Hmo(m): 6.2 MAX Tp(sec): 13. MAX Dp(deg): 25. DATE(gmt): 69021015

MAX WIND SPEED(m/sec): 26. MAX WIND DIRECTION(deg): 335. DATE(gmt): 64120112

WIS ATLANTIC REVISION, 1956 - 1975
LAT: 42.25 N, LONG: 70.50 W, DEPTH: 27 M

STATION: 94

OCCURRENCES OF WIND DIRECTION BY MONTH FOR ALL YEARS

WD(deg) DIRECTION BAND & CENTER	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
337.50 - 22.49 (0.0)	667	581	635	591	395	267	212	370	519	528	574	662	6001
22.50 - 67.49 (45.0)	315	406	422	243	253	181	113	206	388	410	365	365	3667
67.50 - 112.49 (90.0)	250	299	350	256	258	181	85	130	385	313	288	398	3193
112.50 - 157.49 (135.0)	258	357	429	396	400	327	218	252	315	388	359	351	4050
157.50 - 202.49 (180.0)	400	312	419	551	762	771	786	601	554	542	651	440	6789
202.50 - 247.49 (225.0)	562	427	422	711	1097	1461	1648	1429	909	789	657	597	10709
247.50 - 292.49 (270.0)	1097	913	945	1097	1068	1085	1338	1246	957	1101	939	899	12685
292.50 - 337.49 (315.0)	1411	1225	1338	955	727	527	560	726	773	889	967	1248	11346
TOTAL	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440

STATION: 94

SUMMARY OF MEAN Hmo(m) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1956	2.19	0.97	1.15	0.79	0.64	0.63	0.57	0.63	0.72	0.91	0.93	1.05	0.93
1957	1.09	0.98	1.08	0.80	0.60	0.51	0.53	0.45	0.58	0.87	0.99	1.25	0.81
1958	1.58	1.26	1.53	1.27	0.73	0.64	0.51	0.54	0.72	1.02	1.00	1.19	1.00
1959	1.27	1.00	1.16	0.77	0.58	0.53	0.54	0.57	0.57	0.92	1.07	1.23	0.85
1960	1.13	1.46	1.33	0.72	0.61	0.71	0.50	0.44	0.68	0.95	0.90	1.12	0.88
1961	1.27	0.98	1.10	0.96	0.80	0.60	0.47	0.47	0.85	1.31	1.17	1.09	0.92
1962	1.01	1.25	1.77	0.82	0.65	0.51	0.52	0.72	0.87	0.96	1.38	1.34	0.98
1963	1.16	1.15	1.08	0.94	0.74	0.58	0.52	0.51	1.00	0.96	1.45	1.16	0.93
1964	1.56	1.37	1.21	0.78	0.75	0.68	0.59	0.61	0.76	0.83	1.00	1.40	0.96
1965	1.53	1.17	0.95	0.78	0.57	0.69	0.51	0.63	0.71	0.90	1.06	0.98	0.87
1966	1.82	1.03	0.91	0.60	0.65	0.57	0.57	0.50	0.77	0.84	1.22	1.41	0.91
1967	1.20	1.33	1.25	1.48	1.13	0.66	0.49	0.51	0.93	0.82	1.13	1.62	1.04
1968	1.50	1.33	1.28	1.07	0.69	0.61	0.52	0.60	0.66	0.88	1.36	1.45	1.00
1969	1.31	2.55	1.46	0.91	0.83	0.59	0.70	0.61	0.72	0.93	1.40	1.76	1.14
1970	1.17	1.14	1.05	0.90	0.78	0.65	0.58	0.62	0.64	0.96	1.16	1.66	0.94
1971	1.20	1.10	1.22	1.04	0.80	0.57	0.60	0.66	0.65	0.79	1.16	1.32	0.92
1972	1.07	1.32	1.15	0.85	0.87	0.76	0.49	0.63	0.93	0.93	1.23	1.29	0.96
1973	1.27	1.51	1.27	1.16	0.78	0.67	0.68	0.56	0.71	0.97	1.06	1.36	1.00
1974	0.98	1.39	1.17	1.01	0.80	0.66	0.57	0.60	0.75	0.97	1.49	1.41	0.98
1975	1.52	1.06	1.53	1.32	0.70	0.78	0.79	0.61	0.82	1.00	1.20	1.93	1.11
MEAN	1.34	1.27	1.23	0.95	0.73	0.63	0.56	0.57	0.75	0.94	1.17	1.35	

STATION: 94

MAX Hmo(m)*10 WITH ASSOCIATED Tp(sec) AND Dp(deg/10) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MAX
1956	7615 6	26 619	5110 3	33 8 8	22 635	13 414	16 518	29 8 6	25 7 8	28 8 4	27 810	28 630	7615 6
1957	34 8 8	23 7 3	39 9 8	26 7 2	16 6 0	18 619	14 519	10 423	13 532	25 7 3	30 8 5	48 9 5	48 9 5
1958	4610 9	41 9 8	4911 8	5610 4	22 8 8	16 5 1	13 530	16 519	31 736	31 8 8	23 631	33 8 5	5610 4
1959	3913 9	23 631	41 9 9	18 517	12 519	11 420	16 512	12 8 9	26 7 3	361011	41 9 5	44 9 4	44 9 4
1960	25 631	42 9 9	7012 6	26 7 3	13 412	23 7 5	13 517	11 519	20 710	37 9 5	22 529	6211 4	7012 6
1961	5110 7	6011 6	33 8 9	31 736	21 6 6	16 519	11 432	11 421	32 8 9	5510 4	38 8 1	36 8 5	6011 6
1962	23 617	41 9 7	6713 8	25 7 9	14 530	11 426	12 428	31 7 0	33 8 7	35 7 0	5710 2	5410 2	6713 8
1963	37 8 0	39 835	31 8 8	23 633	19 7 5	15 5 3	12 518	18 516	29 8 5	46 9 1	5110 9	34 735	5110 9
1964	5410 5	41 8 3	29 8 5	18 619	13 425	16 520	16 5 1	17 517	23 636	21 635	34 8 3	6010 1	6010 1
1965	45 9 5	50 911	28 7 3	26 736	12 518	13 519	12 421	13 428	16 535	21 632	24 620	36 836	50 911
1966	5310 7	38 8 0	23 7 1	21 6 2	14 516	11 419	11 421	12 421	21 711	22 630	27 912	40 9 4	5310 7
1967	42 910	41 8 3	38 8 2	6112 7	5210 3	25 8 6	11 418	13 519	25 8 7	25 7 1	36 8 1	53 9 1	6112 7
1968	4710 5	6011 4	4810 3	22 619	25 810	17 7 9	12 519	12 421	1712 9	23 6 2	5010 4	39 836	6011 4
1969	31 736	7513 4	5210 4	23 619	18 518	12 518	19 5 1	12 520	19 6 1	25 7 1	41 9 5	561110	7513 4
1970	36 8 1	33 810	33 7 1	24 811	17 6 3	14 519	15 519	25 7 2	14 518	34 8 1	32 7 1	5110 9	5110 9
1971	36 8 1	28 7 9	4610 8	27 7 1	17 6 2	11 423	15 519	20 6 1	14 8 9	21 7 7	38 9 9	37 8 1	4610 8
1972	28 736	4110 8	23 7 0	31 735	37 9 5	31 8 8	14 518	13 423	43 9 4	28 7 8	32 8 2	34 9 9	43 9 4
1973	30 8 8	41 9 3	40 9 3	38 910	18 517	15 712	17 520	12 6 9	17 519	36 9 8	19 618	35 8 9	41 9 3
1974	28 7 1	36 8 1	30 7 9	31 8 7	24 7 2	16 6 3	12 535	15 8 9	20 617	32 8 1	5812 7	5711 9	5812 7
1975	47 9 1	32 8 3	4410 5	46 9 5	39 8 2	21 6 7	26 7 5	17 6 0	21 635	38 8 2	5310 4	6912 6	6912 6
MAX	7615 6	7513 4	7012 6	6112 7	5210 3	31 8 8	26 7 5	31 7 0	43 9 4	5510 4	5812 7	6912 6	

MAX Hmo(m): 7.6 MAX Tp(sec): 15. MAX Dp(deg): 58. DATE(gmt): 56010918

MAX WIND SPEED(m/sec): 26. MAX WIND DIRECTION(deg): 340. DATE(gmt): 64120112

WIS ATLANTIC REVISION, 1956 - 1975
LAT: 42.50 N, LONG: 70.50 W, DEPTH: 55 M

STATION: 95

OCCURRENCES OF WIND DIRECTION BY MONTH FOR ALL YEARS

WD(deg) DIRECTION BAND & CENTER	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
337.50 - 22.49 (0.0)	678	574	661	622	385	263	218	377	507	530	566	675	6056
22.50 - 67.49 (45.0)	314	401	430	237	252	192	116	203	384	384	371	380	3664
67.50 - 112.49 (90.0)	232	296	348	246	253	161	83	127	359	312	278	384	3079
112.50 - 157.49 (135.0)	277	359	416	398	408	330	206	247	320	378	357	359	4055
157.50 - 202.49 (180.0)	393	312	424	573	744	752	774	594	554	557	679	442	6798
202.50 - 247.49 (225.0)	575	443	433	701	1104	1479	1632	1420	930	814	658	615	10804
247.50 - 292.49 (270.0)	1105	917	919	1081	1065	1079	1355	1256	960	1115	957	893	12702
292.50 - 337.49 (315.0)	1386	1218	1329	942	749	544	576	736	786	870	934	1212	11282
TOTAL	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440

STATION: 95

SUMMARY OF MEAN Hmo(m) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1956	2.33	1.03	1.18	0.83	0.66	0.67	0.59	0.65	0.73	0.92	0.97	1.12	0.97
1957	1.11	0.98	1.08	0.85	0.61	0.57	0.54	0.46	0.60	0.88	1.07	1.33	0.84
1958	1.72	1.27	1.58	1.38	0.80	0.71	0.53	0.57	0.74	1.06	1.04	1.18	1.05
1959	1.33	1.01	1.21	0.80	0.59	0.56	0.59	0.57	0.59	1.02	1.16	1.28	0.89
1960	1.14	1.54	1.35	0.75	0.69	0.75	0.53	0.45	0.71	0.97	0.95	1.20	0.92
1961	1.32	1.02	1.13	0.98	0.85	0.61	0.49	0.49	0.89	1.37	1.21	1.12	0.96
1962	1.06	1.27	1.77	0.86	0.71	0.53	0.54	0.75	0.89	1.02	1.43	1.41	1.02
1963	1.18	1.18	1.14	0.95	0.80	0.59	0.55	0.52	1.02	0.98	1.58	1.19	0.97
1964	1.63	1.41	1.24	0.82	0.78	0.69	0.63	0.65	0.80	0.85	1.05	1.47	1.00
1965	1.58	1.24	0.95	0.82	0.58	0.69	0.54	0.67	0.75	0.96	1.12	1.04	0.91
1966	1.86	1.04	0.97	0.59	0.67	0.59	0.60	0.52	0.80	0.88	1.31	1.44	0.94
1967	1.28	1.37	1.23	1.50	1.19	0.68	0.52	0.53	0.98	0.88	1.18	1.67	1.08
1968	1.53	1.36	1.36	1.12	0.70	0.65	0.56	0.61	0.67	0.90	1.40	1.48	1.03
1969	1.35	2.70	1.50	0.97	0.89	0.64	0.75	0.63	0.77	0.95	1.54	1.89	1.20
1970	1.15	1.23	1.07	0.95	0.86	0.70	0.63	0.64	0.66	1.00	1.20	1.70	0.98
1971	1.22	1.19	1.25	1.07	0.85	0.59	0.62	0.68	0.71	0.82	1.21	1.36	0.96
1972	1.09	1.38	1.22	0.87	0.94	0.86	0.52	0.65	0.97	0.96	1.26	1.34	1.00
1973	1.33	1.54	1.33	1.25	0.85	0.74	0.73	0.57	0.75	0.98	1.09	1.47	1.05
1974	1.00	1.37	1.25	1.05	0.84	0.70	0.59	0.61	0.78	1.01	1.52	1.49	1.02
1975	1.58	1.07	1.56	1.34	0.73	0.81	0.83	0.63	0.87	1.02	1.25	2.04	1.15
MEAN	1.39	1.31	1.27	0.99	0.78	0.67	0.59	0.59	0.78	0.97	1.23	1.41	

STATION: 95

MAX Hmo(m)*10 WITH ASSOCIATED Tp(sec) AND Dp(deg/10) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MAX
1956	9314 7	33 719	5610 7	33 8 9	22 635	16 513	18 618	29 8 5	23 6 8	28 8 5	31 718	29 630	9314 7
1957	34 8 9	22 911	38 8 7	27 813	15 6 1	22 619	16 519	10 536	15 519	26 7 3	33 8 6	51 9 6	51 9 6
1958	5010 9	45 9 8	5711 7	54 9 3	23 810	19 5 1	13 514	18 617	29 7 0	33 8 8	26 715	31 7 1	5711 7
1959	4013 9	24 631	49 910	21 615	15 519	14 518	20 611	12 810	24 7 4	501012	46 9 5	48 9 5	501012
1960	25 631	49 9 9	7211 6	23 6 1	16 512	20 6 3	15 518	14 519	22 612	38 8 5	26 619	6610 4	7211 6
1961	5310 7	6711 7	35 8 9	32 8 7	221012	18 619	11 519	11 421	31 810	55 9 4	39 8 2	38 8 5	6711 7
1962	29 813	42 9 8	6712 8	26 7 9	15 518	11 425	14 519	29 7 2	37 8 7	351110	58 9 0	5612 9	6712 8
1963	36 8 0	38 8 1	33 8 7	23 633	25 614	15 5 3	15 518	22 616	30 8 6	46 834	6211 9	35 8 1	6211 9
1964	5710 7	40 8 3	29 8 4	22 619	15 812	18 619	14 519	22 617	22 8 9	22 635	36 912	6210 4	6210 4
1965	47 9 6	561111	28 7 4	24 635	15 519	13 519	15 519	16 520	19 618	27 812	27 719	39 835	561111
1966	6110 4	42 9 4	23 636	18 6 3	16 516	14 520	12 420	14 519	21 712	22 630	35 912	41 9 5	6110 4
1967	48 910	41 8 5	38 8 3	5613 9	5410 5	26 8 6	12 518	17 519	24 7 7	24 636	34 735	5710 6	5710 6
1968	50 9 4	6410 4	51 9 7	26 615	26 711	18 710	16 520	16 519	17 519	23 6 2	5410 6	39 8 0	6410 4
1969	321010	8813 5	5610 6	25 620	23 618	15 517	20 810	12 422	19 6 1	23 6 1	43 9 5	681110	8813 5
1970	35 8 2	36 811	32 7 2	29 812	20 619	17 619	18 619	25 7 6	17 618	32 735	31 7 3	601010	601010
1971	38 9 5	30 812	5210 9	25 7 3	17 812	12 519	18 619	19 6 3	15 531	21 7 8	3910 9	42 9 4	5210 9
1972	26 7 1	4510 9	26 614	29 7 1	36 7 1	35 8 8	18 618	16 520	4710 7	28 8 8	36 9 4	36 9 9	4710 7
1973	31 736	44 9 3	42 9 4	46 910	22 616	19 712	21 619	12 610	21 618	39 9 9	24 618	36 8 6	46 910
1974	26 619	34 734	29 8 9	32 8 7	25 614	20 712	13 535	17 520	25 618	30 734	6010 4	631110	631110
1975	49 9 4	31 7 2	50 9 5	4511 9	38 9 5	20 618	25 8 8	17 6 0	21 635	34 7 1	6010 6	8412 7	8412 7
MAX	9314 7	8813 5	7211 6	5613 9	5410 5	35 8 8	25 8 8	29 7 2	4710 7	55 9 4	6211 9	8412 7	

MAX Hmo(m): 9.3 MAX Tp(sec): 14. MAX Dp(deg): 65. DATE(gmt): 56010918

MAX WIND SPEED(m/sec): 27. MAX WIND DIRECTION(deg): 345. DATE(gmt): 64120112

Worksheet 2-1: Wave Setup Computations (SPM pg. 3-107)

Given:28.0 = Hmo (feet)
12.7 = Tp (seconds)
1/85 = Nearshore slope (see Part 4 of Exhibit)

Compute:Deepwater wave length Lo

Lo = $g*Tp^2/2\pi$ = $(32.2*12.7)/(2*\pi)$ = 826.6 (feet)

Wave Steepness Hmo/Lo

Hmo/Lo = 28.0/826.6 = 0.0339

Using Figure 3-53 on page 3-109 of SPM find

S/Hmo = 0.038 for slope 1/30 at ds/Hmo=0.5
S/Hmo = 0.035 for slope 1/100 at ds/Hmo=0.5

Interpolate S/Hmo for a slope of 1/85

S/Hmo = 0.0352 for slope 1/85 at ds/Hmo=0.5

Compute

2*(S/Hmo at ds/Hmo=0.5) = S/Hmo at shoreline = 0.0705

Compute setup magnitude S in feet

S = Hmo * S/Hmo = 28.0*0.0705 = 1.97 or

<u>Setup is approximately 2.0 feet</u>

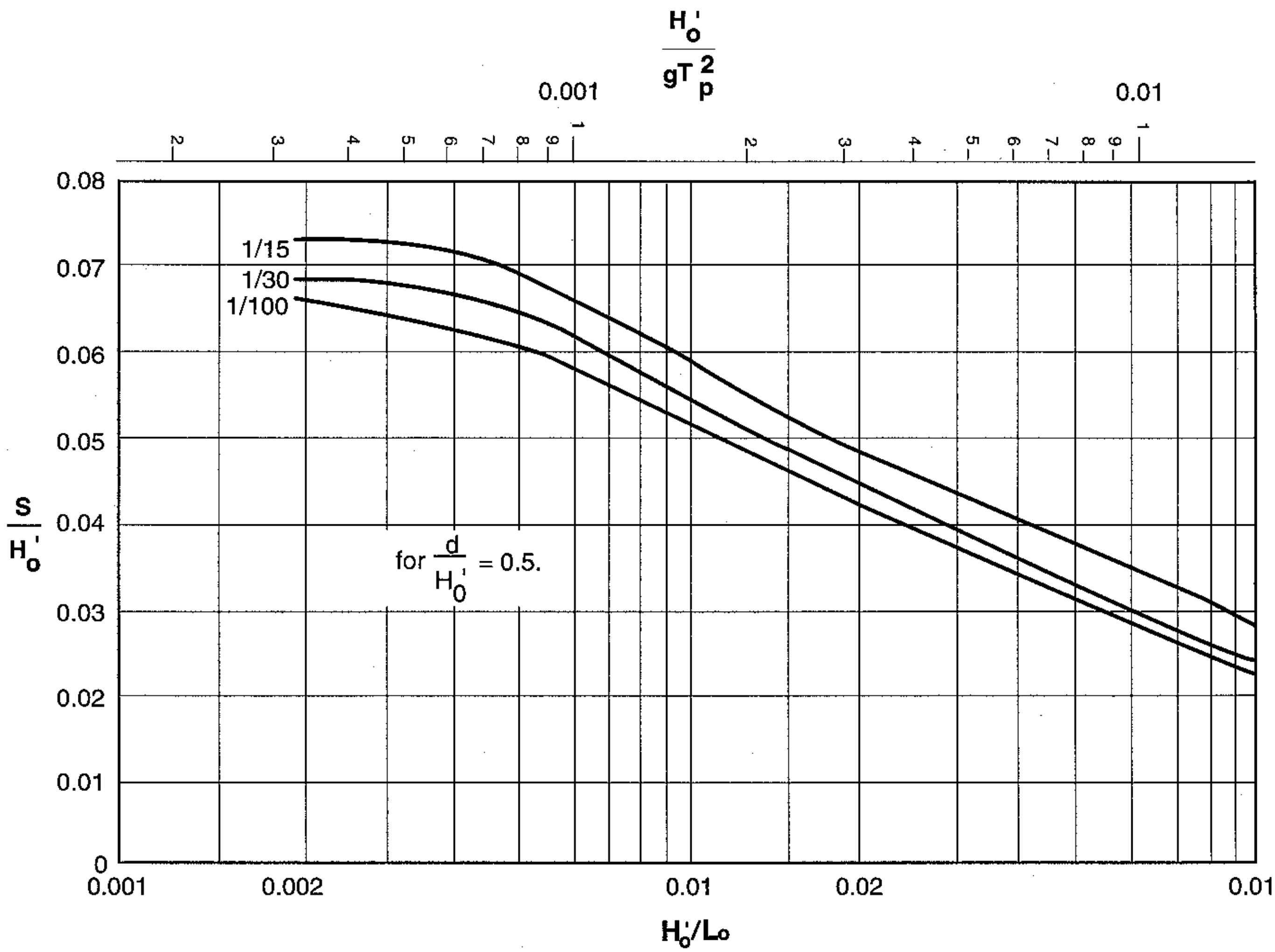


Figure 3-53: Predicted random wave (sea) setup on plane slopes for $\frac{d}{H_0}$ = 0.5.

PART 3: EROSION/SCOUR

Worksheet 3-1: Erosion analysis for Transect A

Pre-Storm Profile

	Station	Elevation
profile-points	-150.0	-4.0
	0.0	0.0
	185.0	5.0
	250.0	10.0
	300.0	20.0
	350.0	25.0
	400.0	20.0
	650.0	10.0
	700.0	8.0
	3500.0	10.4

FEMA's Treatment of sand dune erosion in 100-year event

Unit: ENGLISH
Critical area: 540.0 ft^2
Seaward slope: 1 on 12.5
Approach slope: 1 on 40.0
Face slope: 1 on 1.0
SWFL: 10.4 ft (w/o setup)
Reservoir area: 1440.4 ft^2

Points on profile and eroded profile for case of duneface retreat

Profile			Eroded Profile		
Point	Station	Elevation	Point	Station	Elevation
1	-150.0	-4.0	1	-150.0	-4.0
			start SEAWARD-SLOPE	-99.0	-2.6
			start APPROACH-SLOPE	-53.2	1.0
2	.0	.0	2	(eroded)	
3	185.0	5.0	3	(eroded)	
			intersection with profile	223.2	7.9
4	250.0	10.0	4	(eroded)	
5	300.0	20.0	5	(eroded)	
			start FACE-SLOPE	321.8	10.4
			end erosion	334.8	23.5
6	350.0	25.0	6	350.0	25.0
7	400.0	20.0	7	400.0	20.0
8	650.0	10.0	8	650.0	10.0
9	700.0	8.0	9	700.0	8.0
10	3500.0	10.4	10	3500.0	10.4

Deposition area: 622.68 ft^2

Figure 3-1: Transect A, Pre-Storm/Eroded Ground Elevations

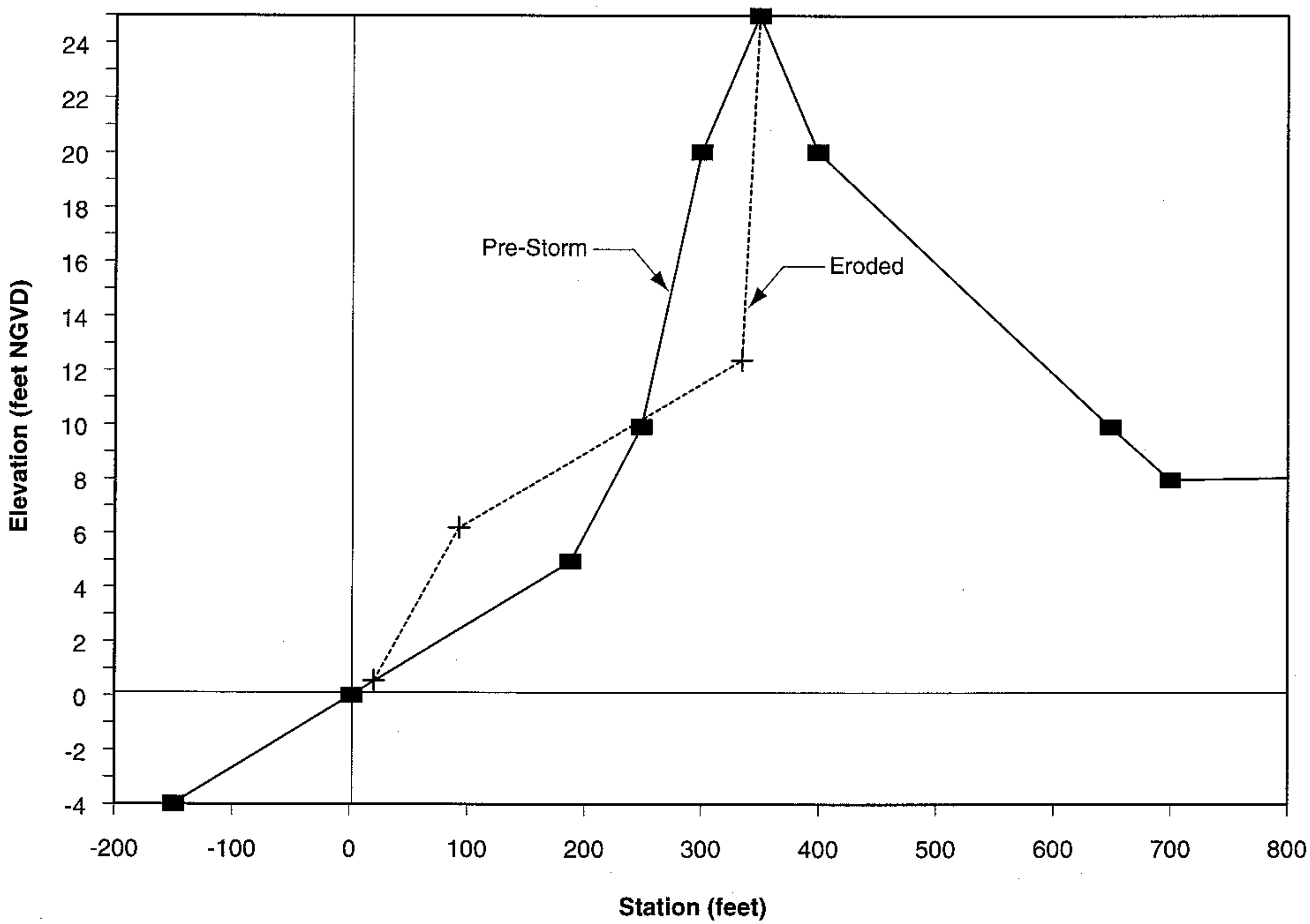


Figure 3-1. Transect A, PFD Retreat

Worksheet 3-2: Erosion analysis for Transect B

Pre-Storm Profile

	Station	Elevation
profile-points	-60.0	-4.0
	0.0	0.0
	75.0	5.0
	100.0	10.0
	200.0	15.0
	275.0	10.0
	400.0	8.0
	3500.0	10.4

FEMA's Treatment of sand dune erosion in 100-year event

Unit: ENGLISH
Critical area: 540.0 ft²
Removal slope: 1 on 50.0
SWFL: 10.4 ft (w/o setup)
Reservoir area: 211.6 ft²
Toe location: point 3

Points on profile and eroded profile for case of dune removal

Profile				Eroded Profile		
Point	Station	Elevation		Point	Station	Elevation
1	-60.0	-4.0		1	-60.0	-4.0
2	.0	.0		2	.0	.0
3	75.0	5.0	start REMOVAL-SLOPE	3	75.0	5.0
4	100.0	10.0		4	(eroded)	
5	200.0	15.0		5	(eroded)	
6	275.0	10.0		6	(eroded)	
				end erosion	302.8	9.6
7	400.0	8.0		7	400.0	8.0
8	3500.0	10.4		8	3500.0	10.4

Dune removal area: 988.89 ft²

Figure 3-2: Transect B, Pre-Storm/Eroded Ground Elevations

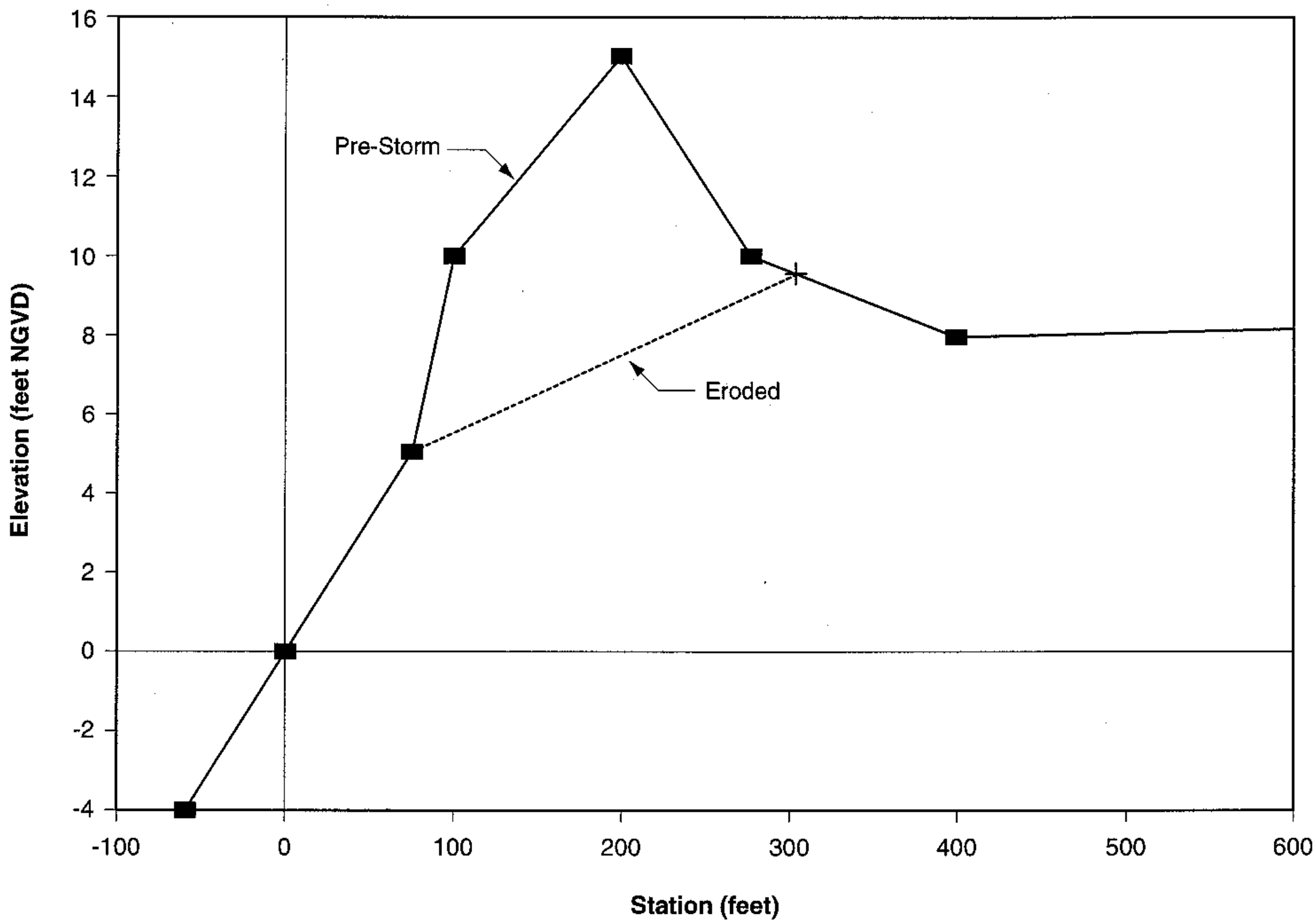


Figure 3-2. Transect B, Dune Removal

Worksheet 3-3: Scour Assessment Transect D

Given: 10.4 = 100-year Stillwater Elevation without setup (ft NGVD
6.0 = Average Ground Elevation from wall base 300 feet seaward (ft NGVD)
12.7 = Tp (seconds)

Compute: Average Water Depth and Shallow Water Wave Length

10.4 - 6.0 = 4.4 feet = average water depth = d
151.2 = Tp * $(g*d)^{1/2}$ = 12.7 * $(32.2 * 4.4)^{1/2}$

Find: Pre-Storm Ground Elevation 1 Wave Length from the Wall

From Figure 3-3 pre-storm ground elevation is approximately = 5.9 feet NGVD

Compute:

Pre-storm water depth one wavelength seaward of wall is approximately: 10.4 - 5.9 = 4.5 feet

Approximate Hs = 0.78/1.6 * 4.5 feet = 2.2 feet

Approximate scour elevation at wall base is approximately = 6.0 - 2.2 = 3.8 feet NGVD

<u>Scour elevation at wall toe is approximately 3.8 feet NGVD</u>

Figure 3-3: Transect D, Pre-Storm/Eroded Ground Elevations

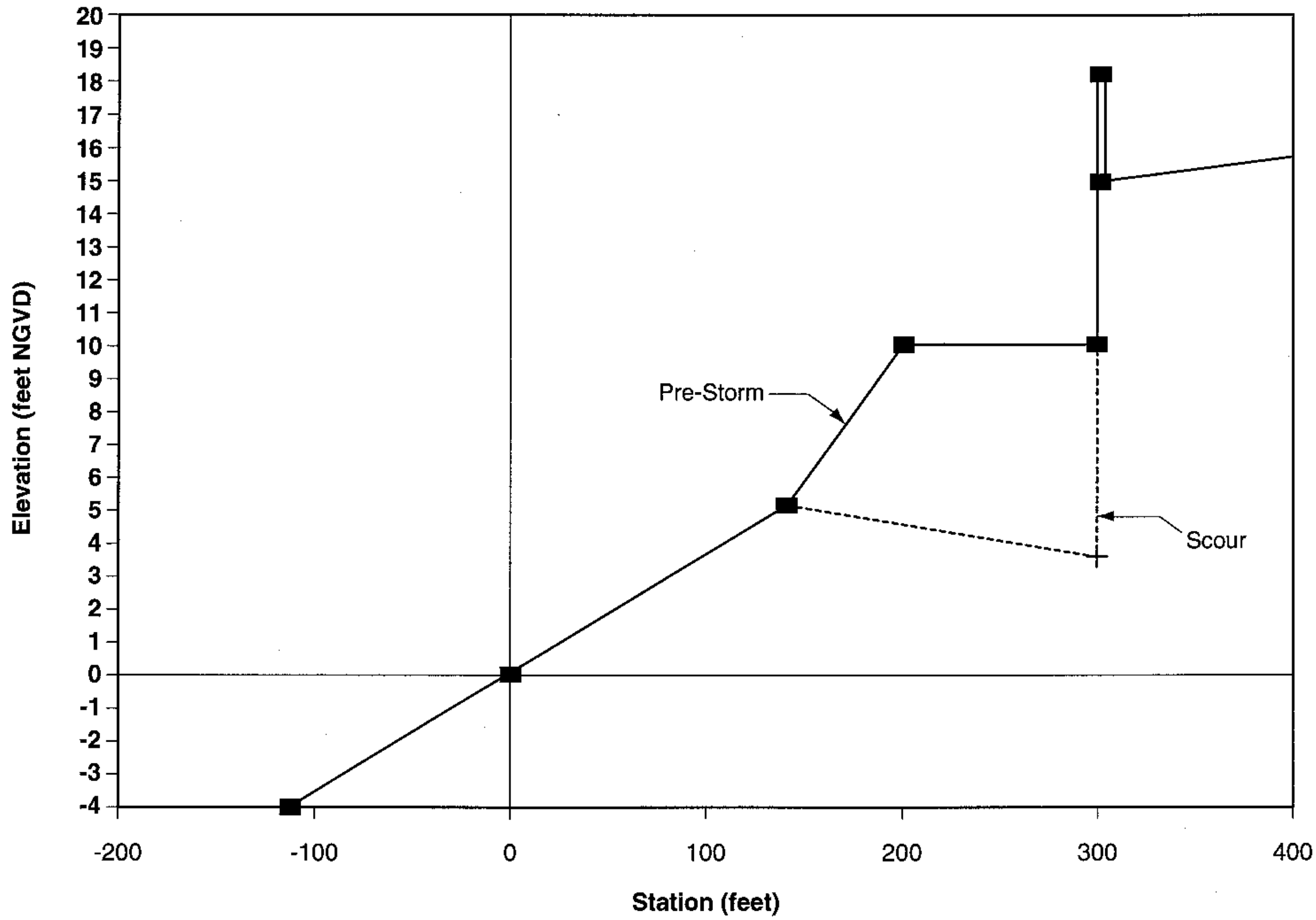


Figure 3-3. Transect D, Seawall

PART 4: WAVE RUNUP/OVERTOPPING

Table 4-1: Summary of Runup Elevations

Transect	Slope Number	Station 1 (ft)	Station 2 (ft)	Computed Runup Elevation (ft NGVD)
A	8	322	335	12.3
B	7	303	-	12.1
C	8	200	220	11.6
D	-	300	300	27.9[1]
E	N/A	N/A	N/A	N/A

[1]Runup elevation exceeds structure crest by more than 3 feet

Column 1: Transect letter
Column 2: Slope number from RUNUP model output
Column 3 and 4: Stations between which water is running up
Column 5: Computed runup elevation (10.4 feet plus average runup magnitude from RUNUP model output)

All sounding taken from USGS quadrangle

Table 4-2: Average Distance to -6 foot sounding (-10 feet NGVD)

Transect	Station (ft from 0 NGVD)
A	-570
B	-900
C	-550
D	-250
E	-650
Avg.	-585

Table 4-3: Average Distance to -18 foot sounding (-22 feet NGVD)

Transect	Station (ft from 0 NGVD)
A	-1500
B	-1500
C	-1600
D	-1400
E	-1500
Avg.	-1500

Table 4-4: Average Distance to -30 foot sounding (-34 feet NGVD)

Transect	Station (ft from 0 NGVD)
A	-2400
B	-2900
C	-2400
D	-2800
E	-2850
Avg.	-2670

Table 4-5: Summary of Average Nearshore Bathymetry

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-585	-10
-1500	-22
-2670	-34

Transect A, RUNUP model input and output

FILE: G&S_TAR.IN

```
  Transect A, G&S Example, Dune Retreat
  Runup calculations on Eroded Transect
1.0
  -34.0 -2670.0 1.0
  -22.0 -1500.0 1.0
  -10.0  -585.0 1.0
   -4.0  -150.0 1.0
   -2.6   -99.0 1.0
    1.0    53.0 1.0
    7.9   223.0 1.0
   10.4   322.0 1.0
   23.5   335.0 1.0
1  25.0   350.0 1.0
 10.4   16.6  10.3
 10.4   16.6  10.8
 10.4   16.6  11.3
 10.4   17.5  10.3
 10.4   17.5  10.8
 10.4   17.5  11.3
 10.4   18.4  10.3
 10.4   18.4  10.8
 10.4   18.4  11.3
```

FILE: G&S_TAR.OUT

```
CLIENT- Transect A, G&S Example, D** WAVE RUNUP-VERSION 2.0  ** ENGINEERED BY          JOB
PROJECT-Runup calculations on Eroded Transect                                   RUN      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
```

****

OUTPUT TABLE

INPUT PARAMETERS			RUNUP RESULTS			
WATER LEVEL ABOVE DATUM (FT.)	DEEP WATER WAVE HEIGHT (FT.)	WAVE PERIOD (SEC.)	BREAKING SLOPE NUMBER	RUNUP SLOPE NUMBER	RUNUP ABOVE WATER LEVEL (FT.)	BREAKER DEPTH (FT.)
10.40	16.60	10.30	2	8	1.66	26.46
10.40	16.60	10.80	2	8	1.83	26.83
10.40	16.60	11.30	2	8	1.99	27.20
10.40	17.50	10.30	2	8	1.75	27.69
10.40	17.50	10.80	2	8	1.92	28.07
10.40	17.50	11.30	2	8	1.92	28.45
10.40	18.40	10.30	2	8	1.84	28.91
10.40	18.40	10.80	2	8	1.84	29.30
10.40	18.40	11.30	2	8	2.02	29.69
					Average 1.89	

Transect B, RUNUP model input and output

FILE: G&S_TBR.IN

```
  Transect B, G&S Example, Dune Removal
  Runup calculations on Un-eroded Transect
50.0
  -34.0 -2670.0 1.0
  -22.0 -1500.0 1.0
  -10.0  -585.0 1.0
   -4.0   -60.0 1.0
    0.0     0.0 1.0
    5.0    75.0 1.0
1   9.6   303.0 1.0
 10.4   16.6  10.3
 10.4   16.6  10.8
 10.4   16.6  11.3
 10.4   17.5  10.3
 10.4   17.5  10.8
 10.4   17.5  11.3
 10.4   18.4  10.3
 10.4   18.4  10.8
 10.4   18.4  11.3
```

FILE: G&S_TBR.OUT

```
CLIENT- Transect B, G&S Example, D      ** WAVE RUNUP-VERSION 2.0  **         ENGINEERED BY            JOB
PROJECT-Runup calculations on Un-eroded Transect                                                    RUN         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		
			87.50	1.00
4	-60.0	-4.0		
			15.00	1.00
5	.0	.0		
			15.00	1.00
6	75.0	5.0		
			49.57	1.00
7	303.0	9.6		

LAST SLOPE 50.00 LAST ROUGHNESS 1.00

OUTPUT TABLE

INPUT PARAMETERS			RUNUP RESULTS			
WATER LEVEL ABOVE DATUM (FT.)	DEEP WATER WAVE HEIGHT (FT.)	WAVE PERIOD (SEC.)	BREAKING SLOPE NUMBER	RUNUP SLOPE NUMBER	RUNUP ABOVE WATER LEVEL (FT.)	BREAKER DEPTH (FT.)
10.40	16.60	10.30	2	7	1.66	26.46
10.40	16.60	10.80	2	7	1.66	26.83
10.40	16.60	11.30	2	7	1.66	27.20
10.40	17.50	10.30	2	7	1.58	27.69
10.40	17.50	10.80	2	7	1.75	28.07
10.40	17.50	11.30	2	7	1.75	28.45
10.40	18.40	10.30	2	7	1.66	28.91
10.40	18.40	10.80	2	7	1.66	29.30
10.40	18.40	11.30	2	7	1.84	29.69
					Average 1.69	

Transect C, RUNUP model input and output

FILE: G&S_TCR.IN

```
  Transect C, G&S Example, Bluff
  Runup calculations on Un-eroded Transect
8.0
  -34.0 -2670.0 1.0
  -22.0 -1500.0 1.0
  -10.0  -585.0 1.0
   -4.0   -80.0 1.0
    0.0     0.0 1.0
    5.0   100.0 0.9
    6.0   120.0 0.6
   10.0   200.0 0.6
   12.5   220.0 0.6
1  20.0   280.0 0.9
 10.4   16.6  10.3
 10.4   16.6  10.8
 10.4   16.6  11.3
 10.4   17.5  10.3
 10.4   17.5  10.8
 10.4   17.5  11.3
 10.4   18.4  10.3
 10.4   18.4  10.8
 10.4   18.4  11.3
```

FILE: G&S_TCR.OUT

```
CLIENT- Transect C, G&S Example, B** WAVE RUNUP-VERSION 2.0  ** ENGINEERED BY          JOB
PROJECT-Runup calculations on Un-eroded Transect                                 RUN      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		
			84.17	1.00
4	-80.0	-4.0		
			20.00	1.00
5	.0	.0		
			20.00	1.00
6	100.0	5.0		
			20.00	.90
7	120.0	6.0		
			20.00	.60
8	200.0	10.0		
			8.00	.60
9	220.0	12.5		
			8.00	.60
10	280.0	20.0		

LAST SLOPE 8.00 LAST ROUGHNESS .90

****

OUTPUT TABLE

INPUT PARAMETERS			RUNUP RESULTS			
WATER LEVEL ABOVE DATUM (FT.)	DEEP WATER WAVE HEIGHT (FT.)	WAVE PERIOD (SEC.)	BREAKING SLOPE NUMBER	RUNUP SLOPE NUMBER	RUNUP ABOVE WATER LEVEL (FT.)	BREAKER DEPTH (FT.)
10.40	16.60	10.30	2	8	1.10	26.46
10.40	16.60	10.80	2	8	1.20	26.83
10.40	16.60	11.30	2	8	1.20	27.20
10.40	17.50	10.30	2	8	1.10	27.69
10.40	17.50	10.80	2	8	1.21	28.07
10.40	17.50	11.30	2	8	1.21	28.45
10.40	18.40	10.30	2	8	1.10	28.91
10.40	18.40	10.80	2	8	1.10	29.30
10.40	18.40	11.30	2	8	1.21	29.69
					Average 1.15	

Worksheet 4-1: Wave Runup Calculations for Transect D (SPM pg.7-25)

Given:

28.0 = Hmo (feet)
12.7 = Tp (seconds)
2.0 = Structure Toe Elevation (feet NGVD)
10.4 = Stillwater Elevation (w/o wave setup)

Compute:

8.4 = Water Depth at Structure Toe (10.4 - 2.0) = ds
17.5 = H bar = 0.625 * 28.0 (feet)
10.8 = T bar = 0.85 * 12.7 (seconds)
0.0047 = H bar/(g*Tp^2)
0.4792 = ds/H bar

From Figure 7-14, page 7-25 of SPM find R/H bar (assume slope of 1/10)

1.0 = R/H bar

Compute Runup Magnitude = R/H bar * H bar = 1 * 17.5 = 17.5 feet

Compute Runup Elevation = 17.5 + 10.4 = 27.9 feet NGVD

<u>Runup Elevation = 27.9 feet NGVD</u>[1]

[1] Computed runup elevation is greater than 3 feet above structure crest

Worksheet 4-2: Overtopping Assessment for Transect D

Given: 18.2 = Eroded crest of structure, dune, or bluff (feet NGVD)
10.4 = 100-year stillwater elevation w/o setup (feet NGVD)
8.4 = Water depth at structure toe (feet) = dt
28.0 = Incident Hos = Hmo for this case

Compute:

7.8 = Freeboard available = 18.2 - 10.4 (feet)
0.3 = dt/Hos

Find from Figure 18 in the <u>Guidelines and Specifications for Wave Elevation determination and V-Zone Mapping</u>

0.34 = F/Hos for Qbar = 1 cfs/ft
1.25 = F/Hos for Qbar = 0.01 cfs/ft

Compute:

9.5 = Freeboard required for Qbar = 1.0 cfs/ft (feet)
35.0 = Freeboard required for Qbar = 0.01 cfs/ft (feet)

Conclusions:

Overtopping exceeds 1 cfs/ft, V-Zone should extend at a minimum 25 feet landward of wall crest, and the wave crest immediately seaward of wall should be extended to this point. Zones AO (depth 2 feet) and AO (depth 1 foot) should be delineated if appropriate.

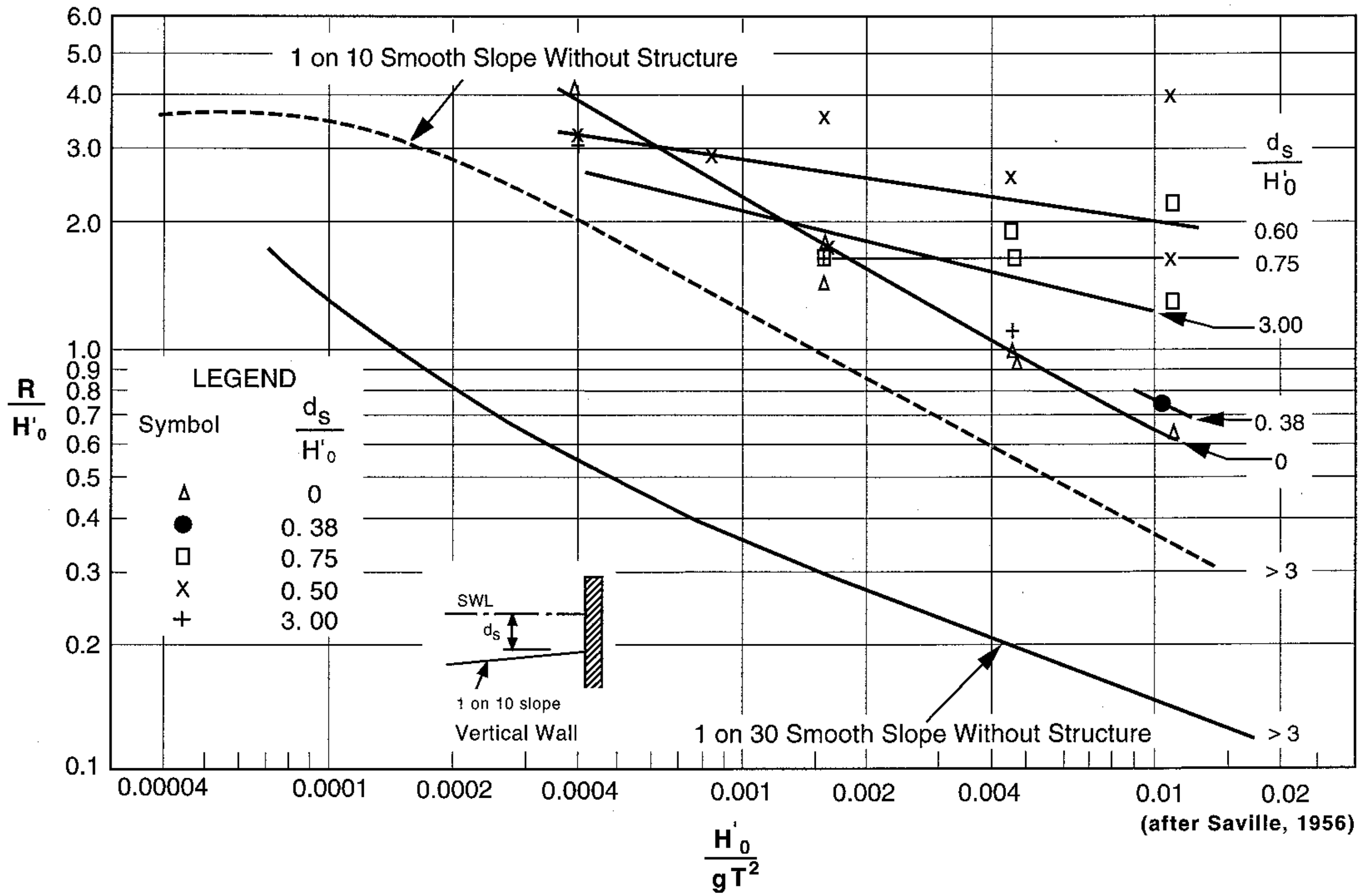


Figure 14. Wave Runup Guidance from Vertical Wall, From Reference 12.

PART 5: WAVE HEIGHT ANALYSIS (WHAFIS)

WHAFIS input and output

FILE: WHAFIS.IN

```
  G&S Example, Transect A (Dune Retreat)
IE      0     0.00    0.000    8.900  12.400   28.000   12.700
IF     13     1.00
IF    289     7.90
IF    388    10.40
IF    390    12.40
AS    706    10.40            10.4000
VH    716    10.00    2        1        0        1
MG   SPAT
VH    766     8.00    2        1        0        1
MG   SPAT
VH   3566    10.40    2        1        0        1
MG   SPAT
ET
  G&S Example, Transect B (Dune Removal)
IE      0     0.00    0.000    8.900  12.400   28.000   12.700
IF     75     5.00
IF    303     9.60
VH    400     8.00    2        1        0        1                 10.400
MG   SPAT
VH   3500    10.40    2        1        0        1
MG   SPAT
ET
  G&S Example, Transect C (Bluff)
IE      0     0.00    0.000    8.900  12.400   28.000   12.700
IF    100     5.00
IF    200    10.00
IF    219    12.40
ET
  G&S Example, Transect D (Seawall)
IE      0     0.00    0.000    8.900  12.400   28.000   12.700
IF    140     5.00
IF    300     3.80
IF    301    12.40
ET
  G&S Example, Transect E (Marsh)
IE      0     0.00    0.000    8.900  12.400   28.000   12.700
IF    165     5.00
VH    500     5.40    2        1        0        1                 10.400
MG   SPAT
VH   4200    10.40    2        1        0        1
MG   SPAT
ET
```

FILE: WHAFIS.OUT

1 *** THE FOLLOWING MESSAGES ARE THE RESULTS FROM THE 100-YR ELEVATION INTERPOLATION FOR THE TRANSECT:

G&S Example, Transect A (Dune Retreat)

1 *** THE FOLLOWING MESSAGES ARE THE RESULTS FROM THE 100-YR ELEVATION INTERPOLATION FOR THE TRANSECT:

G&S Example, Transect B (Dune Removal)

1 *** THE FOLLOWING MESSAGES ARE THE RESULTS FROM THE 100-YR ELEVATION INTERPOLATION FOR THE TRANSECT:

G&S Example, Transect C (Bluff)

1 *** THE FOLLOWING MESSAGES ARE THE RESULTS FROM THE 100-YR ELEVATION INTERPOLATION FOR THE TRANSECT:

G&S Example, Transect D (Seawall)

1 *** THE FOLLOWING MESSAGES ARE THE RESULTS FROM THE 100-YR ELEVATION INTERPOLATION FOR THE TRANSECT:

G&S Example, Transect E (Marsh)

1

WAVE HEIGHT COMPUTATIONS FOR FLOOD INSURANCE STUDIES (VERSION 3.0, 9_88)
G&S Example, Transect A (Dune Retreat)

PART1 INPUT

IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.077	.000
IF	13.000	1.000	.000	12.400	.000	.000	.000	.000	.027	.000
IF	289.000	7.900	.000	12.400	.000	.000	.000	.000	.025	.000
IF	388.000	10.400	.000	12.400	.000	.000	.000	.000	.045	.000
IF	390.000	12.400	.000	12.400	.000	.000	.000	.000	1.000	.000
AS	706.000	10.400	.000	10.400	.000	.000	.000	.000	-.040	.000
VH	716.000	10.000	2.000	1.000	.000	1.000	.000	10.400	-.040	.000
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000
VH	766.000	8.000	2.000	1.000	.000	1.000	.000	10.400	.000	.000
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000
VH	3566.000	10.400	2.000	1.000	.000	1.000	.000	10.400	.001	.000
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000
ET	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000

1

	END STATION	END ELEVATION	FETCH LENGTH	SURGE ELEV 10-YEAR	SURGE ELEV 100-YEAR	INITIAL WAVE HEIGHT	INITIAL W. PERIOD		BOTTOM SLOPE	AVERAGE A-ZONES
IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.077	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	13.000	1.000	.000	12.400	.000	.000	.000	.000	.027	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	289.000	7.900	.000	12.400	.000	.000	.000	.000	.025	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	388.000	10.400	.000	12.400	.000	.000	.000	.000	.045	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	390.000	12.400	.000	12.400	.000	.000	.000	.000	1.000	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
AS	706.000	10.400	.000	10.400	.000	.000	.000	.000	-.040	.000

	END STATION	END ELEVATION	REGION 1	REGION 1 WEIGHT	REGION 2	NO. OF PLANT TYPES	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR	BOTTOM SLOPE	AVERAGE A-ZONES
VH	716.000	10.000	2.000	1.000	.000	1.000	.000	10.400	-.040	.000

	PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO	
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000

PLANT CHARACTERISTICS INCLUDING VALUES SUPPLIED BY THE PROGRAM

PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO
SPAT	.100	1.000	.850	327.000	.023	.011	.011	1.380

	END STATION	END ELEVATION	REGION 1	REGION 1 WEIGHT	REGION 2	NO. OF PLANT TYPES	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR	BOTTOM SLOPE	AVERAGE A-ZONES
VH	766.000	8.000	2.000	1.000	.000	1.000	.000	10.400	.000	.000

	PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO	
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000

PLANT CHARACTERISTICS INCLUDING VALUES SUPPLIED BY THE PROGRAM

PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO
SPAT	.100	1.000	.850	327.000	.023	.011	.011	1.380

	END STATION	END ELEVATION	REGION 1	REGION 1 WEIGHT	REGION 2	NO. OF PLANT TYPES	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR	BOTTOM SLOPE	AVERAGE A-ZONES
VH	3566.000	10.400	2.000	1.000	.000	1.000	.000	10.400	.001	.000

	PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO	
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000

PLANT CHARACTERISTICS INCLUDING VALUES SUPPLIED BY THE PROGRAM

PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO
SPAT	.100	1.000	.850	327.000	.023	.011	.011	1.380

END OF TRANSECT

NOTE:

SURGE ELEVATION INCLUDES CONTRIBUTIONS FROM ASTRONOMICAL AND STORM TIDES.

1

PART2: CONTROLLING WAVE HEIGHTS, SPECTRAL PEAK WAVE PERIOD, AND WAVE CREST ELEVATIONS

LOCATION		CONTROLLING WAVE HEIGHT	SPECTRAL PEAK WAVE PERIOD	WAVE CREST ELEVATION
IE	.00	9.45	12.70	19.01
IF	13.00	8.70	12.70	18.49
	116.50	6.76	12.70	17.13
	220.00	4.80	12.70	15.76
IF	289.00	3.48	12.70	14.84
IF	388.00	1.55	12.70	13.49
IF	390.00	.01	12.70	12.41
AS	706.00	.00	.00	10.40
VH	716.00	.07	.31	10.45
VH	766.00	.23	.57	10.56
	876.00	.47	.80	10.73
	1036.00	.71	.99	10.90
	1196.00	.86	1.13	11.00
	1436.00	.97	1.29	11.08
	1756.00	.95	1.45	11.07
	2396.00	.70	1.70	10.89
	2716.00	.53	1.80	10.77
	2876.00	.44	1.84	10.71
	3036.00	.34	1.89	10.64
	3196.00	.24	1.89	10.57
	3356.00	.14	1.89	10.50
	3516.00	.03	1.89	10.42
VH	3566.00	.01	1.89	10.41

PART3 LOCATION OF AREAS ABOVE 100-YEAR SURGE

BETWEEN 390.00 AND 706.00

PART4 LOCATION OF SURGE CHANGES

STATION	10-YEAR SURGE	100-YEAR SURGE
706.00	8.90	10.40

PART5 LOCATION OF V ZONES

STATION OF GUTTER	LOCATION OF ZONE
313.68	WINDWARD

PART6 NUMBERED A ZONES AND V ZONES

STATION OF GUTTER	ELEVATION	ZONE DESIGNATION	FHF
.00	19.01		
		V11 EL=19	55
12.80	18.50		
		V11 EL=18	55
88.49	17.50		
		V11 EL=17	55
164.14	16.50		
		V11 EL=16	55
239.36	15.50		
		V11 EL=15	55
313.68	14.50		
		V11 EL=15	55
313.68	14.50		
		A 4 EL=14	20
387.11	13.50		
		A 4 EL=13	20
389.83	12.50		
		A 4 EL=12	20
390.00	12.41		
706.00	10.40		
		A 4 EL=10	20
737.90	10.50		
		A 4 EL=11	20
3348.81	10.50		
		A 4 EL=10	20
3566.00	10.41		

ZONE TERMINATED AT END OF TRANSECT

1

WAVE HEIGHT COMPUTATIONS FOR FLOOD INSURANCE STUDIES (VERSION 3.0, 9_88)
G&S Example, Transect B (Dune Removal)

PART1 INPUT

IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.067	.000
IF	75.000	5.000	.000	12.025	.000	.000	.000	.000	.032	.000
IF	303.000	9.600	.000	10.885	.000	.000	.000	.000	.009	.000
VH	400.000	8.000	2.000	1.000	.000	1.000	.000	10.400	.000	.000
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000
VH	3500.000	10.400	2.000	1.000	.000	1.000	.000	10.400	.001	.000
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000
ET	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000

1

	END STATION	END ELEVATION	FETCH LENGTH	SURGE ELEV 10-YEAR	SURGE ELEV 100-YEAR	INITIAL WAVE HEIGHT	INITIAL W. PERIOD		BOTTOM SLOPE	AVERAGE A-ZONES
IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.067	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	75.000	5.000	.000	12.025	.000	.000	.000	.000	.032	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	303.000	9.600	.000	10.885	.000	.000	.000	.000	.009	.000

	END STATION	END ELEVATION	REGION 1	REGION 1 WEIGHT	REGION 2	NO. OF PLANT TYPES	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR	BOTTOM SLOPE	AVERAGE A-ZONES
VH	400.000	8.000	2.000	1.000	.000	1.000	.000	10.400	.000	.000

	PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO	
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000

PLANT CHARACTERISTICS INCLUDING VALUES SUPPLIED BY THE PROGRAM

PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO
SPAT	.100	1.000	.850	327.000	.023	.011	.011	1.380

	END STATION	END ELEVATION	REGION 1	REGION 1 WEIGHT	REGION 2	NO. OF PLANT TYPES	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR	BOTTOM SLOPE	AVERAGE A-ZONES
VH	3500.000	10.400	2.000	1.000	.000	1.000	.000	10.400	.001	.000

	PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO	
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000

PLANT CHARACTERISTICS INCLUDING VALUES SUPPLIED BY THE PROGRAM

PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO
SPAT	.100	1.000	.850	327.000	.023	.011	.011	1.380

--END OF TRANSECT--

NOTE:

SURGE ELEVATION INCLUDES CONTRIBUTIONS FROM ASTRONOMICAL AND STORM TIDES.

1

PART2: CONTROLLING WAVE HEIGHTS, SPECTRAL PEAK WAVE PERIOD, AND WAVE CREST ELEVATIONS

LOCATION		CONTROLLING WAVE HEIGHT	SPECTRAL PEAK WAVE PERIOD	WAVE CREST ELEVATION
IE	.00	9.45	12.70	19.01
IF	75.00	5.41	12.70	15.81
	177.60	3.44	12.70	13.92
	280.20	1.44	12.70	12.01
IF	303.00	1.00	12.70	11.58
VH	400.00	1.24	12.70	11.27
	550.00	1.39	12.70	11.38
	710.00	1.44	12.70	11.41
	870.00	1.43	12.70	11.40
	1190.00	1.35	12.70	11.34
	1830.00	1.01	12.70	11.10
	2150.00	.81	12.70	10.97
	2470.00	.62	12.70	10.83
	2790.00	.43	12.70	10.70
	3110.00	.24	12.70	10.56
	3430.00	.04	12.70	10.43
VH	3500.00	.01	12.70	10.41

PART3 LOCATION OF AREAS ABOVE 100-YEAR SURGE

NO AREAS ABOVE 100-YEAR SURGE IN THIS TRANSECT

PART4 LOCATION OF SURGE CHANGES

STATION	10-YEAR SURGE	100-YEAR SURGE
75.00	8.90	12.02
303.00	8.90	10.89
400.00	8.90	10.40

PART5 LOCATION OF V ZONES

STATION OF GUTTER	LOCATION OF ZONE
200.05	WINDWARD

PART6 NUMBERED A ZONES AND V ZONES

STATION OF GUTTER	ELEVATION	ZONE DESIGNATION	FHF
.00	19.01		
		V10 EL=19	50
12.03	18.50		
		V10 EL=18	50
35.44	17.50		
		V10 EL=17	50
58.84	16.50		
		V10 EL=16	50
75.00	15.81		
		V 9 EL=16	45
91.79	15.50		
		V 9 EL=15	45
146.00	14.50		
		V 8 EL=14	40
200.04	13.50		
		V 7 EL=13	35
200.05	13.36		
		A 4 EL=13	20
253.85	12.50		
		A 4 EL=12	20
303.00	11.58		
		A 4 EL=12	20
328.76	11.50		
		A 4 EL=11	20
400.00	11.27		
		A 4 EL=11	20
3263.31	10.50		
		A 4 EL=10	20
3500.00	10.41		

ZONE TERMINATED AT END OF TRANSECT

1

WAVE HEIGHT COMPUTATIONS FOR FLOOD INSURANCE STUDIES (VERSION 3.0, 9_88)
G&S Example, Transect C (Bluff)

PART1 INPUT

IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.050	.000
IF	100.000	5.000	.000	12.400	.000	.000	.000	.000	.050	.000
IF	200.000	10.000	.000	12.400	.000	.000	.000	.000	.062	.000
IF	219.000	12.400	.000	12.400	.000	.000	.000	.000	.126	.000
ET	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000

1

	END STATION	END ELEVATION	FETCH LENGTH	SURGE ELEV 10-YEAR	SURGE ELEV 100-YEAR	INITIAL WAVE HEIGHT	INITIAL W. PERIOD		BOTTOM SLOPE	AVERAGE A-ZONES
IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.050	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	100.000	5.000	.000	12.400	.000	.000	.000	.000	.050	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	200.000	10.000	.000	12.400	.000	.000	.000	.000	.062	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	219.000	12.400	.000	12.400	.000	.000	.000	.000	.126	.000

--END OF TRANSECT--

NOTE:

SURGE ELEVATION INCLUDES CONTRIBUTIONS FROM ASTRONOMICAL AND STORM TIDES.

1

PART2: CONTROLLING WAVE HEIGHTS, SPECTRAL
PEAK WAVE PERIOD, AND WAVE CREST ELEVATIONS

LOCATION		CONTROLLING WAVE HEIGHT	SPECTRAL PEAK WAVE PERIOD	WAVE CREST ELEVATION
IE	.00	9.45	12.70	19.01
IF	100.00	5.69	12.70	16.38
IF	200.00	1.86	12.70	13.70
IF	219.00	.01	12.70	12.41

PART3 LOCATION OF AREAS ABOVE 100-YEAR SURGE

NO AREAS ABOVE 100-YEAR SURGE IN THIS TRANSECT

PART4 LOCATION OF SURGE CHANGES

STATION	10-YEAR SURGE	100-YEAR SURGE

NO SURGE CHANGES IN THIS TRANSECT

PART5 LOCATION OF V ZONES

STATION OF GUTTER	LOCATION OF ZONE
170.31	WINDWARD

PART6 NUMBERED A ZONES AND V ZONES

STATION OF GUTTER	ELEVATION	ZONE DESIGNATION	FHF
.00	19.01		
		V11 EL=19	55
19.54	18.50		
		V11 EL=18	55
57.56	17.50		
		V11 EL=17	55
95.59	16.50		
		V11 EL=16	55
132.99	15.50		
		V11 EL=15	55
170.31	14.50		
		V11 EL=15	55
170.31	14.50		
		A10 EL=14	50
202.99	13.50		
		A10 EL=13	50
217.62	12.50		
		A10 EL=12	50
219.00	12.41		

ZONE TERMINATED AT END OF TRANSECT

1

WAVE HEIGHT COMPUTATIONS FOR FLOOD INSURANCE STUDIES (VERSION 3.0, 9_88)
G&S Example, Transect D (Seawall)

PART1 INPUT

IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.036	.000
IF	140.000	5.000	.000	12.400	.000	.000	.000	.000	.013	.000
IF	300.000	3.800	.000	12.400	.000	.000	.000	.000	.046	.000
IF	301.000	12.400	.000	12.400	.000	.000	.000	.000	8.600	.000
ET	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000

1

	END STATION	END ELEVATION	FETCH LENGTH	SURGE ELEV 10-YEAR	SURGE ELEV 100-YEAR	INITIAL WAVE HEIGHT	INITIAL W. PERIOD		BOTTOM SLOPE	AVERAGE A-ZONES
IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.036	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	140.000	5.000	.000	12.400	.000	.000	.000	.000	.013	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	300.000	3.800	.000	12.400	.000	.000	.000	.000	.046	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	301.000	12.400	.000	12.400	.000	.000	.000	.000	8.600	.000

--END OF TRANSECT--

NOTE:

SURGE ELEVATION INCLUDES CONTRIBUTIONS FROM ASTRONOMICAL AND STORM TIDES.

1

PART2: CONTROLLING WAVE HEIGHTS, SPECTRAL PEAK WAVE PERIOD, AND WAVE CREST ELEVATIONS

LOCATION		CONTROLLING WAVE HEIGHT	SPECTRAL PEAK WAVE PERIOD	WAVE CREST ELEVATION
IE	.00	9.45	12.70	19.01
	105.00	6.64	12.70	17.05
IF	140.00	5.69	12.70	16.38
	252.00	5.84	12.70	16.49
IF	300.00	5.92	12.70	16.54
IF	301.00	.01	12.70	12.41

PART3 LOCATION OF AREAS ABOVE 100-YEAR SURGE

NO AREAS ABOVE 100-YEAR SURGE IN THIS TRANSECT

PART4 LOCATION OF SURGE CHANGES

STATION	10-YEAR SURGE	100-YEAR SURGE

NO SURGE CHANGES IN THIS TRANSECT

PART5 LOCATION OF V ZONES

STATION OF GUTTER	LOCATION OF ZONE
300.49	WINDWARD

PART6 NUMBERED A ZONES AND V ZONES

STATION OF GUTTER	ELEVATION	ZONE DESIGNATION	FHF
.00	19.01		
		V11 EL=19	55
27.42	18.50		
		V11 EL=18	55
80.78	17.50		
		V11 EL=17	55
133.87	16.50		
		V11 EL=16	55
261.35	16.50		
		V11 EL=17	55
300.01	16.50		
		V11 EL=16	55
300.25	15.50		
		V11 EL=15	55
300.49	14.50		
		A 9 EL=14	45
300.74	13.50		
		A 9 EL=13	45
300.98	12.50		
		A 9 EL=12	45
301.00	12.41		

ZONE TERMINATED AT END OF TRANSECT

1

WAVE HEIGHT COMPUTATIONS FOR FLOOD INSURANCE STUDIES (VERSION 3.0, 9_88)
G&S Example, Transect E (Marsh)

PART1 INPUT

IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.030	.000
IF	165.000	5.000	.000	11.740	.000	.000	.000	.000	.011	.000
VH	500.000	5.400	2.000	1.000	.000	1.000	.000	10.400	.001	.000
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000

VH	4200.000	10.400	2.000	1.000	.000	1.000	.000	10.400	.001	.000
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000
ET	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000

1

	END STATION	END ELEVATION	FETCH LENGTH	SURGE ELEV 10-YEAR	SURGE ELEV 100-YEAR	INITIAL WAVE HEIGHT	INITIAL W. PERIOD		BOTTOM SLOPE	AVERAGE A-ZONES
IE	.000	.000	.000	8.900	12.400	28.000	12.700	.000	.030	.000

	END STATION	END ELEVATION	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR					BOTTOM SLOPE	AVERAGE A-ZONES
IF	165.000	5.000	.000	11.740	.000	.000	.000	.000	.011	.000

	END STATION	END ELEVATION	REGION 1	REGION 1 WEIGHT	REGION 2	NO. OF PLANT TYPES	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR	BOTTOM SLOPE	AVERAGE A-ZONES
VH	500.000	5.400	2.000	1.000	.000	1.000	.000	10.400	.001	.000

	PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO	
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000

PLANT CHARACTERISTICS INCLUDING VALUES SUPPLIED BY THE PROGRAM

PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO
SPAT	.100	1.000	.850	327.000	.023	.011	.011	1.380

	END STATION	END ELEVATION	REGION 1	REGION 1 WEIGHT	REGION 2	NO. OF PLANT TYPES	NEW SURGE 10-YEAR	NEW SURGE 100-YEAR	BOTTOM SLOPE	AVERAGE A-ZONES
VH	4200.000	10.400	2.000	1.000	.000	1.000	.000	10.400	.001	.000

	PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO	
MG	SPAT	.000	.000	.000	.000	.000	.000	.000	.000	.000

PLANT CHARACTERISTICS INCLUDING VALUES SUPPLIED BY THE PROGRAM

PLANT TYPE	DRAG COEFF.	COVERAGE RATIO	AVG. STEM HEIGHT	NUMBER DENSITY	BASE STEM DIAMETER	MID STEM DIAMETER	TOP STEM DIAMETER	LEAF-STEM AREA RATIO
SPAT	.100	1.000	.850	327.000	.023	.011	.011	1.380

--END OF TRANSECT--

NOTE:

SURGE ELEVATION INCLUDES CONTRIBUTIONS FROM ASTRONOMICAL AND STORM TIDES.

1

PART2: CONTROLLING WAVE HEIGHTS, SPECTRAL
PEAK WAVE PERIOD, AND WAVE CREST ELEVATIONS

LOCATION		CONTROLLING WAVE HEIGHT	SPECTRAL PEAK WAVE PERIOD	WAVE CREST ELEVATION
IE	.00	9.45	12.70	19.01
	107.25	6.69	12.70	16.65
IF	165.00	5.19	12.70	15.37
	265.00	4.58	12.70	14.55
	365.00	4.09	12.70	13.80
	465.00	3.66	12.70	13.10
VH	500.00	3.52	12.70	12.86
	630.00	3.16	12.70	12.61
	750.00	2.90	12.70	12.43
	870.00	2.69	12.70	12.28
	1030.00	2.48	12.70	12.13
	1270.00	2.25	12.70	11.97
	1430.00	2.13	12.70	11.89
	1590.00	2.04	12.70	11.83
	1910.00	1.87	12.70	11.71
	2230.00	1.72	12.70	11.60
	2870.00	1.40	12.70	11.38
	3510.00	.73	12.70	10.91
	3830.00	.39	12.70	10.67
	3990.00	.22	12.70	10.55
	4150.00	.05	12.70	10.44
VH	4200.00	.01	12.70	10.41

PART3 LOCATION OF AREAS ABOVE 100-YEAR SURGE

NO AREAS ABOVE 100-YEAR SURGE IN THIS TRANSECT

PART4 LOCATION OF SURGE CHANGES

STATION	10-YEAR SURGE	100-YEAR SURGE
165.00	8.90	11.74
500.00	8.90	10.40

PART5 LOCATION OF V ZONES

STATION OF GUTTER	LOCATION OF ZONE
703.81	WINDWARD

PART6 NUMBERED A ZONES AND V ZONES

STATION OF GUTTER	ELEVATION	ZONE DESIGNATION	FHF
.00	19.01		
		V10 EL=19	50
23.36	18.50		
		V10 EL=18	50
68.82	17.50		
		V10 EL=17	50
114.22	16.50		
		V 9 EL=16	45
159.29	15.50		
		V 9 EL=15	45
165.00	15.37		
		V 8 EL=15	40
271.55	14.50		
		V 6 EL=14	30
407.95	13.50		
		V 5 EL=13	25
500.00	12.86		
		V 5 EL=13	25
703.81	12.50		
		V 5 EL=13	25
703.81	12.50		
		A 5 EL=12	25
2525.07	11.50		
		A 5 EL=11	25
4064.40	10.50		
		A 5 EL=10	25
4200.00	10.41		

ZONE TERMINATED AT END OF TRANSECT

PART 6: WAVE ENVELOPES

Table 6-1: Summary of Flood Zones for Transect A

Station 1 (feet from 0 NGVD)	Station 2 (feet from 0 NGVD)	Elevation (feet NGVD)	Designation
0	200	17	VE
200	400	14	VE
400	700	12	VE[1]
700	3283	11	AE
3283	3500	10	AE

[1]Inland limit of primary frontal dune

Table 6-2: Summary of Flood Zones for Transect B

Station 1 (feet from 0 NGVD)	Station 2 (feet from 0 NGVD)	Elevation (feet NGVD)	Designation
0	200	16	VE
200	400	12	VE[1]
400	3263	11	AE
3263	3500	10	AE
3283	3500	10	AE

[1]Inland limit of primary frontal dune

Table 6-3: Summary of Flood Zones for Transect C

Station 1 (feet from 0 NGVD)	Station 2 (feet from 0 NGVD)	Elevation (feet NGVD)	Designation
0	219[1]	16	VE

[1]V-Zone extended to this station given map scale

Table 6-4: Summary of Flood Zones for Transect D

Station 1 (feet from 0 NGVD)	Station 2 (feet from 0 NGVD)	Elevation (feet NGVD)	Designation
0	200	17	VE
200	325	15	VE[1]
325	525	2[2]	A0

[1]Based on overtopping assesment
[2]Ponding area of depth 2 feet due to overtopping

Table 6-5: Summary of Flood Zones for Transect E

Station 1 (feet from 0 NGVD)	Station 2 (feet from 0 NGVD)	Elevation (feet NGVD)	Designation
0	200	17	VE
200	400	14	VE
400	704	13	VE[1]
704	2525	12	AE
2525	4200	11	AE

[1]WHAFIS VE zone termination

Figure 6-1: Wave Envelope, Transect A

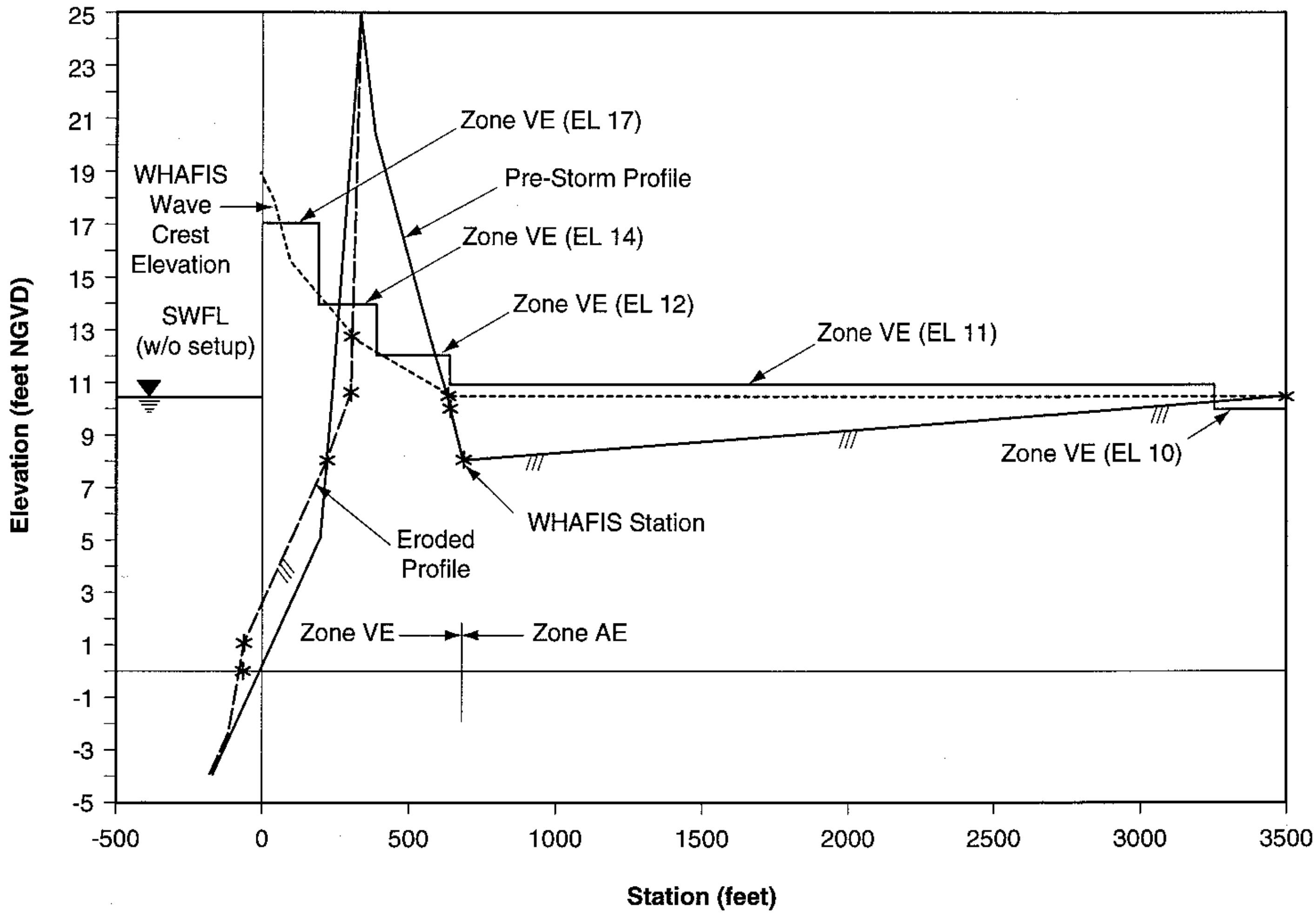


Figure 6-1. Transect A, Dune Retreat

Figure 6-2: Wave Envelope, Transect B

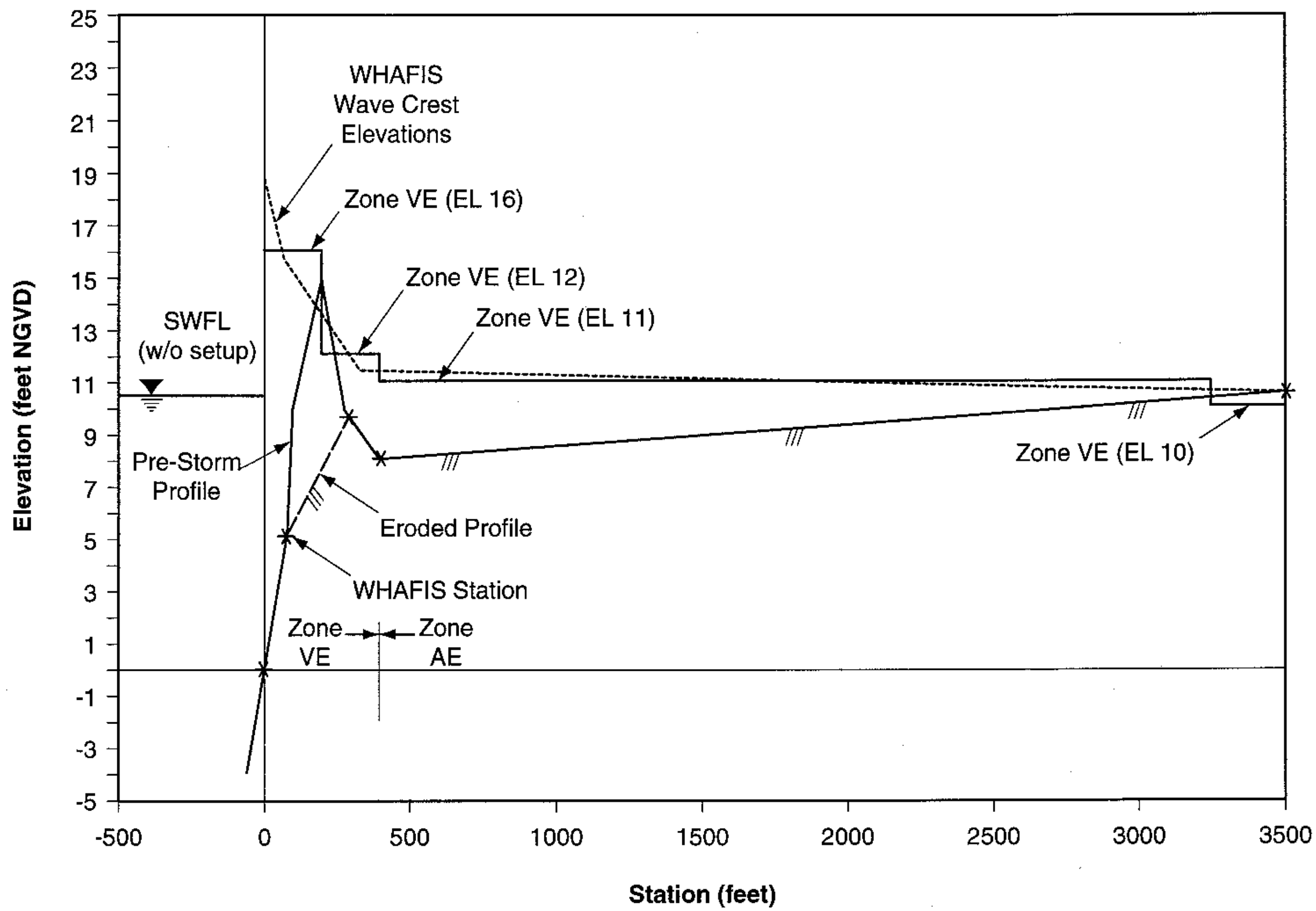


Figure 6-2. Transect B, Dune Removal

Figure 6-3: Wave Envelope, Transect C

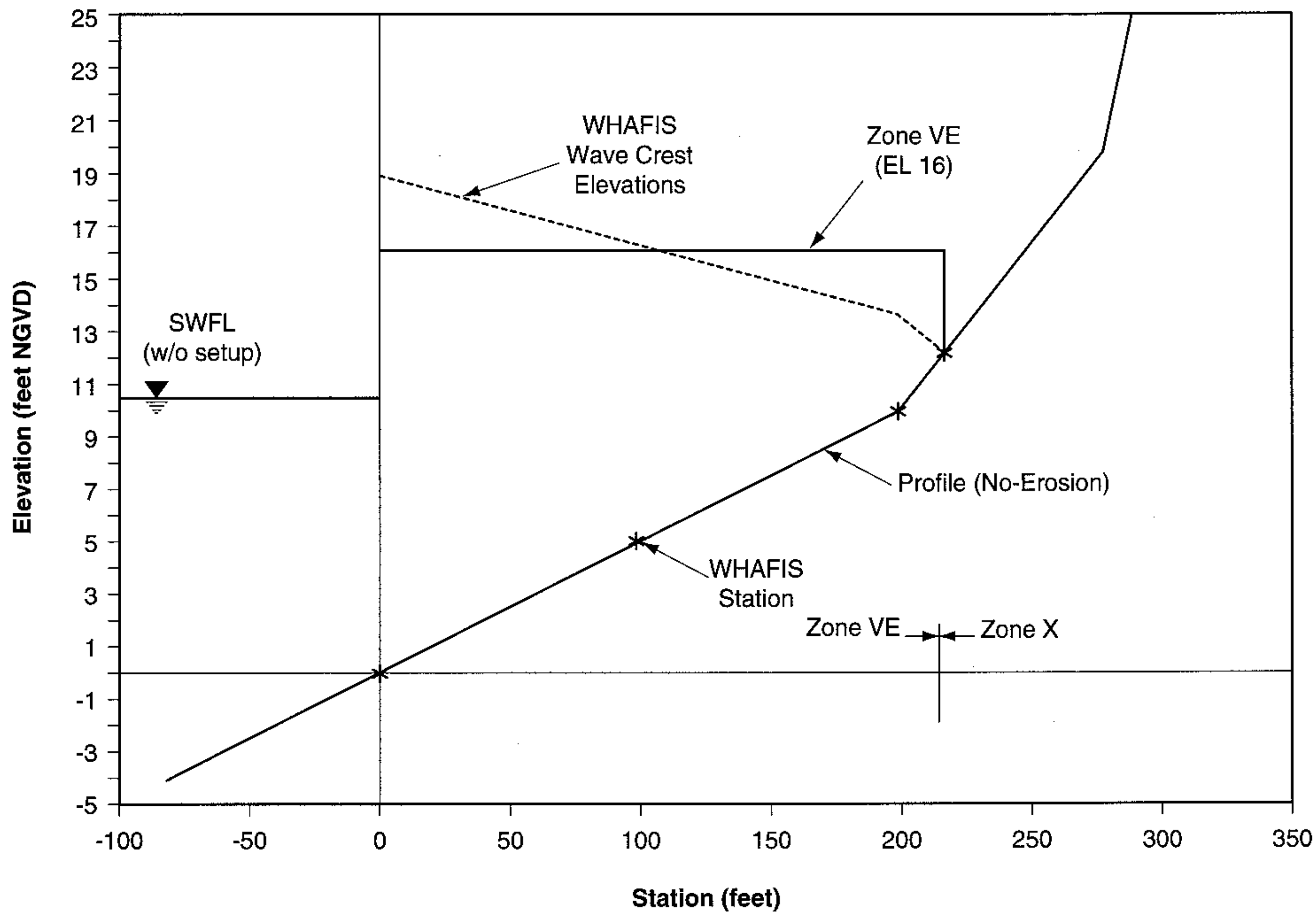


Figure 6-3. Transect C, Buff Reventment

Figure 6-4: Wave Envelope, Transect D

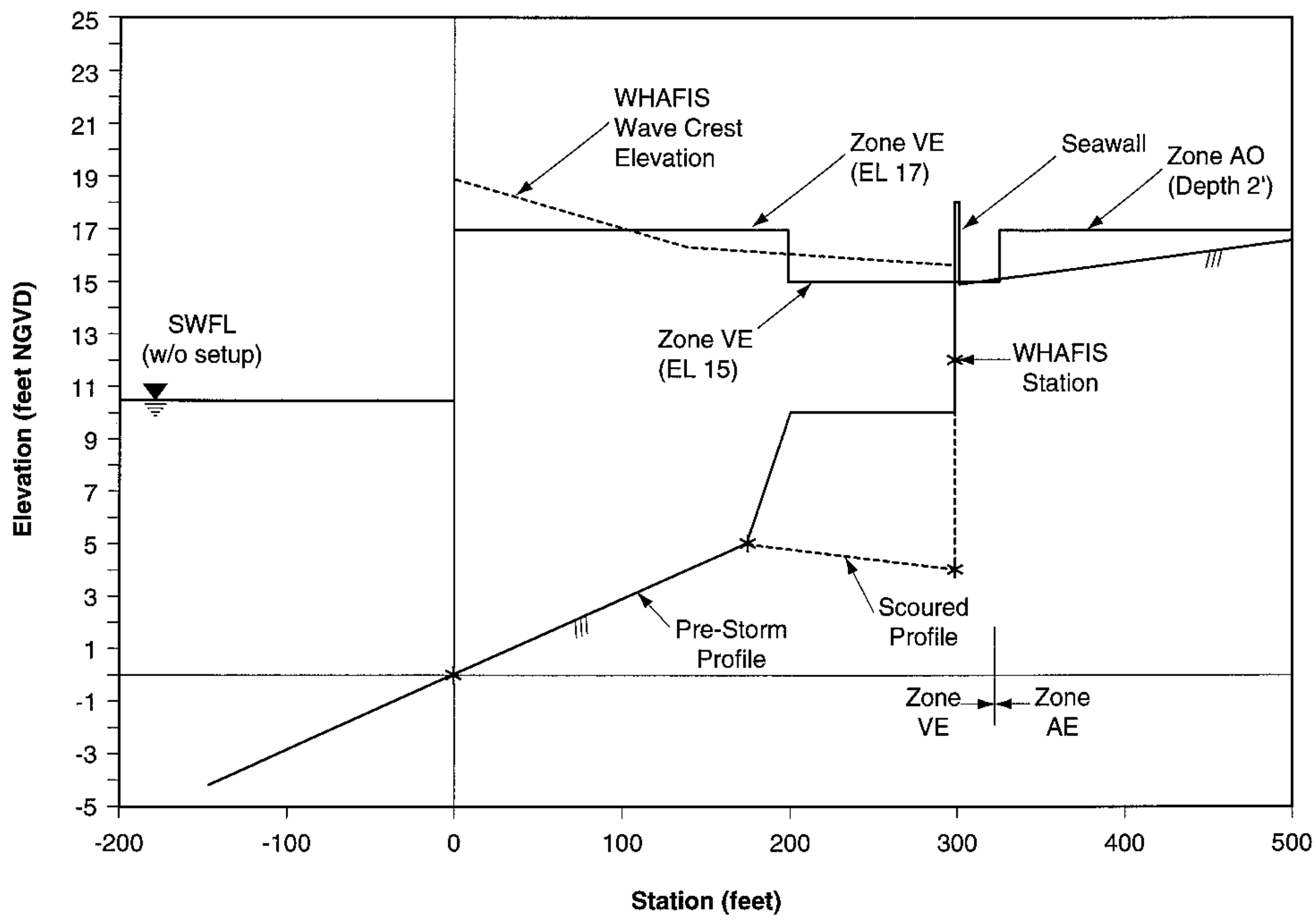


Figure 6-4. Transect D, Seawall

Figure 6-5: Wave Envelope, Transect E

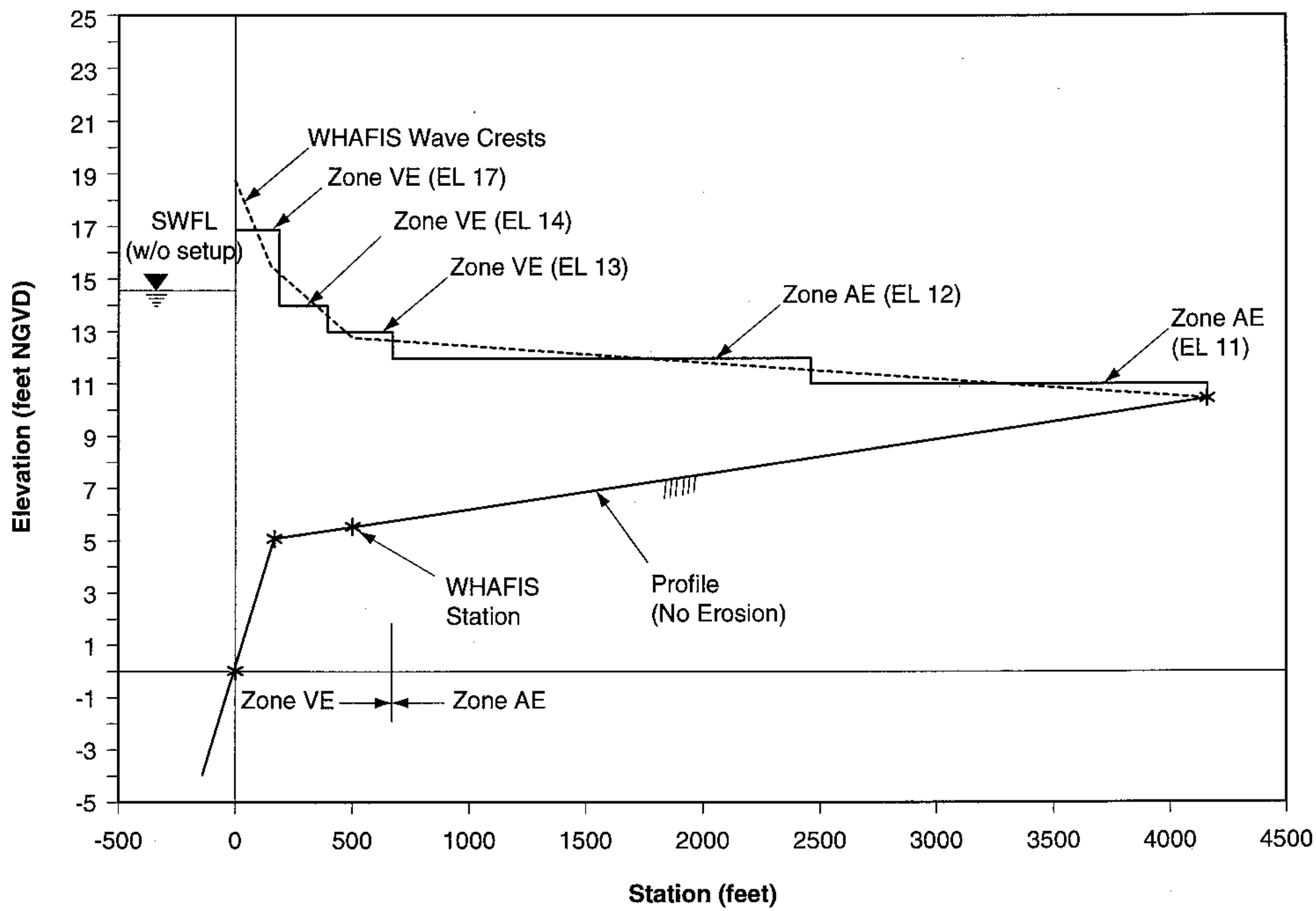


Figure 6-5. Transect E, Marsh

Table 6-6: Pre-Storm Profile, Transect A

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-150	-4.0
0	0.0
185	5.0
250	10.0
300	20.0
350	25.0
400	20.0
650	10.0
700	8.0
3500	10.4

Notes: SWFL should be dropped to 10.4 feet NGVD after dune crest. Marsh grass from approximately station 200 to station 3500. Average tidal flat width is approximately 333 feet. Assuming seaward edge of tidal flat is -4 feet NGVD, and landward edge is +5 feet NGVD, 0 NGVD was interpolated and station zero was set to this point.

Table 6-7: Eroded Profile, Transect A

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-150	-4.0
-99	-2.6
-53	1.0
223	7.9
322	10.4
335	23.5
350	25.0
400	20.0
650	10.0
700	8.0
3500	10.4

Table 6-8: WHAFIS Elevations, Transect A

Station[1] (ft from 0 NGVD)	Elevation (ft NGVD)
-66	0.0
-53	1.0
223	7.9
322	10.4
324	12.4
640	10.4
650	10.0
700	8.0
3500	10.4

[1]Stationing in this table is from 0 NGVD on prestrom profile; however, stationing in WHAFIS model was adjusted to correspond to 0 NGVD on the eroded profile

Table 6-9: WHAFIS Wave Envelope, Transect A

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-66	19.0
-53	18.5
23	17.5
98	16.5
173	15.5
248	14.5
321	13.5
324	12.5
324	12.4
640	10.4
672	10.5
3283	10.5
3500	10.4

Table 6-10: Pre-Storm Profile, Transect B

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-60	-4.0
0	0.0
75	5.0
100	10.0
200	15.0
275	10.0
400	8.0
3500	10.4

Notes: SWFL dropped to 10.4 feet NGVD after crest of dune. Marsh grass from station 400 to station 3500. Average tidal flat width is approximately 132 feet. Assuming seaward edge of tidal flat is -4 feet NGVD, and landward edge is +5 feet NGVD, 0 NGVD was interpolated and station zero was set to this point. The first and only row of houses is assumed to be removed due to erosion.

Table 6-11: Eroded Profile, Transect B

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-60	-4.0
-0	0.0
75	5.0
303	9.6
400	8.0
3500	10.4

Table 6-12: WHAFIS Elevations, Transect B

Station (ft from 0 NGVD)	Elevation (ft NGVD)
0	0.0
75	5.0
303	9.6
400	8.0
3500	10.4

Table 6-13: WHAFIS Wave Envelope, Transect B

Station (ft from 0 NGVD)	Elevation (ft NGVD)
0	19.0
12	18.5
35	17.5
59	16.5
75	15.8
92	15.5
146	14.5
200	13.5
200	13.4
254	12.5
303	11.6
329	11.5
400	11.3
3263	10.5
3500	16.4

Table 6-14: Pre-Storm Profile, Transect C

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-80	-4.0
0	0.0
100	5.0
200	10.0
280	20.0
300	30.0
350	50.0

Notes: Randomly placed quarry stone from approximately station 120 to station 220. Grass from station 220 inland. Average tidal flat width is approximately 180 feet. Assuming seaward edge of tidal flat is -4 feet NGVD, and landward edge is +5 feet NGVD, 0 NGVD was interpolated and station zero was set to this point. Revetment found to be stable during 100-year event.

Table 6-15: WHAFIS Elevations, Transect C

Station (ft from 0 NGVD)	Elevation (ft NGVD)
0	0.0
100	5.0
200	10.0
219	12.4

Table 6-16: WHAFIS Wave Envelope, Transect C

Station (ft from 0 NGVD)	Elevation (ft NGVD)
0	19.0
20	18.5
58	17.5
96	16.5
133	15.5
170	14.5
203	13.5
218	12.5
219	12.4

Table 6-17: Pre-Storm Profile, Transect D

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-110	-4.0
0	0.0
140	5.0
200	10.0
300	10.0
300	18.2
302	18.2
302	15.0
950	20.0

Notes: Wall cap surveyed from filed at 18.2 NGVD. Average tidal flat width is approximately 250 feet. Assuming seaward edge of tidal flat is -4 feet NGVD, and landward edge is +5 feet NGVD, 0 NGVD was interpolated and station zero was set to this point. Toe scour was approximated. Wall found to be stable during 100-year event.

Table 6-18: Eroded Profile, Transect D

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-110	-4.0
-0	0.0
140	5.0
300	3.8
300	18.2
302	18.2
302	15.0
950	20.0

Table 6-19: WHAFIS Elevations, Transect D

Station (ft from 0 NGVD)	Elevation (ft NGVD)
0	0.0
140	5.0
300	3.8
301	12.4

Table 6-20: WHAFIS Wave Envelope, Transect D

Station (ft from 0 NGVD)	Elevation (ft NGVD)
0	19.0
27	18.5
81	17.5
134	16.5
300	15.5
301	12.4

Table 6-21: Pre-Storm Profile, Transect E

Station (ft from 0 NGVD)	Elevation (ft NGVD)
-135	-4.0
0	0.0
165	5.0
500	5.4
4200	10.4

Notes: SWFL should be dropped to 10.4 feet NGVD at approximately station 500. Marsh grass from approximately station 165 to station 4200. Average tidal flat width is approximately 300 feet. Assuming seaward edge of tidal flat is -4 feet NGVD, and landward edge is +5 feet NGVD, 0 NGVD was interpolated and station zero was set to this point. No erosion performed.

Table 6-22: WHAFIS Elevations, Transect E

Station (ft from 0 NGVD)	Elevation (ft NGVD)
0	0.0
165	5.0
500	5.4
4200	10.4

Table 6-23: WHAFIS Wave Envelope, Transect E

Station (ft from 0 NGVD)	Elevation (ft NGVD)
0	19.0
23	18.5
69	17.5
114	16.5
159	15.5
165	14.5
272	13.5
408	12.5
500	12.4
704	10.4
2525	10.5
4064	10.5
4200	10.4

PART 7: MAPPING

Figure 7-1: Transect Locations

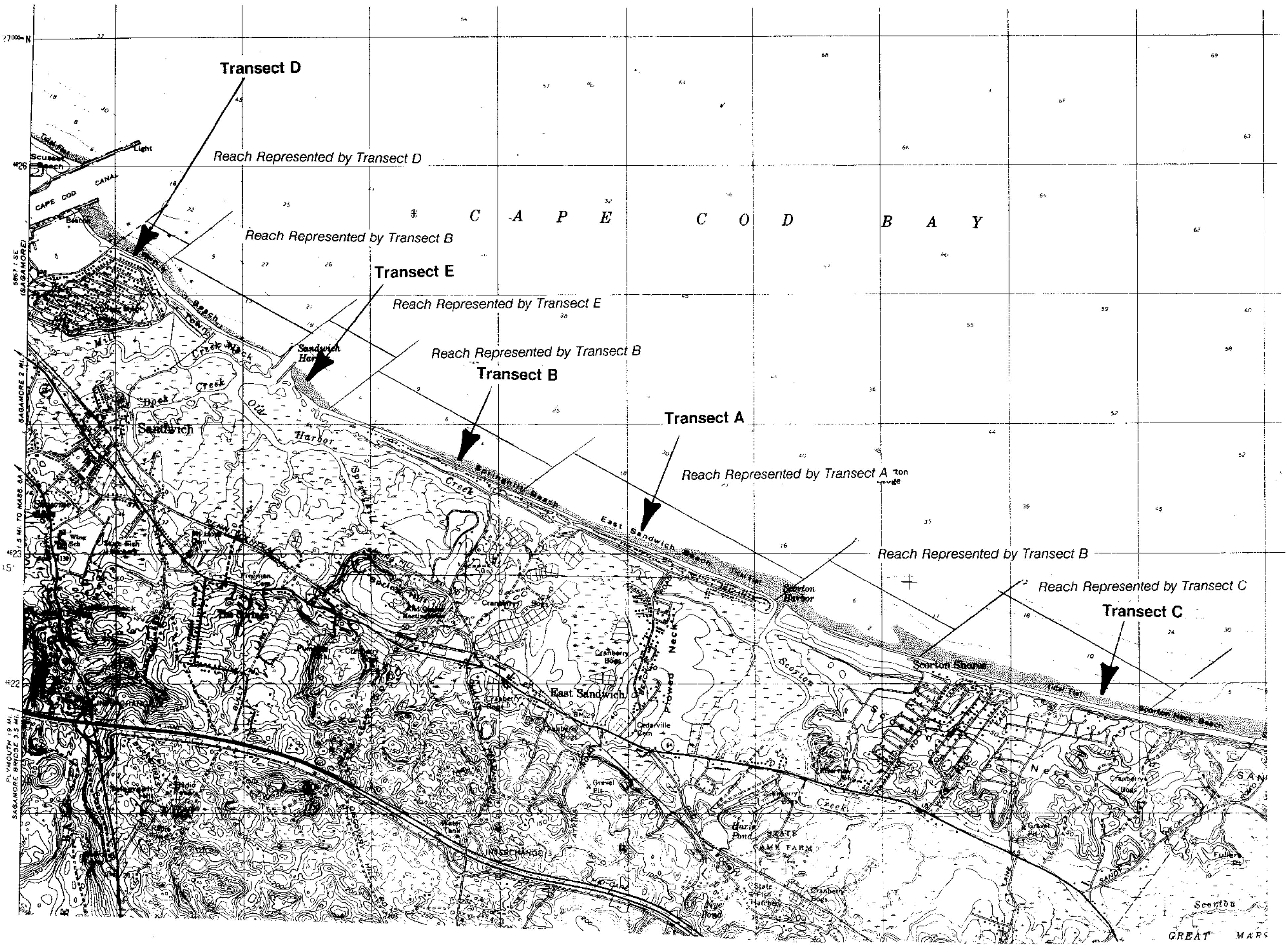

Transect D
Reach Represented by Transect D
Reach Represented by Transect B
Transect E
Reach Represented by Transect E
Reach Represented by Transect B
Transect B
Transect A
Reach Represented by Transect A
Reach Represented by Transect B
Reach Represented by Transect C
Transect C
C A P E C O D B A Y
CAPE COD CANAL
Sandwich
Sandwich Harbor
East Sandwich Beach
East Sandwich
Plowed Neck
Scorton Harbor
Scorton Shores
Scorton Neck Beach
Tidal Flat
SAGAMORE 2 MI.
PLYMOUTH 19 MI.
SAGAMORE BRIDGE 3.5 MI.
GREAT MARS

Figure 7-2: Flooding Delineation

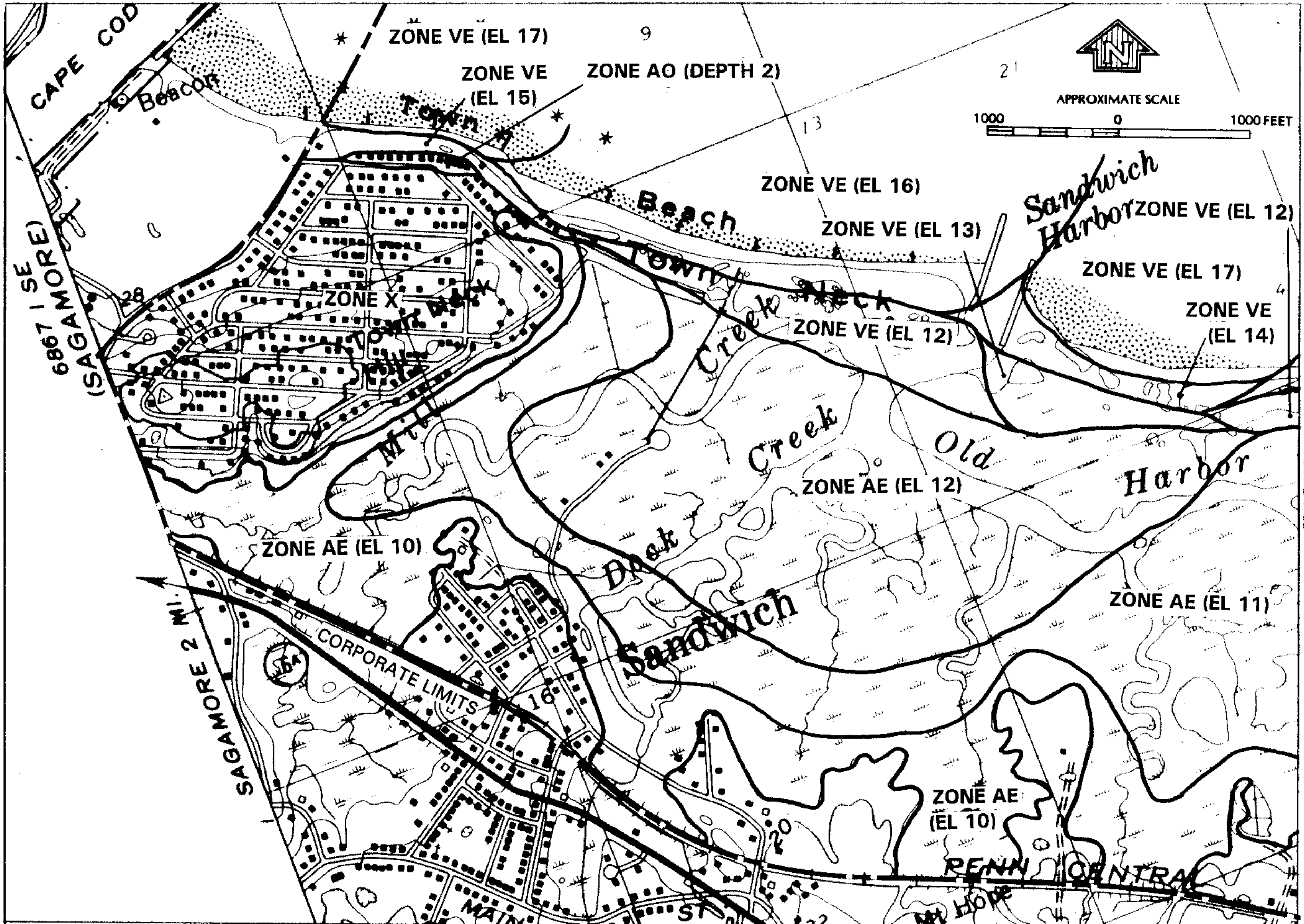
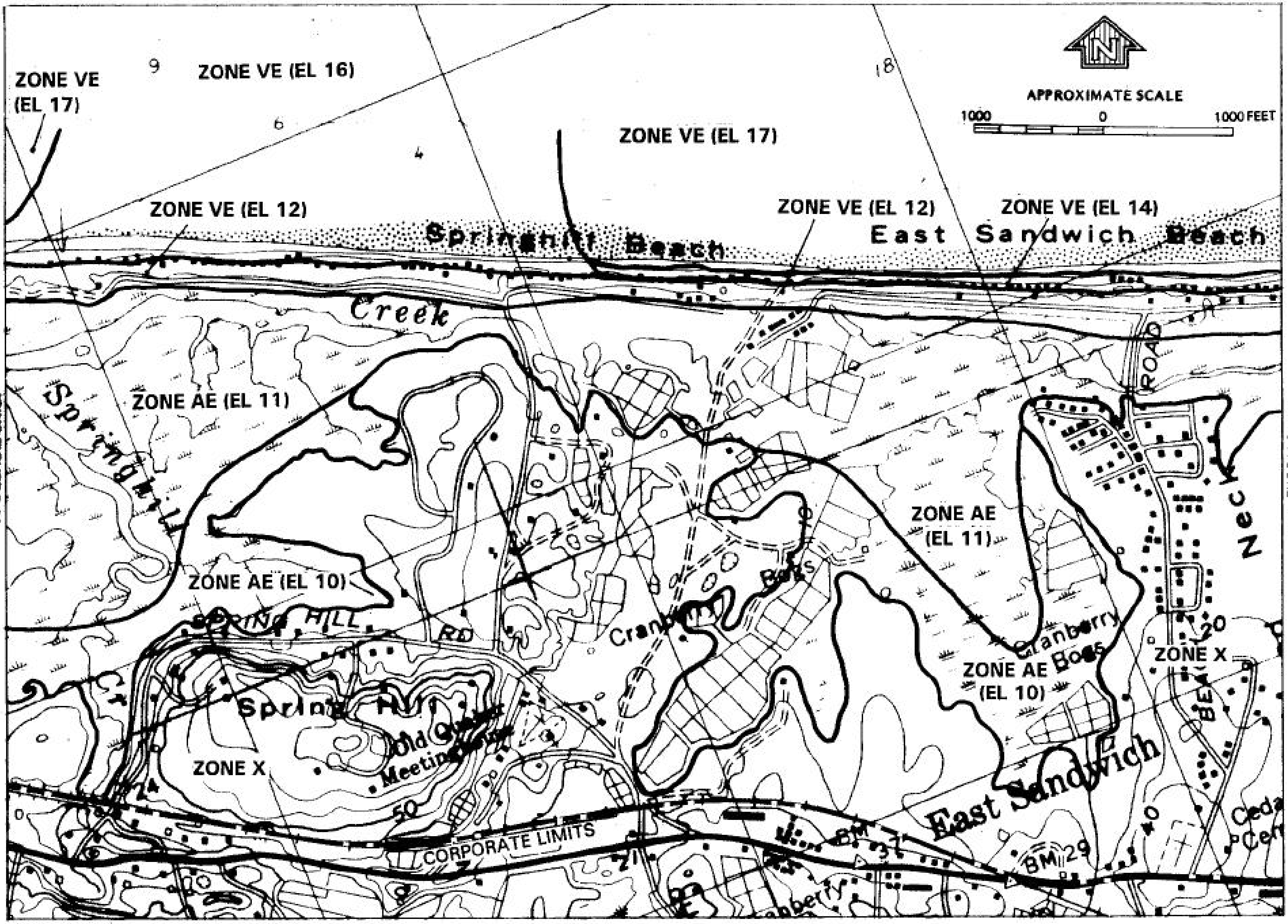


FIGURE 7-2A

JOINS FIGURE 7-2B

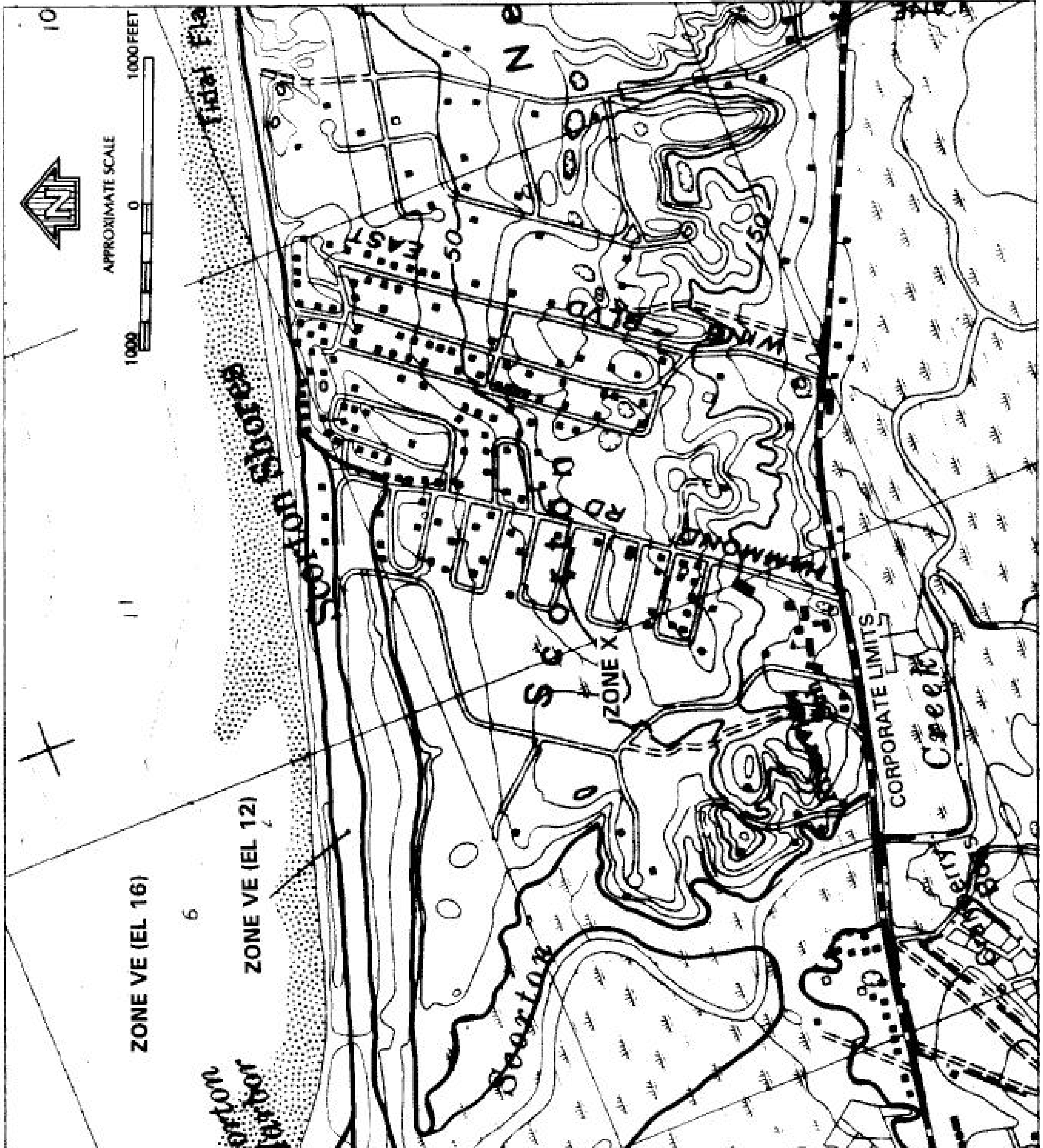
JOINS FIGURE 7-2A



JOINS FIGURE 7-2C

FIGURE 7-2B

JOINS FIGURE 7-2D



JOINS FIGURE 7-2B

FIGURE 7-2C

JBINS FIGURE 7-2C

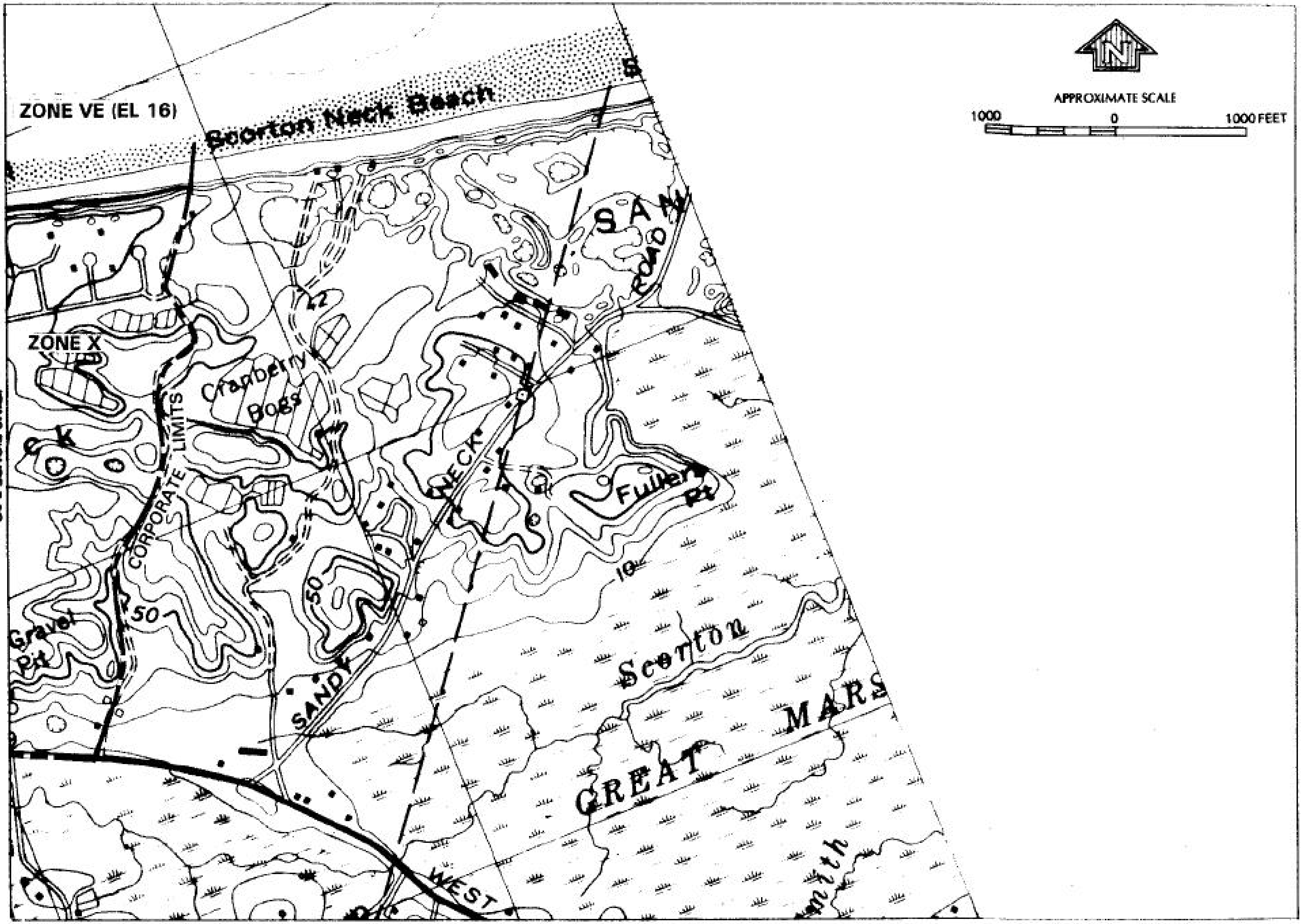


FIGURE 7-2D

APPENDIX B

EVALUATING COASTAL

FLOOD PROTECTION STRUCTURES

(From Reference 28)

Criteria for Evaluating Coastal Flood Protection Structures

Background

Many property owners and communities along the U.S. coast are resorting to the construction of coastal flood control structures to protect existing or new development from potential damage associated with hurricanes and other major coastal storm events. Flooding and erosion caused by natural processes, sea level rise, and/or man-made influences are factors contributing to the decision to construct structures such as seawalls, revetments, bulkheads, and coastal levees/dikes. Although there is continued debate on the overall impact of these coastal structures, their construction and use requires that FEMA evaluate their effectiveness for reducing flood risk and their viability as an alternative to the non-structural flood loss reduction approaches required for community participation in the National Flood Insurance Program (NFIP).

The areas protected by coastal flood protection structures are frequently designated as Coastal High Hazard Areas (V zones) on the Flood Insurance Rate Maps (FIRMs) published by FEMA. FEMA is often requested to revise FIRMs to reflect the protection provided by a coastal structure against the base (100-year) flood. Because of the different types of coastal structures, materials, and construction methods, FEMA must perform a detailed review of these requests to assure that the structure is adequately designed and constructed to provide the stated level of protection, and to withstand the 100-year flooding event.

Part 65 of the NFIP regulations requires that any requestor of a FIRM revision based on flood protection structures provide an analysis of the revised flood hazards, demonstrate and certify that the structure is designed and constructed for 100-year flooding conditions, and provide assurance that the structure will be maintained. Revision requests based on coastal structures are currently reviewed on a case-by-case basis using these regulations. A wide variation has been found in the quality of data submitted. Some possible reasons for this variation include the requestor's inexperience or unfamiliarity with the different types of structures, the available design guidance, and/or the base (100-year) flood considered by the NFIP. In order to improve the quality of information submitted, and the ability of FEMA to review revision requests based on coastal structures, FEMA has decided to establish minimum design criteria that must be addressed in the request.

FEMA commissioned the U.S. Army Corps of Engineers, Waterways Experiment Station (WES), Coastal Engineering Research Center to identify or develop criteria for evaluating the effectiveness of all types of coastal flood protection structures in preventing or reducing damages and flooding from the 100-year event. This study identified and defined the different coastal structures that provide protection against flooding to property landward of the structure, and documented successful and unsuccessful cases for each structure type. The minimum criteria, considerations, and/or conditions applicable to the 100-year flooding event that are necessary for an evaluation of a coastal structure were also identified. The WES study recommended a procedure using these criteria to evaluate the adequacy, of a coastal flood protection structure to survive the 100-year flooding event, and to provide protection against flooding, wave runup and overtopping, wave forces, and erosion.

The WES Technical Report CERC-89-15 "Criteria for Evaluating Coastal Flood

Protection Structures" was used as the basis for these criteria. These criteria will also be used to resolve appeal challenges and in the conduct of flood insurance studies, when sufficient design and construction data are available.

<u>Mapping of areas protected by coastal flood protection structures</u>.

(a) 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, this 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 Part 65 of this subchapter 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.

(b) 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 (100-year) flooding event, including the following:

(i) Design water levels evaluated should range from the mean low water level at the site to the 100-year stillwater surge elevation. The full range of elevations must be examined to determine the critical water level since 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 height and periods should be used for "flexible" structures such as revetments, with larger wave height, up to the one-percent 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) <u>Shore Protection Manual</u> (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 COE 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 two feet above the 100-year stillwater surge elevation.

(3) Toe Protection. The loss of material and profile lowering seaward of the structure 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 COE'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 the toe protection, or an analysis of scour potential such as found in "Criteria for Evaluation 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 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 material by rundown over the structure, by drainage landward, under, and laterally around the ends of the structure; as well as through joints, seams, or drainage openings in the structure.

(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 structure, and maximum toe scour must be submitted.

(i) For coastal dikes and revetments, a 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, of seaward overturning, and of 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 100-year 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 blocks' 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, of landward overturning, and of foundation adequacy using the maximum pressures developed in the sliding and overturning calculations must be submitted.

(iv) For anchored bulkheads, engineering analyses of shear 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, regardless of type described above, 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 which 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.

(c) Adverse Impact Evaluation. 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.

(d) Community and/or State Review. 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.

(e) Maintenance Plans and Criteria. 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, and 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.

(f) Certification Requirements. Data and analyses submitted to support that a given coastal flood protection structure complies with the structural design requirements set forth in paragraphs (b)(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.