FEMA Great Lakes Coastal Guidelines, Appendix D.3 Update

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D.3 Coastal Flooding Analyses and Mapping: Great Lakes

This section of Appendix D provides guidance for coastal flood hazard analyses and mapping specific to the Great Lakes Shorelines of the United States, generally referred to as "guidelines." They are intended to provide guidance that is generally independent of other Appendix D sections, and that is based on the specific physical processes that influence coastal flooding along the shorelines of the Great Lakes.

This section focuses on the coastline of the Great Lakes, as shown in Figure D.3-1. The Atlantic and Gulf coastlines and the Pacific coastline are specifically addressed in Sections D.2 and D.4, respectively.

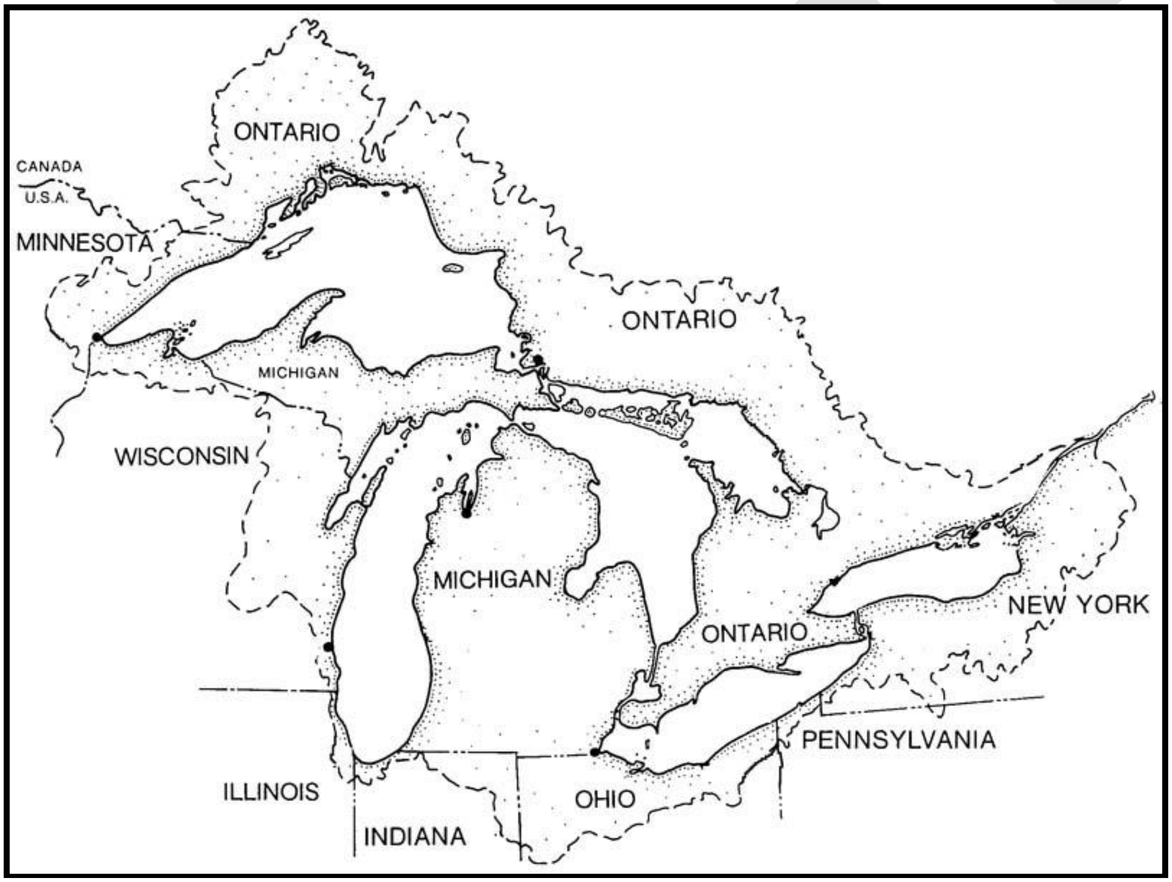


Figure D.3-1. Appendix D.3 Applicable Area – Great Lakes Guidelines

D.3.1 Great Lakes Guidelines Overview

Section D.3 is organized to:

- Present background information and discuss the contributors to coastal flooding in the Great Lakes (Section D.3.1);
- Provide guidance on selecting study methodologies for storm sampling, statistical analysis, and analyses of coastal processes (Sections D.3.2 to D.3.3);
- Provide guidance on selecting methods to analyze the different coastal processes that influence the flood hazard (Sections D.3.4 to D.3.8)
- Provide guidance on flood hazard mapping (Section D.3.9);
- Provide guidance on study documentation (Section D.3.10); and
- Provide documentation of references, notations, and acronyms (Sections D.3.11 to D.3.13)

Figure D.3.1-1 shows the general layout of the document. Section D.3.1, provides an overview of the guidelines and discusses important contributors to the coastal flood hazard in the Great Lakes. Section D.3.2 provides a framework for analyzing coastal processes that are relevant to the Great Lakes flood hazard that Mapping Partners can use; and it refers to more detailed analysis methods in subsequent sections. Section D.3.3 discusses the storm selection and statistical analysis methodology and important considerations in its implementation. In some cases, multiple methods are presented for analysis of a single coastal process. Often, coastal processes are such that the analysis begins offshore and proceeds onshore to produce hazard zone designations for a coastal Flood Map Project. Sections D.3.2 and D.3.3 provide guidance on selecting analysis methods that are applicable to particular coastal settings and on linking the analysis of individual coastal processes together in a study methodology. In this sense, the document is organized with a set of general implementation instructions given in Sections D.3.2 and D.3.3, and a selection of specific coastal process prediction methods in Sections D.3.4 to D.3.8. The appropriate tools must be selected based on study objectives, coastal exposure, geomorphic setting, and available data. Section D.3.9 documents flood hazard mapping procedures, while D.3.10 addresses study documentation requirements. Section D.3.11 provides a detailed list of references, while D.3.12 and D.3.13 document the notation and acronyms that are used in this document.

Coastal flooding on the Great Lakes is a product of combined offshore, nearshore, and shoreline processes. The interrelationships of these processes are complex, and their relative effects vary significantly from one setting to another. These complexities present challenges in the determination of the Base (1-percent annual chance of occurrence or being exceeded) Flood Elevation (BFE) for Federal Emergency Management Agency (FEMA) hazard mapping purposes. The fundamental philosophy of this appendix is to provide sound and defensible technical approaches for characterizing the coastal inundation and wave hazards; and to provide a set of validated tools and methods for implementing the approach, which can be selected and applied as needed in light of specific site conditions and physical processes relevant to the local flood hazard.

These guidelines offer insight and recommended methods, and they will be most effective when employed along with sound technical judgment and experience. This document does not constitute a completely prescriptive technique that can be applied uniformly in all study areas. A proactive application of best engineering practices is always preferable to the rote application of the analysis options discussed in this document. While these guidelines are applicable to a wide range of settings, they do not necessarily address all settings and conditions. The Mapping Partner may determine that minor modifications or deviations from these guidelines are necessary to adequately define the coastal flooding conditions and to map flood insurance risk zones in specific areas. In these cases, documentation of differences is required as part of intermediate and final study submittals. Deviations from guidance herein must be documented and approved by the FEMA Study Representative.

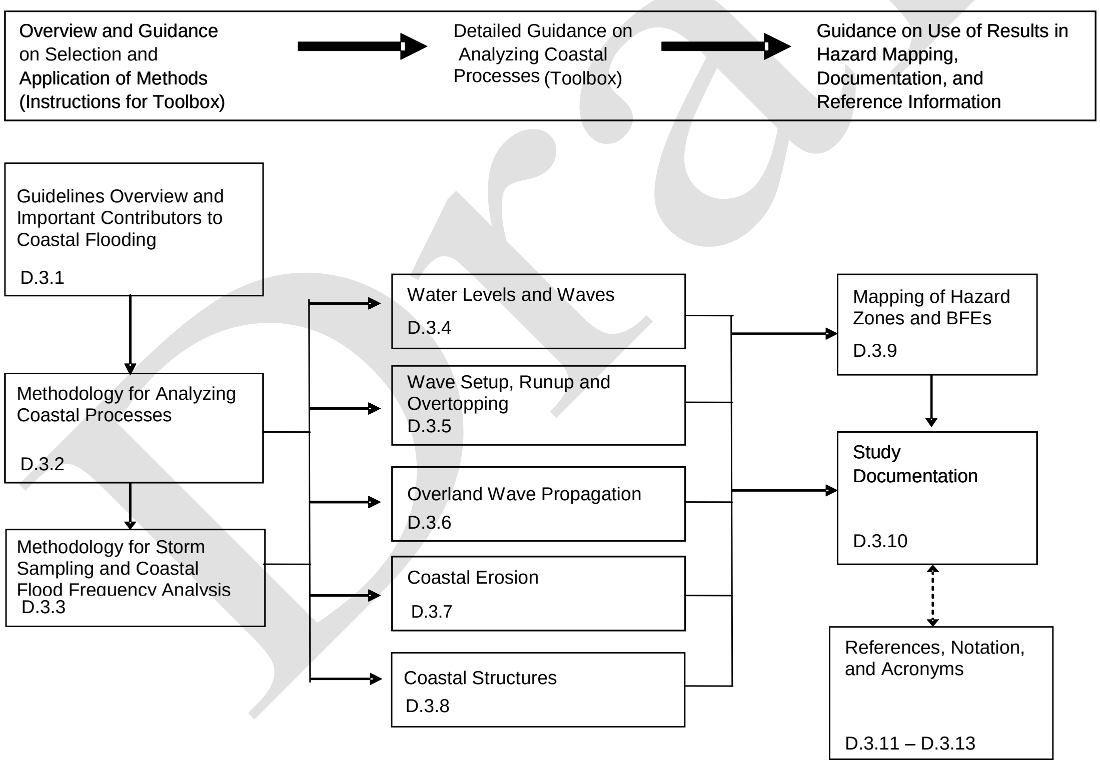


Figure D.3.1-1. Great Lakes Coastal Guidelines Overview

Other appendices provide specific information on subjects such as project scoping (Appendix I), aerial mapping and surveying (Appendix A), treatment of levee systems (Appendix H), formats for Flood Insurance Study (FIS) reports and Flood Insurance Rate Maps (FIRMs) (Appendices J and K), formats for draft digital data and Digital Flood Insurance Rate Map (DFIRM) databases

(Appendix L), and data capture standards and guidelines (Appendix M). The guidance provided here supplements these sections with information specific to Great Lakes coastal flooding. The Mapping Partner shall refer to other appendices where specific guidance is required on technical elements common to most FEMA Flood Map Projects.

In the remainder of this section, Section D.3.1.1 provides an overview of contributors to coastal flooding in the Great Lakes, and Section D.3.1.2 provides an introduction to FEMA Flood Mapping Projects for the Great Lakes coastline.

D.3.1.1 Contributors to Coastal Flooding

The Great Lakes have a mainland shoreline of 3,678 statute miles that fall within the United States, with more than 1,000 additional miles when island shorelines are taken into account. The Great Lakes contain 18 percent of the total freshwater in the world and about 90 percent of the total freshwater in the United States.

Coastal flooding in the Great Lakes can arise due to elevated still water level and/or storm waves, with energetic storm waves occurring concurrently with elevated water levels being of particular concern. In comparison to the Pacific and Atlantic coasts, the Great Lakes are unique in that they are not subject to astronomical tides of any significance; however, they are subject to changes in water level due to a number of other processes, which act over three distinctly different time scales. One of these processes is long-term lake level change. The added complexity of a fluctuating lake level is analogous to that associated with a varying mean sea level on the open ocean coasts. The magnitude of historic lake level changes renders this a very important consideration. A severe storm occurring during a low lake level might cause no flooding, but at high lake level the same storm could cause devastating flooding. The other two drivers of water-level change are seasonal-scale changes and storm event-scale changes.

Long-term lake level changes take place gradually, primarily in response to fluctuations in precipitation and evaporation. Lower precipitation leads to lower runoff from the watershed; similarly, higher evaporation draws water from the lakes, causing levels to decline. Long-term lake level fluctuations occur over decadal time scales in response to regional and continental-scale forcing, including the El Niño/La Niña cycles and their effect on rainfall.

Lake levels also change on a seasonal basis; they are lowest during the winter, when a majority of the precipitation in the region is frozen as ice and snow, and evaporation increases as dry winter air passes over the lakes. Levels increase during the spring and early summer as a result of the spring runoff of melting snow and ice, and high monthly rainfall. Water control operations also influence lake level variability, with the locks at Sault Ste. Marie influencing Lake Superior's discharge and the dam on the St. Lawrence River near Massena influencing Lake Ontario's levels.

Concurrent with these longer time-scale changes, storm events can cause significant short-term increases in water level. Atmospheric pressure gradients and persistent wind can result in water piling up along the coast. This effect is called storm surge and can last for the duration of the event, which could be a day or more. The same winds that cause a storm surge also can create large waves that impact the shoreline, increasing the chance for flooding.

Fluctuating water levels from various sources, each having different time scales, make the assessment of flood hazard risk on the Great Lakes a challenge. Accurate assessment requires an in-depth understanding of the various contributors to BFEs, the relative magnitude of each one, and how the absolute and relative magnitudes of the various BFE contributors can vary within a lake.

D.3.1.1.1 Long-Term Lake Level Changes

Long-term lake level changes in the Great Lakes are a result of both the natural processes mentioned above and anthropogenic activities. The long-term lake level variability is assumed to be a stationary process over the past 50 years. Further, the findings of Baedke and Thompson (2000) indicate the level of Lake Michigan has been stable for over 3,000 years. This is important in the consideration of water-level probabilities and must be evaluated for each lake.

Adjustment values to account for changes in lake conditions and water-control operations over time due to anthropogenic activities such as channel deepening, water diversion, or water management regulations are applied to mean monthly lake levels derived from National Oceanic and Atmospheric Administration (NOAA) water-level measurements in order to estimate lake levels that would have existed historically had the lakes been operated under current regulations and physical conditions. These modifications, called the Basis of Comparison (BOC) adjustments, were developed as a product from the International Joint Commission (IJC) Levels Reference Study in 1993 and then more recently in 2003. The modified mean monthly lake levels are adopted in these guidelines to characterize the current state of both the expected range and variability in long-term and seasonal-scale lake level changes.

For Lake Michigan, the range in long-term lake level changes during the period of 1960 to 2010 was approximately 6 feet. For Lake Erie, the range is a bit smaller, approaching 5 feet (Figure D.3.1-2). Table D.3.1-1 shows the variation in mean long-term lake levels and the variance in those levels for each of the Great Lakes. One notable difference is the larger variance in long-term lake level for Lakes Michigan and Huron (which are coupled), versus the lower variance for Lakes Superior and Ontario (which are both regulated lakes).

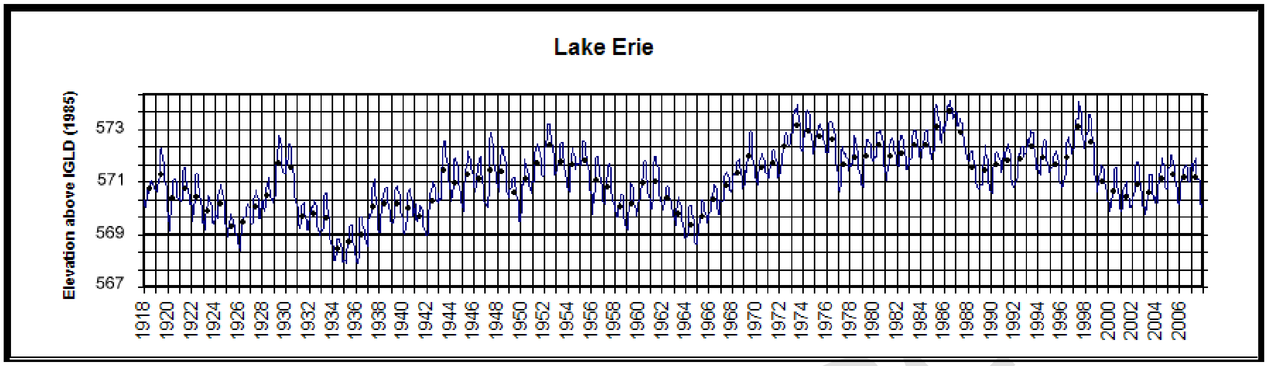


Figure D.3.1-2. Typical Long-Term Water-level Variations

Table D.3.1-1. Statistical Parameters for the Long-Term Lake Levels

	Lake Superior	Lake Michigan	Lake Huron	Lake Ontario	Lake Erie
Mean (ft, IGLD 1985)	601.59	578.73	578.73	245.19	571.17
Variance, σ^2	0.22	1.36	1.36	0.24	0.92

D.3.1.1.2 Seasonal Lake Level Changes

Lake levels also vary seasonally as a result of precipitation, evaporation, and runoff variability, along with anthropogenic activities. Figure D.3.1-3 shows all years of monthly average lake level plotted individually for the Ludington, MI gage for the period 1970 to 2009. In this plot, monthly mean values have been “de-meaned” by subtracting the mean for that year. The annual variation in monthly means for each year is plotted as a family of blue lines. The mean of all years is the red line.

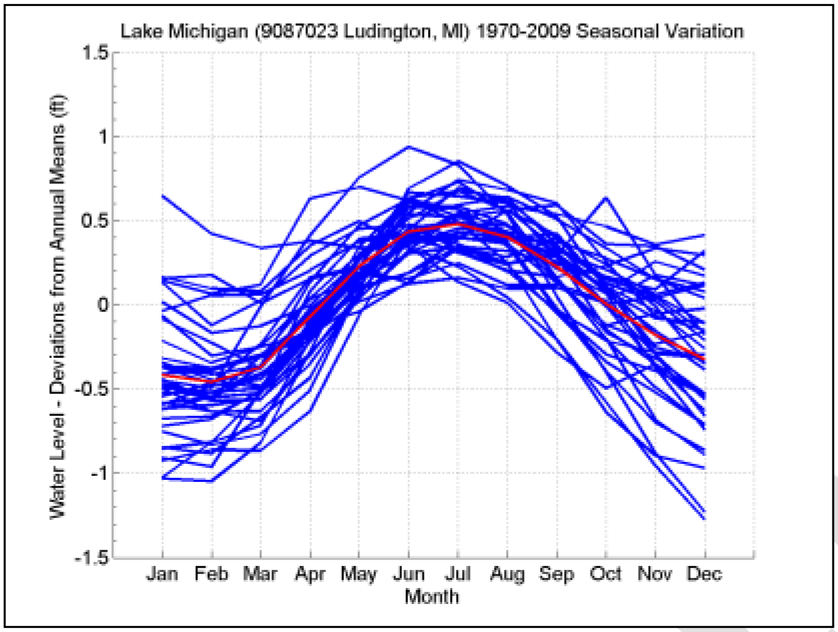


Figure D.3.1-3. Ludington, MI seasonal variability of measured monthly mean water levels 1970 – 2009.

From this figure, the seasonal cycle is clear: a minimum in January and February, and a maximum in June and July. The range in historical monthly mean water levels from 1970 to 2009 varies from a maximum of 1.7 feet in December-January to a minimum of 0.7 foot in July. As is the case for long-term lake levels, the range of seasonal water-level changes is lake-specific and must be examined for each lake. Results for Lake Michigan are shown here only to illustrate this source of variability. For Lakes Michigan and Huron and Lake Superior, the range in seasonal lake levels is roughly 1 foot, whereas seasonal changes for Lakes Ontario, Erie and St. Clair are higher, approaching 2 feet.

D.3.1.1.3 Storm Surge

In the Great Lakes, significant changes in water level can occur on time scales of hours and days. Generally, these water-level fluctuations are caused by one of several types of strong storms: 1) non-convective storms that originate in Canada and move to the east through the lakes region, 2) non-convective storms that originate in the southern and central Rockies and move east through the lakes region, 3) extra-tropical systems that move north from the Gulf Region, and 4) convective storms or thunderstorm frontal passages. Most of the strong winter storms are low-pressure non-convective systems (Lacke et al. 2006, Niziol and Paone 1991). The movement of high-pressure systems through the region often precedes or follows the occurrence of a low-pressure system. Low-pressure systems spin counter-clockwise, while high-pressure systems spin the opposite way. So winds on the eastern side or leading edge of a low-pressure system are typically in the northerly direction, while winds on the eastern side of a high-pressure system are in the southerly direction. High winds and large atmospheric pressure variations are commonly associated with these storm events, and they can cause elevated water levels, or storm surge, along the lake shoreline.

In the development of these guidelines, a decision was made to neglect the effects of convective storms (local fast-moving fronts, squall lines, and thunderstorms). This issue was examined by Melby et al. (2012), and a number of analyses were performed to support this decision. Those analyses indicated that, in general, neglecting convective events had minimal influence on extremal water-level statistics. The decision also was made, in large part, because of insufficient spatial and temporal resolution of wind and pressure data with which to resolve these types of isolated weather systems in storm surge and wave modeling.

Several physical processes contribute to generation of storm surge. The contribution of wind to storm surge is often called wind setup. Wind blowing over the water causes a shear stress that is exerted on the surface of the water, pushing water in the direction of the wind. Wind shear stress is a highly nonlinear function of the wind speed (i.e., wind speed raised to the third power assuming a linear variation of surface drag coefficient with wind speed). For example, a wind of 30 knots produces roughly 27 times the surface wind stress of a 10-knot wind.

Wind is most effective in creating wind setup when it blows over shallow water, because the effect of wind and the water level is inversely proportional to water depth. In addition to water depth, wind setup also is a function of wind duration and fetch, or the distance over which a wind blows. A longer fetch is associated with a greater potential for wind setup. To illustrate, Figure D.3.1-4 shows the configuration and bottom bathymetry of Lake Michigan (deeper areas in blue and green, shallower areas in orange and red). Much of the main Lake Michigan water body is characterized by deep water. The north and south areas of Lake Michigan, where shallow water is most prevalent, is where wind setup in Lake Michigan is greatest. The potential for higher wind setup at the north and south ends also is due to the fact that the largest fetch is for winds from either the north or the south.

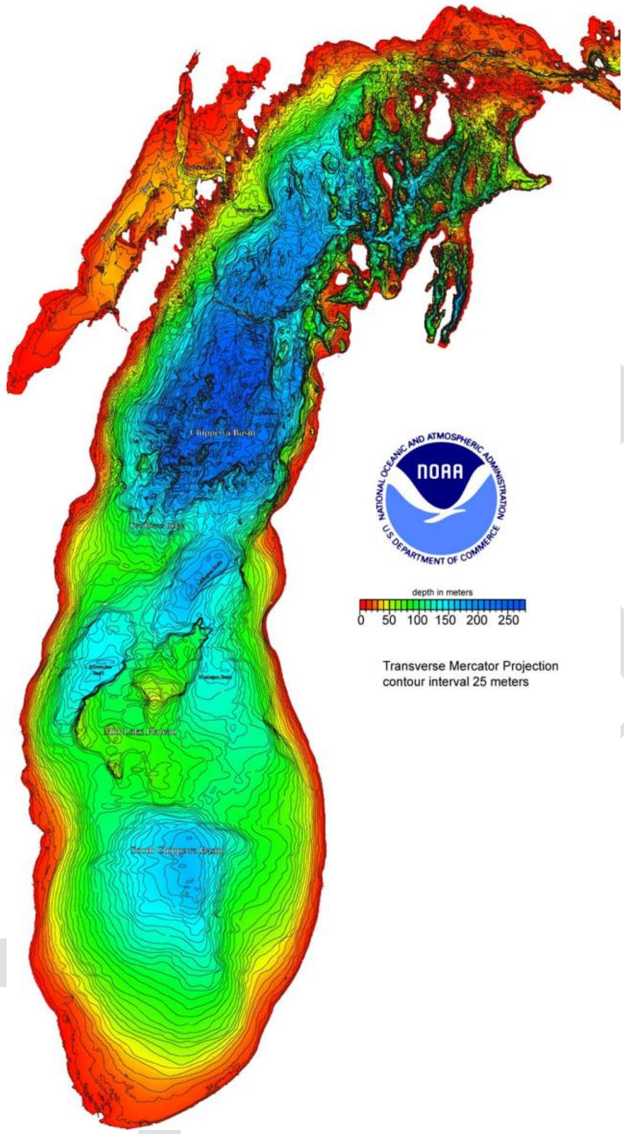


Figure D.3.1-4. Lake Michigan and Green Bay bathymetry. Image courtesy of NOAA, National Oceanographic Data Center.

Within Green Bay, which is the smaller, elongated water body on the western side of the lake, wind is relatively more effective in creating wind setup, because the bay is much shallower. The north and south ends of the bay are more prone to the development of wind setup, because the bay's elongated shape results in the longest wind fetch for winds from the north and the south.

The storm surge of record for the southern end of Green Bay is approximately 5 feet, whereas the surge of record for Calumet at the southern end of Lake Michigan is only 3 feet. Although Lake Michigan has a much longer fetch for strong winds from the north and the south, the effect of shallow water and an elongated embayment is more dominant and plays a stronger role in Green Bay. This difference in peak surge illustrates the key role of water depth in the generation of wind setup and storm surge. Wind setup along much of the east and west coasts of Lake

Michigan and Green Bay is smaller due to the absence of extensive shallow water and/or the shorter fetches for strong winds from the east to the west that would tend to pile up water against those coastlines.

Atmospheric pressure gradients are another forcing mechanism that contributes to changes in water level as water is forced from regions of high atmospheric pressure toward regions of low pressure. There is an elevated water-surface dome under the center of low pressure systems. This effect can be enhanced by a region of high pressure that is simultaneously situated over the opposing end of the lake. In the case of Lakes Michigan and Huron, which are coupled through narrow but deep straits, high atmospheric pressure over one lake and low pressure over the other lake will force water from the region of high pressure toward the region of low pressure, forcing water to move from one lake to the other through the Straits of Mackinaw. This pattern of water movement is generally not static; instead, it changes as the storm system moves through the region. Lakes Huron and Michigan respond rapidly to this pressure difference.

The component of storm surge associated with gradients in atmospheric pressure can be as much as 1 to 1.5 feet in Lake Michigan. In the central region of Lake Michigan, along the east and west shorelines where wind setup effects are less, this pressure-driven contribution can be as large as the wind setup contribution. In northern and southern Lake Michigan and Green Bay, the contribution due to wind setup is usually larger than the pressure contribution. Jensen et al. (2012) discuss in greater detail the storm surge response in Lake Michigan, associated with non-convective storm systems. Storm surge generation is very lake-specific and depends on prevailing storm winds and pressures, lake shape, and bathymetry. Storm surge can vary considerably around the periphery of a lake. Storm surge processes for Lake Michigan are discussed here to illustrate the different contributions to storm surge and how they can vary within a lake.

D.3.1.1.4 Seiche

A *seiche* is a standing wave that has been formed in an enclosed or semi-enclosed body of water. Seiches produce regular, periodic fluctuations of water levels as the standing wave travels between opposing shores within the lake. The most common cause of a large seiche in the Great Lakes is a storm that moves over the lake, with the resulting wind blowing parallel to the long axis of a lake for an extended period of time. The downwind portion of the lake is subject to the wind setup, where water piles up against the coast due to wind stress; the water level at the upwind end of the lake decreases, effectively tilting the water surface in the direction of the wind. When the storm abates and wind forcing is removed, the water that had piled up against the downwind shoreline flows back away from it and into the lake, exciting a wave motion as the water begins sloshing back and forth across the lake. Frictional losses cause the seiche amplitude to diminish over time. Winds and pressure gradients associated with squall lines can also produce a seiche.

Seiche events are not considered a unique flood hazard, since by definition they produce water elevations which are equal to or lower than the wind setup event that initiated them. Furthermore, as seiche is a water-level response observed primarily after the passage of a storm, the high water levels associated with the event will not necessarily be accompanied by large wave heights, which decrease rapidly as wind forcing subsides. Figure D.3.1-5 illustrates the occurrence of a large storm surge event for southern Green Bay, followed by a lower-amplitude seiche after the storm moves away. As is the case with storm surge, seiche generation and its characteristics are very lake-specific.

The Great Lake most affected by seiche is Lake Erie. The shallows in the western end of the lake and the narrowing profile of its eastern end, combined with the lake's long axis being aligned to the principle storm direction, make it ideal for large surges and strong seiching. An extreme event in January, 2008 is shown in Figure D.3.1-6. Large wind setup at Buffalo on the eastern end of the lake causes a matching strong set down at Toledo on the western end of the lake. Water then oscillates across the lake for several days with a long seiche period. Typical seiche events can last for 1 to 3 days with initial amplitudes of 3 to 5 feet observed on an annual basis in Lake Erie.

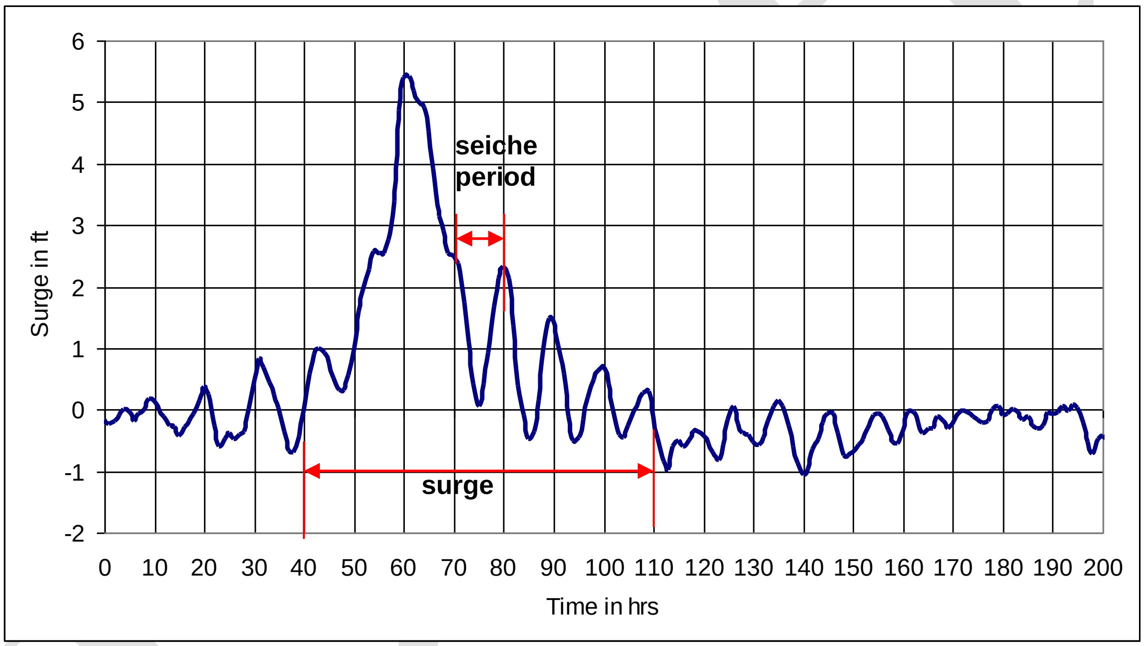
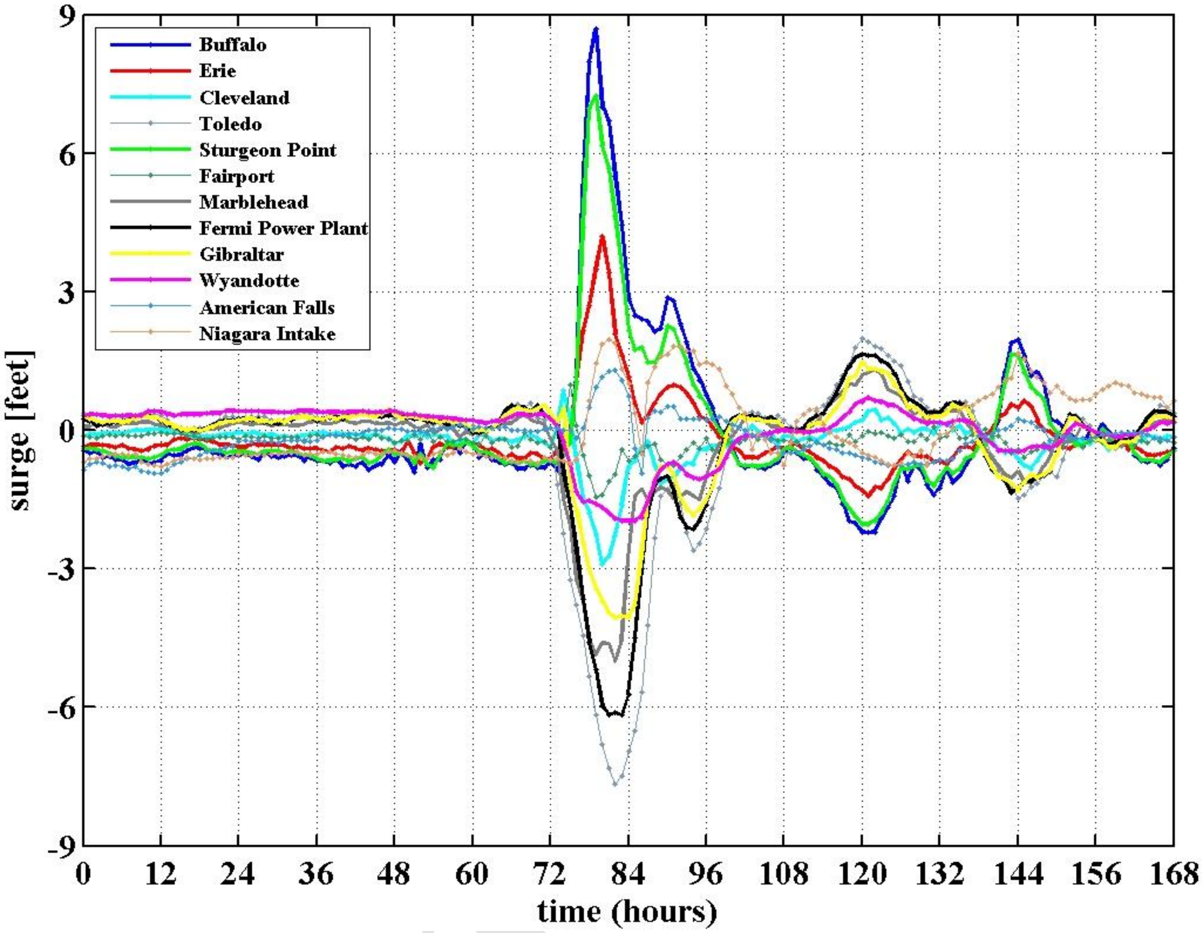


Figure D.3.1-5. Time series of water-level measurements showing storm surge and seiche from Green Bay, WI gage for a storm on Dec 3, 1990



(b)

Figure D.3.1-6. Time series of water-level measurements showing storm surge and seiche from gages around Lake Erie for a storm on January 30, 2008.

In the Great Lakes area, any sudden rise in the water of a harbor or a lake is sometimes called a seiche, whether or not it is oscillatory. This usage is inaccurate in a strict sense, but well established in the Great Lakes area nonetheless.

D.3.1.1.5 Tides

The Great Lakes are subject to the same astronomical forces that produce the tides observed along the ocean shoreline. The Canadian Hydrologic/Hydrographic Service reports a tidal response of less than 2 inches in the Great Lakes, the strongest being on Lakes Superior and Erie. These fluctuations are so small that their presence is masked by the water body's normal fluctuations due to atmospheric forcing. For all practical purposes, the Great Lakes can be treated as if no tidal signal exists, and this contribution to water levels is neglected within the analyses discussed in this appendix.

D.3.1.1.6 Storm Waves

Energetic short-period waves are generated by storm winds, which at elevated water levels can pose a significant coastal flood hazard (Figure D.3.1-7). Similar to the generation of wind setup, storm wave characteristics (height, period and direction) are strongly influenced by wind speed, direction, fetch, and duration of the wind from a particular direction. Higher wind speed, greater fetch distance and longer duration produce higher wave energy (height) and longer wave periods, in general. In the Great Lakes, fetch is strongly influenced by wind direction, due to the elongated nature of the water bodies. For example, the north and south coasts of Lake Michigan are more vulnerable to higher wave energy than the east and west coasts, because the fetch is much greater along the long axis of the lake. The generation of waves within the lakes is quite complex due to the sometimes rapid movement of storm systems through the region and the rapid changes in both wind speed and direction that occur. However, unlike storm surge, waves are very effectively generated in deep water and the most energetic waves are usually found in deeper water. Significant wave heights associated with severe storms in Lake Superior can exceed 30 feet, such as the storm that sunk the Edmund Fitzgerald on November 10, 1975. However, the largest buoy observed waves on the Great Lakes exceed 20 feet with wave periods in excess of 10 sec. In more sheltered areas, storm wave heights and wave periods are generally smaller. Great Lakes storm wave energy tends to grow quickly and diminish just as rapidly, responding directly to increases/decreases in wind speed.

As wind waves propagate into shallow water they refract, or bend. Incoming waves seek to align themselves in such a way that wave crests approach in a direction that is increasingly more parallel to the shoreline with decreasing water depth. This process of wave refraction generally results in decreases in wave height as waves approach the coast, although complex irregular bathymetry can create patterns of locally increased/decreased wave height. In shallow water, wave energy is dissipated due to bottom friction and white-capping and wave heights can decrease further. Waves eventually experience much stronger energy dissipation and subsequent decreases in wave height due to breaking in very shallow water.

Figure D.3.1-7. Wave Overtopping on the coast of Lake Ontario for a 1973 Storm, Edgemere Drive, Monroe County, NY. Photo Courtesy of Dr. Martin

As waves break on a beach, wave heights decrease and the flux of wave momentum in the onshore direction is reduced. In time-steady conditions, the excess wave force is balanced by a slope in the average water level called wave setup. The magnitude of wave setup is largest in shallow water, and the value is roughly 10 to 20 percent of the incident breaking wave height at the still water shoreline. Note that wave setup is only important in the breaking region, with the most pronounced effect in the inner surf zone and near the still-water shoreline.

At elevated water levels, broken waves run up on beaches and structures where they can pose a significant flood hazard. For incident waves having a significant wave height of 20 feet, wave runup elevations can reach 15 feet or more for a steep beach slope.

Breaking waves also can erode a beach berm, dune or bluff, especially when water level is elevated, due either to storms and/or elevated lake levels, exacerbating wave runup and overtopping. Dunes and bluffs are more susceptible to erosion at higher lake levels. Persistent overtopping of a dune can lead to erosion of the dune crest and loss of dune elevation, possibly causing complete degradation of the dune. If dune removal occurs, much greater wave energy can propagate inland, with the potential for increased damage to infrastructure and property. The duration of concurrent high water levels and energetic wave action associated with a storm is a strong factor in the magnitude of beach and dune erosion that occurs.

Wave generation and transformation, and the characteristics of waves, are lake-specific. Like storm surge, wave conditions are a function of the nature of storms that pass over the lake, of the wind patterns and speeds that are created, and of the shape and bathymetry of the lake. The variation of incident wave conditions, wave height decay in the surf zone, and generation of wave setup and runup, all can vary considerably from site to site within a lake.

D.3.1.1.7 Ice Cover Effects on Flooding

Ice cover along lake and bay shorelines can affect flooding risks. The typical extent and duration of winter ice cover changes from year to year, and from lake to lake. Ice cover typically reaches its maximum extent in late February. Ice cover is most consistently observed within shallower enclosed or semi-enclosed bays such as Sodus Bay on Lake Ontario. Long-term changes in ice cover might occur in the future because of global climate change. In the implementation of these guidelines, future ice conditions associated with climate change are not considered in the analysis. The nature and variability of ice cover is assumed to be that which has been experienced during the past 40 to 50 years.

In general, stable ice cover in the winter serves to reduce the flooding risk due to storm surge and wave action. Stable shore-fast ice cover along the coastline can serve to limit or wholly prevent wave energy from impacting the shoreline. Extensive ice cover across any region of the lake also can limit the generation of waves and storm surge as the wind stress has a shorter fetch upon which to act.

However, lower concentrations of ice have been found to increase wind stress that acts on the water surface. Banke and Smith (1973) examined the effect of sea ice on surface wind stress. More recent work (Birnbaum and Lupkes (2002) and Garbrecht et al. (2002)) has quantified the effect of form drag on the specification of wind drag coefficients within marginal ice zones, increasing wind stress under certain ice conditions. Wind-ice-water interaction is a highly complex process and not well understood.

Along with these potential reductions to flood risk, heavy ice cover in winter can reduce the amount of evaporation from the Great Lakes, and in turn lead to higher water levels the following spring. Conversely, ice-free winters and dry Arctic air masses passing over the lakes can increase evaporation loses in the winter.

Ice can also cause significant direct damage along Great Lakes shores. Figure D.3.1-8 shows an ice event from Lake Huron.

Figure D.3.1-8. Ice Event along the shore of Lake Huron

D.3.1.2 Coastal Flood Hazard Analysis and Mapping Considerations

This section introduces Great Lakes coastal flood hazard studies through a discussion of general study considerations, including the consideration of regional versus local studies and special considerations for sheltered waters. Descriptions of flood insurance risk zone definitions and reporting requirements also are provided. Detailed descriptions of flood insurance risk zone mapping and study documentation requirements are provided in Sections D.3.9 and D.3.10, respectively.

Guidance relating to preliminary study concerns such as Mapping Needs, Validation, and Scoping can be found in Appendix I.

D.3.1.2.1 Sheltered Waters

In comparison to open ocean coastlines along the Atlantic and Pacific oceans, the Great Lakes as a whole could technically be considered "sheltered waters," in that they are wholly bounded by land. The term "sheltered" often implies small or no storm surge and much lower wave energy. While recognizing that significant differences between oceanic and lake shorelines exist, because of the size of the lakes, the majority of Great Lakes shorelines are subject to flooding from both significant storm surge and large waves, the magnitudes of which are similar to storm surges and waves that can occur along parts of the open ocean coastline of the U.S. It is in relation to this modified understanding of "open coast" that the term "sheltered waters" is used in Section D.3 of these guidelines.

For the purposes of these guidelines, "sheltered" is assumed to imply a significant sheltering effect on wind and on the inland propagation of waves by land masses and vegetation. "Sheltered waters" are water bodies or smaller regions of a larger water body that experience diminished forces from wind and/or wave action relative to the open coast due to the presence of physical barriers, both natural and man-made, either on land or under water.

Sheltered water areas are exposed to the same flood-causing processes as are open coastlines (i.e., high winds, wave setup, runup and overtopping), but sheltering effects reduce the wave energy and potential for flooding. The Mapping Partner shall evaluate these potential sheltering effects, particularly at local scales. Detailed guidance on the analysis of sheltered waters is provided in Guidance for Coastal Flood Hazard Analyses and Mapping in Sheltered Waters (FEMA, 2008).

The basic presumption in conducting coastal wave analyses is that wave direction must have some onshore component in order to influence the 1-percent-annual-chance flood elevation. This presumption appears generally appropriate for open coasts and along many mainland shores of large bays, where the 1-percent-annual-chance flood elevation includes some contribution from storm surge and thus requires an onshore wind component that also generates onshore-directed wave energy. However, an assumption of onshore waves coincident with a high surge may require detailed justification along the shores of connecting channels, in complex embayments, and behind protective islands.

D.3.1.2.2 Beach Nourishment and Constructed Dunes

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

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

The Mapping Partner shall determine whether beach nourishment affects a study area, research any past beach nourishment projects, identify any available data that would allow the performance of the beach nourishment project(s) to be assessed, and determine whether or not the beach nourishment is likely to persist and have an effect on flood hazard mapping. If it is determined that beach nourishment will likely affect flood insurance risk zones or BFEs, the

Mapping Partner shall contact the FEMA Study Representative to determine whether an exception to current FEMA policy should be considered.

D.3.1.2.3 Special Regulatory Consideration—Primary Frontal Dune

As a result of changes to the NFIP regulations, coastal flood studies undertaken since the 1990s have analyzed and mapped dune ridge systems and assessed whether these features are able to withstand storm-induced erosion and remain as barriers to coastal flooding. A sample of a narrow coastal dune on a barrier beach is presented in Figure D.3.7-18 for the Eastern Lake Ontario site, in Oswego County.

Figure D.3.1-9. Dune on Barrier Beach, Eastern Lake Ontario, Oswego County

Those dunes meeting specific NFIP criteria are designated as primary frontal dunes (PFDs). Section 59.1 of the NFIP regulations defines a PFD as "a continuous or nearly continuous mound or ridge of sand with relatively steep seaward and landward slopes immediately landward and adjacent to the beach and subject to erosion and overtopping from waves during major coastal storms." The regulations also state that the inland limit of the PFD, also known as the heel of the dune, "occurs at the point where there is a distinct change from a relatively steep slope to a relatively mild slope." The inland limit of the PFD establishes the minimum landward limit of the VE Zone area. See section D.3.9 for more guidance on flood hazard mapping.

There are some locations in the Great Lakes Basin that feature very large relic coastal dunes that formed following a high phase in Great Lakes water levels known as the Nippising Transgression over 4,000 years ago (Baedke and Thompson, 2000). These dunes, often parabolic in shape, can exceed 100 feet in elevation and have a footprint of many hundreds of feet inland. Further, there can be successive rows of the parabolic dunes and thus the overall footprint of the dune field can be very large. Although these dunes are susceptible to toe erosion at high lake levels, the entire dune will not erode for single storm events. A sample of these large relic dunes is seen in Figure D.3.7-19 below.

Figure D.3.1-10. Sample of a Large Relic Dune, Mount Baldy, Indian Dunes National Lakeshore, Lake Michigan

Although these large dunes often form a continuous ridge and the face is subject to erosion at high lake levels, given their overall size, care must be taken when determining the location of the PFD as Section 59.1 of the NFIP regulations may not exactly apply to these features.

D.3.1.2.4 Data Requirements

To conduct a study for a coastal county, the Mapping Partner shall first collect the wide variety of quantitative data and other site information necessary to perform the required analyses. In addition to the necessary quantitative information, the Mapping Partner shall collect descriptions of previous flooding and descriptions of the county in general to aid in the evaluation of flood hazards and for inclusion in the FIS report. In addition to state agency and university resources, national resources such as the United States Army Corps of Engineers (USACE), United States Geological Survey (USGS) and NOAA, GLERL (within NOAA) also offers a wealth of expertise and data specific to the Great Lakes.

D.3.1.2.4.1 Transects Layout

At the county-scale flood hazard mapping is done for reaches along the coast with similar physical characteristics. It is important to ensure that there is a visible distinct change in physical characteristics between reaches. Transects that represent each reach should, in general, be selected perpendicular to the local bathymetric contours and shoreline. The Mapping Partner performing the analysis shall locate transects with careful consideration of the physical and cultural characteristics of the land so that transects will closely represent conditions in their locality. Transects shall be placed closer together in areas of complex topography, dense development, unique flooding, and areas where computed wave heights and runup are expected to vary significantly. Wider spacing may be appropriate in areas with more uniform characteristics. For example, a long stretch of undeveloped shoreline with a continuous dune or bluff of fairly constant height and shape and similar landward features might require transects to be placed every 1 to 2 miles. However, a developed area with various building densities, protective structures having different characteristics, and vegetation cover might require transects to be placed every 1,000 feet or less.

If good judgment is exercised in placing required transects, the Mapping Partner will avoid excessive interpolation of BFEs between transects, while also avoiding unnecessary study effort. In areas where wave runup might be significant, the proper location of transects is governed by variations in beach morphology (e.g., barred versus unbarred profiles, dune versus no-dune, bluff versus dune) and surf zone beach slope. On coasts with sand dunes, the Mapping Partner shall site transects according to major variations in the dune geometry (e.g., dune crest elevation and the dune volume per unit length of shoreline that is present above the historic high lake level elevation) and the upland characteristics. In areas where dissipation of wave heights in inundated areas may be most significant in the computation of flood hazards, the Mapping Partner shall base transect locations on variations in topography and land cover (i.e., buildings, vegetation, and other factors) that can influence wave transformation. The Mapping Partner should site a separate transect at each flood protection structure.

The physical and cultural characteristics used to identify the reaches that define the coastal transects should be documented. The characteristic data for each transect should not be taken along the line, but rather be representative of the characteristics of the reach.

D.3.1.2.4.2 **Bathymetry**

Bathymetry data are required for the lakewide modeling of regional-scale storm wave and water-level information. In general, the best available data that meets the resolution requirements of the modeling effort should be used. Data can be acquired from the NOAA GLERL and other NOAA sources although any reliable source may be used. The density of NOAA data is generally sufficient for regional-scale modeling of the offshore and refraction/shoaling zones.

For county-scale transect analyses it is not possible to provide precise guidance on the lakeward extent of bathymetry needed for a Great Lakes FIS. The extent primarily depends on the magnitude of incident storm wave conditions. For most shore types and open coast settings, bathymetry out to water depths of approximately 30 feet is required for wave transformation evaluations. In more sheltered areas with less energetic storm wave conditions, bathymetry out to water depths of 10 feet or even less might suffice.

LIDAR data provide an excellent source of shallow water bathymetry data for characterizing the surf zone and inundation zones, from which to extract information along transects. LIDAR data, where available from NOAA, Joint Airborne Lidar Bathymetry Technical Center of Expertise (JALBTCX), or USGS, are the primary data source to be used to define nearshore bathymetry and coastal topography along transects for Great Lakes flood risk mapping purposes. Beach profile surveys, or bottom elevations inferred from nautical charts or from USACE bathymetric surveys, are other alternative data sources for defining nearshore bathymetry. Bathymetric data can be acquired from NOAA National Ocean Survey and the NOAA Coastal Services Center's Digital Coast web site, and from the USACE for their holdings.

D.3.1.2.4.3 **Topography**

Detailed guidance on topographic data standards can be found in Appendix A of these guidelines. Use of accurate, high resolution topography data is of primary importance for producing a correct and defendable FIS. Topographic data must extend at least to the Low Water Datum (LWD) defined for each Great Lake, as listed in Table D.3.1-2, and landward to the inland limit of flooding at the 0.2-percent level. LIDAR data, where available, can be used to define both topography and bathymetry elevations for Great Lakes mapping studies.

LWD was established in 1933 and has remained unchanged. While vertical datums used on the Great Lakes have changed several times since 1933, the definition of LWD has not. Presently, LWD is described in terms of the International Great Lakes Datum of 1985 (IGLD85). The Mapping Partner shall convert to NAVD88 from IGLD85 for each coastal flood hazard analysis site. There needs to be some care in transferring IGLD85 elevations to NAVD88. If elevations used are based on land benchmarks, hydraulic correctors and/or dynamic height adjustments may need to be applied. NAVD88 is required as the datum for the topographic data.

Table D.3.1-2. Elevations of Low Water Datum on the Great Lakes

Location	Low Water Datum Elevation	
	Feet Above IGLD85	Feet Above NAVD88
Lake Superior	601.1	601.0
Lake Michigan	577.5	577.6
Lake Huron	577.5	577.6
Lake St. Clair	572.3	572.5
Lake Erie	569.2	569.4
Lake Ontario	243.3	243.4

The topographic data, usually in the form of digital elevation data or maps, must be recent and must reflect current conditions or, at a minimum, conditions at a clearly defined time. Transects do not need to be surveyed unless available topographic data are unsuitable or incomplete. The Mapping Partner shall examine the topographic data to confirm that the information to be used in the analysis and mapping represents the actual planimetric features that might affect identification of coastal hazards.

If possible, the Mapping Partner shall field-check shore topography to note any changes caused by construction, erosion, coastal engineering, or other factors. The Mapping Partner shall document any significant changes with location descriptions, drawings, and/or photographs.

The community, county, and State can be sources for topographic data, including LIDAR data. Other sources are LIDAR surveys flown by the JABLTCX, USGS, and NOAA. If gaps in the LIDAR data exist, the best available data should be identified. If the best available data does not meet the standards set forth in Appendix A, the Mapping Partner should consult with the FEMA Study Representative for approval of its usage.

D.3.1.2.4.4 **Land Cover**

Transect land-cover data is necessary for runup calculations (slope roughness factors), and for inundation and overland wave propagation considerations (wave damping and effects on storm surge). The necessary information includes descriptions of both structures and vegetation. It is imperative that the contractor obtain aerial photography not more than five years old unless the data can be supplemented by field reconnaissance. A local county, State, or Federal agency may have the coastline photographed on a periodic basis. That agency may provide the photographs or give permission to obtain them from its contractor. Because topographic maps are often developed from aerial photographs, the Mapping Partner also shall contact the mapping contractor for the topographic maps, if available.

Aerial photographs can provide the required data on buildings, tree and bush-type vegetation, and can be used to identify marsh areas, though not the specific type of grass-like vegetation. National Wetland Inventory maps from the U.S. Fish and Wildlife Service, and color infrared aerial photographs can provide the more specific data required for marsh plants. Land-use and land-cover-type maps from the USGS and NASA can be helpful, as can other types of remotely sensed imagery that is acquired by federal agencies, such as JABLTCX, and others. Buildings should be confirmed to be slab-on-grade or pile-elevated foundations. Ground-level photographs and site reconnaissance are also useful in providing information on plants (e.g., density, species). State offices of coastal zone management, park and wildlife management, and/or natural resources as well as local universities and Sea Grant programs should be able to provide information on significant vegetation types. Also, many communities now have digital land use data. The Mapping Partner may conduct field site reconnaissance in lieu of the above sources, but on-the-ground reconnaissance is most cost effective when used only to verify some of the data obtained from these other sources.

D.3.1.2.4.5 **Historical Floods**

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. Quantitative and qualitative information, such as the areal extent of flooding, high water marks, and location of buildings flooded and damaged by wave action, can be used to verify the results of the coastal analyses. Detailed information on pre- and post-storm beach or dune profiles is valuable in checking the results of the erosion assessment. When quantitative data are available on historical flooding effects, the Mapping Partner shall make an effort to acquire all recorded water elevations and wave conditions for the vicinity.

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

D.3.1.2.4.6 **Storm, Meteorological, Ice, Wave and Water-Level Data**

A number of different types of data are required to facilitate selection of storm events and to develop wave and water-level information for each storm at a lakewide scale. These data types include storm track and climatology data, meteorological data (such as winds and atmospheric pressures) that constitute forcing for waves and storm surge, data describing ice cover during storms that can influence generation of surge and waves, water-level data for characterizing long-term and seasonal-scale lake level changes, and wave and water-level data for model skill assessment. Most of the required data sets are produced by, and are available from, federal agencies; although state and local agencies and universities might also be valuable sources of data and local knowledge. A number of useful federal sources for these types of storm, water level, wave, ice and meteorological data are cited below

Storm Climate Data

Storm climate and historical storm data can be acquired from the following sources:

U.S. Dept. of Agriculture Weekly Weather and Crop Bulletin:
http://usda.mannlib.cornell.edu/MannUsda/viewDocumentInfo.do?documentID=1393

NOAA National Weather Service: http://www.nws.noaa.gov/

University of Wisconsin Satellite Observations:
http://www.ssec.wisc.edu/sose/glwx_activity.html

NASA Atlas of Extratropical Storm Tracks: http://data.giss.nasa.gov/stormtracks/

Meteorological Data

A record of available station information can be obtained from (http://coastwatch.glerl.noaa.gov/marobs/).

Meteorological data can be acquired from the NOAA National Climatic Data Center (NCDC) at:
http://cdo.ncdc.noaa.gov/pls/plclimprod/poemain.accessrouter?datasetabbv=DS3505

The Global Integrated Surface Hourly (ISH) data base is the most complete archive of meteorological information, which can be retrieved from NOAA's National Climate Data Center (NCDC) (http://www.ncdc.noaa.gov/oa/climate/climatedata.html). Software and documentation are also available from the following site (http://www1.ncdc.noaa.gov/pub/data/ish/)

NOAA's National Oceanographic Data Center (NODC) has all of the NOAA National Data Buoy Center (NDBC) data which includes met data (http://www.nodc.noaa.gov/BUOY/buoy.html)

National Centers for Environmental Prediction Climate Forecast System Reanalysis (CFSR) is based on a re-analysis program of all meteorological products generated by NOAA's National Center for Environmental Predictions and can be accessed at: http://dss.ucar.edu/pub/cfsr.html

Canadian data can be obtained from the National Climate Data and Information Archive at http://www.climate.weatheroffice.gc.ca/climateData/canada_e.html.

Ice Data

NOAA GLERL Ice Concentration Data Base (1960 to 1979). http://nsidc.org/data/g00804.html

NOAA GLERL Digital Ice Atlas (1973 to 2002), http://www.glerl.noaa.gov/data/ice/atlas/

NOAA GLERL ice thickness data (1966 to 1979), http://nsidc.org/data/g00803.html

NOAA GLERL digital ice cover data (2003 to 2009); obtain directly from GLERL

Water-Level Data

Water-level data acquired and served by NOAA can be found at the following site:
http://tidesandcurrents.noaa.gov/station_retrieve.shtml?type=Great+Lakes+Water+Level+Data

Wave Data

Measured wave data from NOAA NDBC buoys can be obtained from NOAA's National Oceanographic Data Center (http://www.nodc.noaa.gov/BUOY/buoy.html)

Hindcast wave data can be acquired from the U.S. Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Wave Information Studies web site: http://frf.usace.army.mil/cgi-bin/wis/atl/atl_main.html

D.3.1.2.5 Reporting Requirements

The data standards for these requirements are described in Appendix M: Data Capture Standards of these Guidelines.

Due to the complexity of coastal studies, intermediate data submissions are required from the Mapping Partner. Intermediate data submissions provide defined milestones in the coastal flood study process and independent reviews are conducted to confirm that the methods and findings are acceptable to FEMA. The primary purpose of this submission and review process is to minimize revisions to analysis methods later in the study. Specific information on reporting requirements is provided in Section D.3.10. Intermediate data submissions are not required to conform to Appendix M standards, though much of the information assembled for the intermediate data submittals will also be required for the final study documentation and data archival.

D.3.2 Methodology for Analyzing Coastal Processes

This section provides guidance for selecting and combining specific technical methods and data into a study methodology for characterizing coastal processes and their role in flooding. The selection of a specific method will depend on the coastal setting and available data.

In this appendix, “methods” refers to the individual techniques used to make specific computations. “Study methodology” is the combination of appropriate methods and data necessary to develop flood insurance risk zones for depiction on a FIRM. A variety of technical methods are available for application to the unique settings along the coast, with those most appropriate for the Great Lakes coasts presented in Sections D.3.3 through D.3.7. In some cases, several methods may apply to a specific coastal setting, and in some cases, methods used for one setting might differ from those used for a different setting. The objective of this section is to provide guidance for developing an appropriate methodology based on the coastal setting and available data.

The recommended study methodology for Mapping Partners to follow in developing flood insurance risk zones and maps is summarized below. It is important to remember that the objective of this document is to provide the guidance necessary to develop flood hazard zones and maps. All coastal processes that can produce the 1-percent-annual-chance flood elevation must be considered. Consideration must be given to what data and technical methods are appropriate for application, and what existing data is valid to use in the determination of BFEs and flood insurance risk zones. The level of technical analysis should remain consistent with this objective. It is only necessary to obtain data and conduct analyses required to accomplish this objective. Because there are often several methods available to conduct similar analyses, the Mapping Partner must choose methods that are technically sound and consistent, are applicable for the study setting, and have been validated to the extent possible for Great Lakes coastal settings.

Decisions regarding which methods and methodology to apply must consider the importance of, and the relative contributions of, various coastal processes to the BFEs. For example, in Lake Michigan, long-term lake level changes vary over a range of 6 feet; seasonal lake level changes vary over a range of 1 foot. Storm surge can reach 2 to 5 feet along most of the shoreline. Offshore significant wave heights can reach 20 feet along some sections of shoreline. Values of wave runup that correspond to this level of incident wave energy can reach 15 feet, respectively. The level of analysis effort devoted to characterizing each contributor to the BFE (lake level, storm surge, offshore waves, nearshore waves, wave runup, erosion, etc.) should be consistent with its relative contribution and importance to the BFE. The relative roles of the various contributors to the BFE are lake-specific and they can vary considerably along the lake shoreline.

At the outset of the study process, the Mapping Partner should begin the onshore analysis by identifying the information required to develop the flood insurance risk zones and map. This involves identifying all physical coastal processes likely to contribute to flood hazards in the study area, and their interaction with particular coastal settings in the onshore, nearshore surf and wave shoaling zones, and offshore in the study area. In some cases, this initial review will not resolve all questions related to coastal processes and hazard zone definition. The review should identify the data requirements for one or more methods that can be applied to make these determinations.

After a review of probable hazards at the shoreline, the Mapping Partner should proceed offshore, considering what data and analyses are required at each level and for each setting within the study area to accomplish the onshore analysis. This will establish the limit of the offshore data and computations necessary to conduct the analyses. Once the offshore data requirements for the study are established, the wave data and other information will be “brought” back onshore to determine the information needed to develop the hazard zones. In other words, the mapping needs are established by progressing from the hazard map to the offshore area, but the analysis is done in a manner that is consistent with the physical processes — from offshore to onshore. Different data requirements are associated with different analysis methods. More advanced methods generally require additional data. New methods that have been developed for wave runup and setup, and beach/dune erosion, might require a higher-level of input data preparation, and the level of effort expended to acquire and prepare the input data for a particular method depend on its significance to the flood hazard in the detailed coastal analyses.

Figure D.3.2-1 summarizes the basic steps in selecting analysis methods. This logic may be applied to both the overall study methodology and to selection of methods for each major coastal process to be analyzed in developing flood hazard zones. The basic process begins with the definition of objectives, which should focus on the development of flood hazard zones at an appropriate resolution and level of accuracy that considers potential damages, inherent uncertainty in the analyses, schedule, and budget. The geomorphic setting is a key factor in identifying the dominant physical processes that must be analyzed and the appropriate methods for analysis. Potential methods applicable to a given setting may have different data requirements, and the availability of data may influence the selection of methods. Once a methodology has been defined (a combination of methods and data), the Mapping Partner must confirm that the methodology satisfies the study objectives, including time and budget constraints.

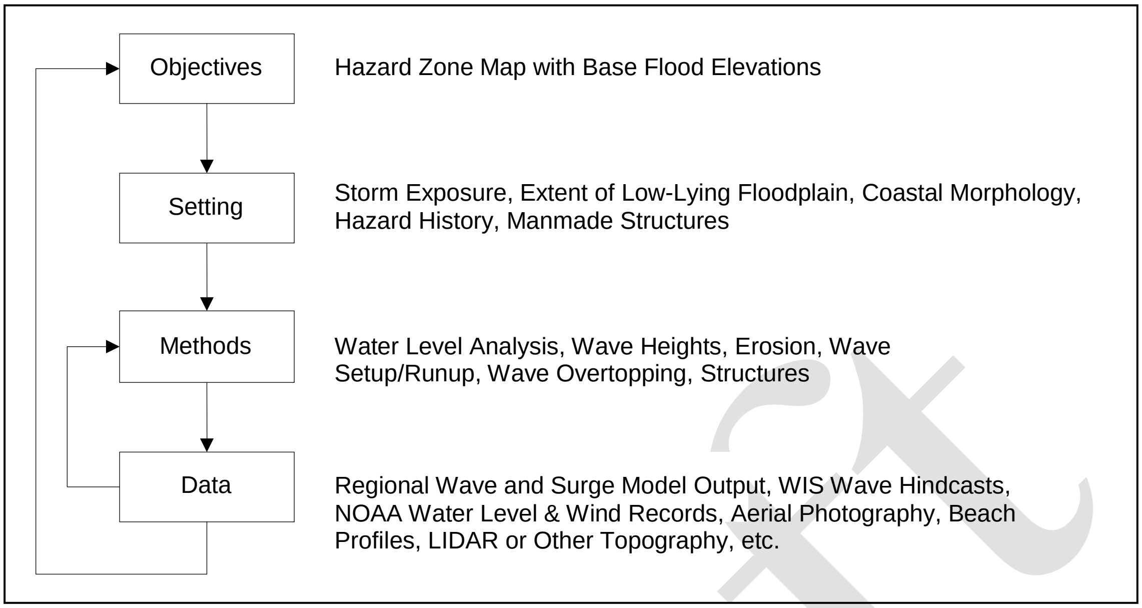


Figure D.3.2-1. Study Methodology Development Considerations

D.3.2.1 Overview

First, a composite storm set is defined using the methods outlined in Section D.3.3 and described in greater detail by Melby et al. (2012) and Nadal-Caraballo et al. (2012). Events in the response-based approach are the storms from the period of record that make up the composite storm set. Each storm is simulated using high-resolution regional-scale, lakewide 2-D storm surge and wave models, and the models are applied with the historic lake level that existed at the time of the storm, as the initial lake level for the model simulations. Once the full set of storm simulations is completed, water-level and wave responses (i.e., the times series of waves and water levels) are available for each storm event and for many locations throughout a lake. A flooding event in the response-based approach corresponds to a set of time-dependent wave and water-level conditions taken as a paired data set with a specific duration.

These storm responses are then used directly in a statistical analysis to establish BFEs, or used to support additional analyses done at transects in order to establish BFEs. Storm responses at transects (i.e., waves and water levels) can be used to determine other responses for analysis of the surf zone and backshore zone. The maximum response from each storm event is identified and then the maxima for all events in the period of record are statistically analyzed to determine the 1-percent-annual-chance flood response. The POT/GPD method outlined in Section D.3.3 is used to compute BFEs and any other extremal statistics such as the 0.2-percent-annual-chance flood levels.

Depending on the coastal setting, the 1-percent-annual-chance determination for flood level is made based on a statistical analysis of the still water level (lake level plus storm surge) or total water level which is the sum of still water level and runup. The 1-percent-annual-chance responses can be determined at the boundary of any one of the coastal zones described in Section

D.3.2.3. However, the further the response-based approach can be practically carried onshore, the better the estimate of the 1-percent-annual-chance flood response in the backshore zone.

The dominant flood hazard (i.e., the hazard resulting in the greatest BFE) for a given reach of shoreline is typically caused by either wave runup or overland wave propagation. Wave runup is the uprush of water from wave action on a beach or shore barrier. A shore barrier can be a beach, dune, steep bluff, shore protection structure (e.g. rubble revetment or seawall), etc. Overland wave propagation refers to the propagation of waves inland in areas inundated by flooding associated with the still water level (lake level plus storm surge). The primary factors determining the dominant flood hazard are the slope of the ground or barrier, depth of flooding, and wave height.

Wave runup will likely be the dominant flood hazard along reaches of shore where the still water level intersects relatively steep terrain and the steep terrain allows waves relatively close to shore before breaking. The water wedge from a broken wave generally thins and slows during its excursion up the barrier as residual forward momentum is reduced or reflected. The BFE in runup areas is the wave runup elevation--the vertical height above the still water level ultimately attained by the extremity of the uprushing water. In these guidelines the measure of wave runup that is adopted is the 2-percent wave runup elevation, i.e., the elevation that is exceeded by only 2 percent of the individual incident wave runups. Wave runup is discussed in greater detail in Section D.3.5.

Overland wave propagation will likely be the dominant flood hazard along reaches of shore where the 1-percent-annual-chance still water elevation inundates relatively low, flat terrain. Waves will become depth-limited as they propagate inland and are dampened by obstructions such as vegetation and buildings. By the time the wave reaches the point of intersection between the still water level and ground, the wave energy is typically small and negligible. Thus, the boundary of flooding, or limit of the Special Flood Hazard Area, is located at the point where the ground elevation equals the 1-percent-annual-chance still water elevation in areas dominated by overland wave propagation. Wave propagation over inundated areas is discussed in more detail in section D.3.6.

There might be areas where it is difficult to determine the primary hazard associated with the BFE or where base flood conditions are defined by a combination of wave runup and overland wave propagation. An example of such an area is a low-lying bluff that is similar in elevation to annually-occurring still water levels. The 1-percent-annual-chance still water level will be greater than the bluff elevation and inundate the area inland of the bluff. However, using the response-based approach, wave runup will be calculated with water levels less than the 1-percent-annual-chance still water level causing wave runup incident on the bluff face. The 1-percent-annual-chance runup elevation might be greater than the wave crest elevation at the bluff calculated in the overland wave propagation analysis. In areas like this or in areas where the dominant flood hazard is not obvious, it will be necessary to evaluate the 1-percent-annual-chance flood hazard for both wave runup and overland wave propagation and construct a wave envelope profile.

D.3.2.2 Coastal Setting Considerations

The study area setting and flood hazard history will determine which methods and data are necessary and/or appropriate. Important considerations include the coastal exposure (open coast or sheltered water), morphology (e.g., sandy shoreline, dunes, bluffs, cliffs, etc.), and the shore conditions (topography, irregularity of nearshore bathymetry, presence or absence of a protective structure and its type, presence of absence of vegetation and its type, development and infrastructure presence, etc.). Consideration of each of these factors frames the data requirements and the appropriate analysis methods.

D.3.2.2.1 Open Coast and Sheltered Water

A primary consideration is the exposure of the coast: either open coast or sheltered water. Open coast settings are exposed to the full influence of storm waves and whatever storm surge might be present. In sheltered water, the waves will be strongly fetch-limited and the local surge may be different from that on the open coast. However, there might be instances where the transmission of open coast wave conditions into sheltered areas dominates the flooding scenario. The degree to which this occurs depends on the geometry of the connections between water bodies, the bathymetry and wind and wave direction. The degree to which open coast storm surge penetrates into a sheltered area is influenced by a number of factors, including: 1) magnitude and duration of storm surge, i.e., hydrograph shape, 2) presence or absence of a channel or conduit for water to flow from the open coast to the sheltered area, 3) characteristics of the channel or other conduit such as length and cross-sectional area and how they vary with the level of inundation, and 4) size and complexity of the water body in the sheltered area. On the open coast, the interrelationships among waves and water-level processes might be quite complex, and simultaneous measurements and/or model simulations of these processes are recommended to avoid having to reconstruct, overly simplify, and approximate the complex interrelationships.

While most methods for open coasts are also applicable to sheltered water, a number of special considerations for sheltered water exist. Detailed guidance on the analysis of sheltered waters is provided in *Guidance for Coastal Flood Hazard Analyses and Mapping in Sheltered Waters* (FEMA, 2008).

D.3.2.2.2 Different Shoreline Types

The shoreline morphology determines which analysis tools are appropriate for estimating shoreline responses. The six general shoreline settings on the Great Lakes coast include:

- Sandy beach, possibly backed by a low sand berm or dune; or erosion-resistant beach profile having a small lens of mobile sand;
- Sandy beach backed by coastal development or shore protection structures;
- Cobble, gravel, shingle beach or mixed grain size beach;
- Erodible coastal bluffs and cliffs;
- Non-erodible coastal bluffs and cliffs;

- Wetlands; and
- Alvars1.

The shoreline morphology determines which analysis tools are appropriate for estimating shoreline response. Details of the specific methods for each coastal setting are given in Sections D.3.7 and D.3.8.

In all settings, the existing shoreline conditions must be determined. These are required to determine the present location of the shoreline; condition of structures; and ascertain if the profile includes an erodible sand berm, dune or bluff that requires consideration of event-based erosion. Profiles with a shore protection structure in the active coastal zone will require consideration of the structure's influence on flooding (see Section 3.8).

If required, an appropriate model will be used to yield an eroded profile. If the eroded profile results in dune breaching, structure failure, or bluff recession, then an adjusted final profile must be determined. Wave setup, runup, overtopping, and overland propagation are then determined for the final profile. These results are then used for mapping the flooding hazards.

D.3.2.3 Coastal Processes

Figure D.3.2-2 shows the cross-shore profile divided into four zones. The offshore zone is the region where waves, and to a lesser degree wind setup, are not substantially influenced by bathymetry. Dominant processes in this zone include lake level, wave growth and propagation, wave energy dissipation due to white capping, and storm surge. The shoaling zone is the area outside the surf zone where offshore wave conditions are transformed by interaction with bathymetry or topography and wind has a greater influence on generation of wind setup and storm surge. Wave transformation in this zone includes wave refraction, shoaling, diffraction, energy dissipation due to bottom friction effects and white-capping. The surf zone is where waves break as they interact with very shallow water and wave energy is limited by the local water depth. Dominant processes include lake level, storm surge, wave breaking, strong energy dissipation, generation of wave setup, runup, overtopping, beach and dune erosion, and wave interaction with structures.

The backshore zone is the area that is outside the normal coastal surf zone, but may be subject to inundation, wave propagation, breaking and energy dissipation arising from a number of sources during coastal flooding events. This area often contains development and infrastructure and is the critical area for determination of flood hazards.

Figure D.3.2-2 shows the coastal processes as they are referenced in the description of analysis methods given in Sections D.3.4 through D.3.7. It should be noted that "offshore" does not necessarily imply deep water conditions, which for waves are defined according to water depth and wave length. Although this deep water condition is typical, an "offshore" designation might

[1] Alvars are an ecosystem unique to the Great lakes consisting of grassland, savanna and sparsely vegetated rock barrens that develop on flat limestone or dolostone bedrock where soils are very shallow.

only mean that the processes being considered are far outside the surf zone. If the offshore zone is not in deep water, then the offshore and shoaling zones are characterized by similar processes.

Computations made in each zone use data from the preceding zone and pass the results to the next zone. Computations generally start in the offshore zone.

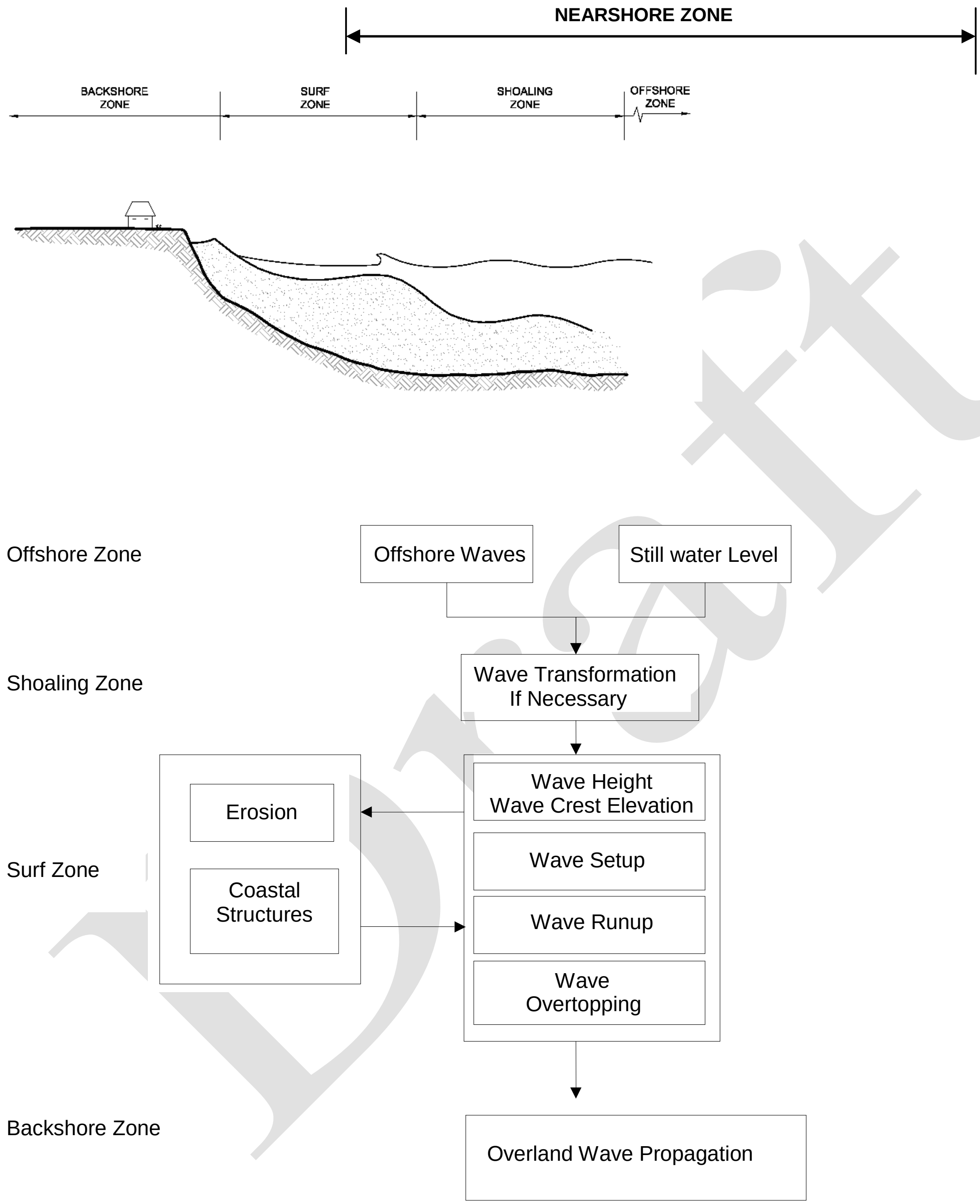


Figure D.3.2-2. Coastal Zones and Processes

D.3.2.3.1 Offshore Zone

In light of the movement of many large and intense storm systems through the Great Lakes region, and the sometimes rapid changes in wind and atmospheric pressure, wave and water-level information for the offshore zone should be developed using time-dependent regional-scale (lakewide) modeling. Two-dimensional spectral wave models and two-dimensional storm surge models are recommended to resolve the spatial patterns of storm surge and wave height. The offshore modeling is conducted to simulate water levels and wave conditions for historical storms for the main water bodies that comprise each lake.

Wave modeling in sheltered areas, such as lakes and bays can be handled in the lakewide modeling or using procedures in FEMA guidelines for sheltered areas. The resulting estimates for waves and water levels are then passed to the shoaling zone where wave transformation is evaluated.

D.3.2.3.2 Shoaling Zone

In the shoaling zone offshore waves are transformed onshore to a desired water depth, either inside or outside the breaker zone, using either a 2-D or 1-D wave transformation model, or perhaps an even simpler calculation method in those situations where the bottom bathymetry is regular with straight and parallel contours and assumptions inherent in the simple calculation method are valid. Wave transformation in the shoaling zone can be handled within the same regional wave and surge model domain that is used to generate information for the offshore zone if the wave model is applied with sufficient resolution to properly simulate the effects of refraction and breaking particularly in areas having irregular nearshore bathymetry. Storm surge for the shoaling zone can be treated in the regional, lakewide storm surge modeling.

D.3.2.3.3 Surf Zone

After waves have been transformed across the shoaling zone, the results are then passed on to the surf zone analysis. In some cases, dependent upon the incident wave conditions and the local beach slope and irregularity of local nearshore bathymetry, portions of the surf zone can be reliably treated in the modeling that was done for the offshore and/or the shoaling zones, provided the modeling is done with adequate resolution in shallower water. However, in general, the inner surf zone for most Great Lakes coastal settings will not be resolved well with the degree of model resolution that is typically adopted for regional wave modeling that includes the shoaling zone (30- to 300-m resolution is typically adopted for these zones). The inner surf zone is where the beach slope oftentimes has its maximum steepness, where irregularly-shaped bars are oftentimes present, and where substantial wave energy dissipation occurs in an oftentimes narrow zone adjacent to the shoreline. The inner surf zone is where wave height and wave setup gradients are greatest, and where much of wave set up is generated including the maximum wave setup at the shoreline.

Use of one-dimensional surf zone dynamics models for transects, applied at a cross-shore resolution on the order of meters, allows for treating the following important coastal processes in a single calculation step: 1) surf zone breaking and wave energy dissipation that accounts for the influence of irregular morphology, 2) beach erosion which creates a steeper foreshore slope during storms which in turn increases the wave runup, 3) possible erosion of dunes that have been created during the low lake levels and subsequent increase in flood hazard that can arise from dune degradation at higher lake levels, and 4) a better estimate of wave setup and runup at the shoreline where the maximum value of wave setup occurs. Wave setup is a significant contributor to storm surge on the Great Lakes, comparable in magnitude to other contributors in some lakes. Wave runup is the dominant contributor to BFEs for large segments of the Great Lakes shoreline.

An alternative to 1-D surf zone dynamics modeling is to use simple computational formulas for calculating storm responses such as wave runup, overtopping of structures, beach erosion etc. For those cases where the local coastal setting and wave/water-level conditions are similar to those that were used to derive the simple empirical prediction methods, such as wave runup on a planar slope or overtopping of a planar sloped rubble-mound coastal structure, the simple calculation approaches provide an alternate and less computationally intensive method.

The surf zone results do not influence wave transformations in the shoaling zone, so wave transformation in the shoaling zone may be determined independently of the surf zone. The structure of this appendix reflects this independence. Surf zone computations use nearshore bathymetry and either the wave conditions determined outside the breaker line or conditions from the regional-scale modeling in the shoaling or surf zones.

D.3.2.3.4 Backshore Zone

In the backshore zone, information from the surf zone is combined with topography and data describing land use type to evaluate overland wave propagation with FEMA's WHAFIS program.

D.3.2.4 Summary of Analysis Methods

Table D.3.2-1 is a summary of methods presented in Sections D.3.3 through D.3.9. This table provides an overview of available methods and a reference to the appropriate section of the guidelines document where more detail is provided.

Table D.3.2-1. Summary of Methods Presented in Section D.3

Zone/Process	Method	Comments
All Zones	**Storm Sampling and Statistics (D.3.3)** 0.2- and 1-percent-annual-chance-conditions are computed using Peak over Threshold approach and Generalized Pareto Distribution statistical analysis method, with distribution fitting using the CDF and Q-Q analysis techniques.	Storm response maxima for period of record are used to determine the 0.2- and 1-percent-annual-chance storm responses. Storm sampling approach generally follows that of Melby et al. (2012) and Nadal-Caraballo et al. (2012) GPD fitting using method of Nadal-Caraballo et a. (2012)

Table D.3.2-1. Summary of Methods Presented in Section D.3

Zone/Process	Method	Comments
Offshore Zone	**Water Level (D.3.4)** Storm surge modeling using two-dimensional (2-D) time-dependent shallow water long-wave model, and validated for an appropriate number of historic severe storm events Simulations made for historic events	Computed using measured lake level at the time of each storm, using most recent set of IJC BOC modifications In most cases, the measured monthly mean value, or running 30-day average value can be used for mean lake level associated with each storm For surge model input use best available climatological and ice field data or hindcast Best available climatological data or hindcast should be used for storm selection.
Offshore Zone	**Waves (D.3.4)** Wave Generation and Propagation Two-dimensional (2-D), time-dependent spectral wave models to simulate events, and validate for an appropriate number of severe historic storm events Coastal Engineering Manual (CEM) simple parametric methods for sheltered areas, subject to approval	Simple parametric models should only be used in sheltered waters with restricted fetches. Waves computed using measured lake level at the time of each storm, using most recent BOC modifications For wave model input use best available climatological and ice field data or hindcast. Best available climatological data or hindcast should also be used for storm selection
Shoaling Zone	**Wave Transformations (D.3.4)** Straight and parallel bathymetric contours Simple calculation method for refraction, shoaling using Snell's Law, breaking 1-D surf zone dynamics model if regular contours Nearshore transformations over irregular bathymetric contours 2-D spectral and time domain models	2-D numerical models are typically only required for complex bathymetry. Couple surge and wave models if effect of storm surge on water depth and wave transformation is important, and perhaps to treat radiation stress contribution to storm surge if wave setup major contributor to storm surge. Use wave and water-level data derived from regional-scale wave and surge modeling as input

Table D.3.2-1. Summary of Methods Presented in Section D.3

Zone/Process	Method	Comments
Surf Zone	**Wave Setup and Runup (D.3.5)** Beaches 1-D surf zone dynamics model preferred Advanced Model – Boussinesq Empirical methods- Modified Mase or Stockton runup method where 1-D surf zone model not applicable Direct Integration Method (DIM) for wave setup if not implicitly included Structures Empirical methods- Van Gent, CEM 1-D surf zone dynamics model, particularly if empirical method not appropriate Advanced model - Boussinesq or RANS class of model	Most runup methods implicitly include wave setup. 1-D surf zone dynamics model should be used on transects unless other methods are more applicable. Apply 1-D surf zone model in fixed bed mode. If applicable, apply 1-D surf zone model in erodible bed mode to obtain eroded profile then run model in fixed bed mode on eroded profile. Advanced models are only considered for highly complex conditions and/or situations with unusually high consequences. Use Stockton runup method for dissipative, gently sloping beaches Couple surge and wave models to treat effects of storm surge on water depth and radiation stress contribution to storm surge.
Surf Zone and Backshore Zones	**Erosion (D.3.7)** Beaches 1-D surf zone dynamics model Shore Protection Structures 1-D surf zone dynamics model for scour estimation CEM scour equations Cobble Beaches Observed storm profiles 1-D surf zone dynamics model if appropriate Erodible Bluffs 1-D surf zone dynamics model if appropriate Simple empirical methods Non-Erodible Bluffs and Cliffs No erosion Mud Flats and Wetlands No erosion	

Table D.3.2-1. Summary of Methods Presented in Section D.3

Zone/Process	Method	Comments
Surf Zone and Backshore Zones	**Overtopping (D.3.5)** Beaches Goda, CEM, EurOtop 1-D surf zone dynamics model Boussinesq or RANs model Structures Goda, CEM, EurOtop 1-D surf zone dynamics model Boussinesq or RANS model	Advanced models are considered for complex structure/beach configurations and/or situations with unusually high consequences.
Backshore Zone	**Overland Wave Propagation (D.3.6)** Wave Height Analysis for Insurance Studies (WHAFIS)	
Backshore	**Flood Hazard Mapping (D.3.9)** Runup depth Overtopping splash distance Overland wave crest elevation Primary Frontal Dune	

D.3.3 Methodology for Storm Sampling and Statistical Analysis

This section outlines general features of statistical and storm sampling methods that are to be used in a Great Lakes coastal Flood Insurance Study, including basic flood frequency analysis and storm sampling tools that are used. Important considerations in implementing the statistical approach also are covered.

D.3.3.1 1-Percent-Annual-Chance Flood Elevation

The primary goal of a coastal Flood Insurance Study (FIS) is to determine the flood elevations throughout the study area that have a 1-percent-annual-chance of being equaled or exceeded in any given year. The elevation at this frequency at a given location is called the 1-percent-annual-chance flood elevation, or level, at that location; and it has a probability of 0.01 of being equaled or exceeded in any given year. The terms flood level and flood elevation are used interchangeably through this appendix.

The 1-percent-annual-chance elevation might result from a single flood process or from a combination of processes that were discussed in Section 3.1.1. However, there is no one-to-one correspondence between the 1-percent-annual-chance flood elevation and any particular storm or other flood-producing event. The 1-percent-annual-chance level may be produced by any number of mechanisms, or by the same mechanism in different instances. For example, an incoming wave with a particular height and period for a particular still water level (storm surge plus long-term lake level) might produce the 1-percent-annual-chance runup elevation, as might a quite different wave with a different combination of height and period and still water level.

Furthermore, the flood hazard maps produced as part of an FIS do not necessarily display, even locally, the spatial variation of any one realistic physical hydrologic event. For example, the 1-percent-annual-chance water levels just outside and just inside an inlet will not generally show the same relation to one another as they would during the course of any real physical storm event because the inner waterway may respond most critically to storms of an entirely different character from those that affect the outer coast. Where a flood hazard arises from more than one source, the mapped level is not the direct result of any single storm or process, but is a construct derived from the statistics of all storms and sources. Note, that the 1-percent-annual-chance flood level is an abstract concept based as much on the statistics of floods as on the physics of floods.

Because the 1-percent-annual-chance flood level cannot be rigorously associated with any particular storm, it is erroneous to think of some observed event as having been the 1-percent-annual-chance flood event. A more intense storm located at a greater distance might produce the same flood level, or the same flood level might be produced by an entirely different storm and mechanism.

D.3.3.2 Statistical Analysis Methodologies

The flood level experienced at any coastal site is the complicated result of a large number of interrelated and interdependent factors. For example, coastal flooding by wave runup depends upon both the local waves and the level of the underlying still water upon which they "ride." That still water level (SWL), in turn, depends on the contribution of the transient storm surge and lake level at the time of the storm. The wave characteristics that control runup include wave height, period, and direction, all of which depend on the meteorological characteristics of the generating storm. Furthermore, the resulting wave characteristics are affected by variations of water depth over their entire propagation path, from offshore through the surf zone and the foreshore beach slope, and thus depend also on the varying storm surge. Still further, the beach profile is variable, changing in response to wave-induced erosion and causing variation in the wave transformation and runup behavior. Catastrophic erosion of a dune system might also cause a fundamental change in still water elevations. All of these interrelated factors may be significant in determining the coastal 1-percent-annual-chance flood levels. Simplifying assumptions are inevitable, whichever method is used, even in a response-based study, which attempts to simulate the full range of important processes over the duration of a storm.

These guidelines offer insight and methods to address the complexity of coastal flood processes. However, the inevitable limitations of the guidance must be kept in mind. No fixed set of rules can be appropriate in all cases, and the Mapping Partner must be alert to special circumstances that violate the assumptions of the methodology. A proactive application of best engineering practices is always preferable to the rote application of the analysis discussed in this document.

D.3.3.2.1 Event Selection Method

A great simplification is made if one can identify a single event (or a small number of events) that produces a flood level that represents the 1-percent-annual-chance flood elevation. This might be possible if, for example, a single event parameter is believed to dominate the final elevations, so the 1-percent-annual-chance value of that particular storm parameter might suffice to determine the 1-percent flood level. For example, in determining the wave runup elevation corresponding to a 1-percent annual chance of exceedance, one might identify a significant wave condition (height and period) thought to be exceeded with only 1-percent annual chance, and then to follow this single wave through its nearshore transformation, breaking and runup on the shoreline. This is the event-selection method.

Used with caution, this method may allow reasonable estimates to be made with minimal analysis effort. It is akin to the concept of a design storm, or to constructs such as the standard project storm or probable maximum storm. The inevitable difficulty with the event-selection method is that multiple parameters are always important, and it may not be possible to assign a frequency to the result with any confidence because other unconsidered factors always introduce uncertainty. In the case of runup, for example, smaller waves with longer periods might produce greater runup than the larger waves and shorter periods selected for analysis.

An event-based analysis for evaluation of the overland wave propagation hazard is recommended. This is a consequence of requiring the WHAFIS computer program for computing the effects of inland propagation of waves for FIS studies. For these cases, different wave and water-level conditions are derived from a joint probability surface and are modeled to determine the combination that best represents the BFE, as opposed to the modeling of specific storms that is characteristic of the response-based approach. There are a number of possible approaches to generating the statistical wave and water-level condition that is used as input to WHAFIS and these are discussed in section D.3.6.3.

D.3.3.2.2 Response-Based Approach

With the advent of highly capable and economical computers, a preferred and more defensible approach that considers all (or most) of the complexity of the contributing processes has become practical; this is the *response-based approach*. In the response-based approach, one attempts to simulate the full complexity of the physical processes controlling flooding, and to derive flood statistics from the results (i.e., the local storm responses) of that complex simulation. For example, given knowledge of local storm climatology, one can simulate a large number of historical or hypothetical storms in such a way as to create an equivalent long period of record, from which the statistics of storm surge elevations could be derived. In a wave-dominated environment, if given the historical time history of offshore waves in terms of height, period, and direction, one might compute the wave runup responses for the entire time series, using all data and not pre-judging which waves in the record might be most important in terms of generating wave runup. Further, with knowledge of the erosion process, storm-by-storm erosion of the beach profile can also be considered, so its feedback effect on wave behavior, transformation over an irregular beach and wave runup on a beach or structure, can be taken into account.

At the end of this process, one would have developed a long-term simulated record of total water level at the site, which could then be analyzed to determine the 1-percent-annual-chance flood elevations. Clearly, successful application of such a response-based approach requires effort to characterize the individual component processes and their interrelationships, and the computational resource to carry out the calculations. However, those computational resources presently and routinely exist and they enable adoption of a response-based approach.

A response-based approach for the evaluation of water levels, wave runup and overtopping in the Great Lakes was developed and documented by Melby et al. (2012) and Nadal-Caraballo et al. (2012). This response-based approach is the recommended methodology for all Great Lakes coastal FIS studies. Key considerations in the implementation of a response-based approach for the Great Lakes are discussed in the following sections.

D.3.3.3 Storm Sampling Approach

The essence of a response-based approach is to consider a particular storm response at the study site for a set of extreme events over a period of years, and to perform an extreme value frequency analysis of the full set of storm-response maxima. Flood responses can be the still water level (SWL), the runup elevation, also referred to as the total water level (TWL), or some other response such as wave crest elevation.

A number of excellent texts have been written on analysis of extreme values with particular emphasis on environmental variables (e.g. Coles 2001, Haan 1977). For coastal applications a continuous times series is sampled to obtain a population of extreme values. Two types of samples can be produced for subsequent statistical analysis: Annual Maximum Series (AMS) or Partial Duration Series (PDS). The most common partial duration series is obtained by selecting all event peaks over a certain threshold, termed the Peaks-Over-Threshold method (POT). In this method, only independent, identically distributed peaks are selected in order to avoid counting multiple peaks for a single storm. The AMS method simply uses the maximum value for each year over the duration of the data. Both methods are commonly used but the POT has begun to dominate in recent years because the method considers all extremes while the AMS method discards significant storms if multiple occur in one year.

It is common for intense storms to be clustered over several years and for this clustering to repeat on a decadal scale. This can be associated with El Nino/La Nina or similar decadal-scale climatic cycles. Figures D.3.3-1 and D.3.3-2 from Melby et al. (2012) show the distribution in time of the twenty largest surge events for each of the NOAA gage sites in northern and southern Lake Michigan, respectively. The data clearly show years in which severe storm surge events are clustered; they also show years, sometimes a continuous decade or more, with no extreme events ranked in the top 20. The result of this storm clustering is that the AMS of storm responses will, in general, contain fewer of the most extreme events than will the PDS. Therefore, the AMS will be less accurate in predicting higher return period values, particularly if the return period is greater than the duration of the statistical population. For that reason, the PDS method is adopted in these guidelines for defining extreme values of storm responses.

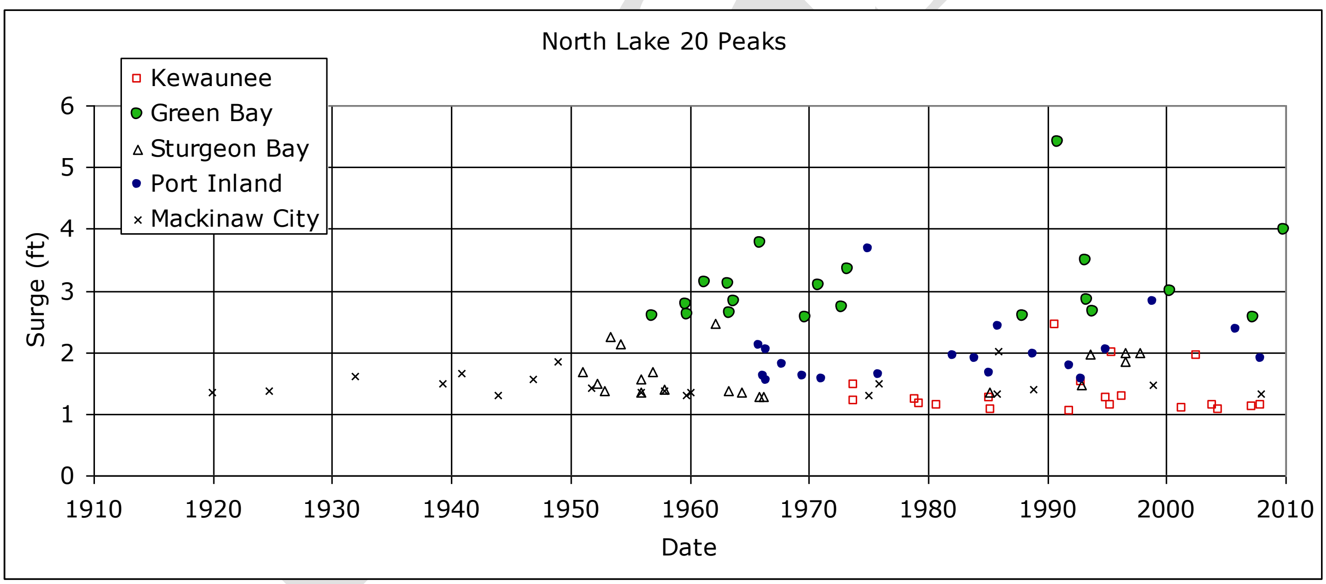


Figure D.3.3-1. Top 20 events ranked by storm surge for north lake water-level gages.

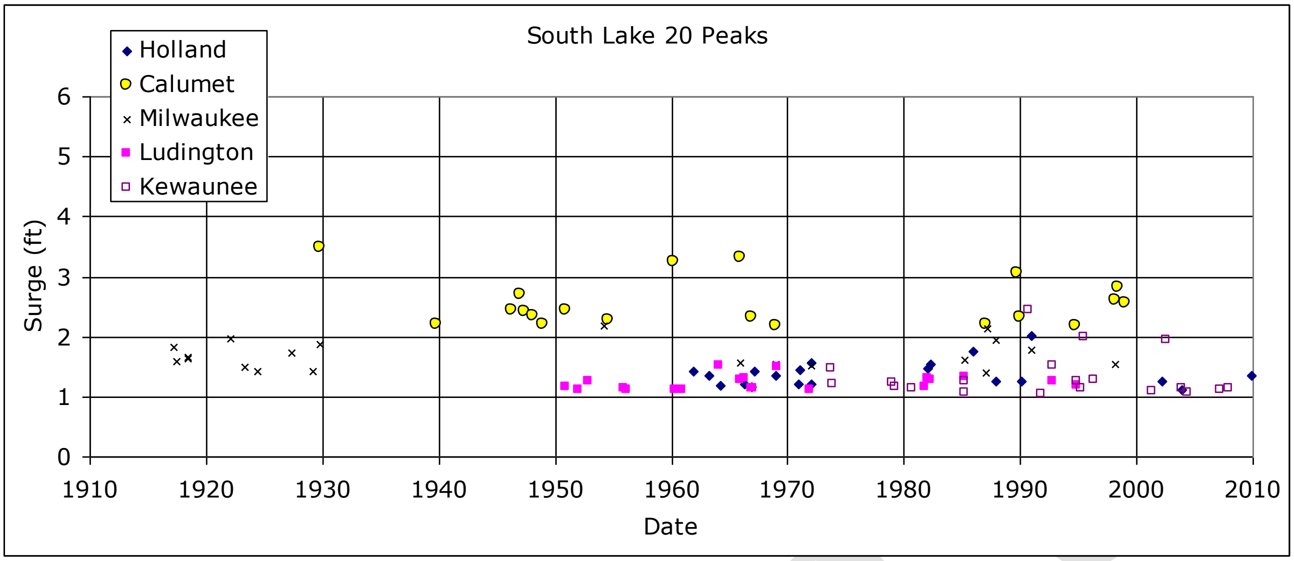


Figure D.3.3-2. Top 20 events ranked by storm surge for south lake water-level gages.

D.3.3.4 Record Length

Length of record for storm responses, surge or water level for example, dictates the annual exceedance probability of each, and therefore the BFE. In the POT method, selecting a higher lamda (λ) value (i.e., a value that corresponds to the average number of storms per year) gives more storms from a given record length; however, this may have relatively little impact on the BFE, because the BFE is controlled mostly by events in the tail of the probability distribution. On the other hand, a longer historical record has significant impact on the accuracy of the extremal analysis.

At some point, the record length becomes sufficiently long so that additional years do not change the overall shape. Increasing record length will continue to move the distribution down and toward the right in the figure. In most cases, a shorter record length will yield a more conservative estimate of the BFE than a longer record length. For short record lengths, the extremal distribution is too steep and crosses the more accurate distributions that are based on longer record lengths. However, if the record length is too short, then the extremal distribution will be too steep, possibly resulting in a significant over-prediction of the BFE and the 0.2-percent-annual-chance elevations. The Mapping Partner should evaluate the available measured water-level data to determine the appropriate minimum record length for each lake (e.g. Melby et al., 2012).

D.3.3.5 Storm Selection

In the implementation of the high-resolution 2-D hydrodynamic and wave modeling (Section D.3.4) a set of significant storm events must be selected in order to construct accurate extremal distributions of total water-level and wave events. Modeling every historical storm event that occurred for a given time period of sufficient length is simply not feasible due to time, computational and funding constraints. Instead, it is recommended that historical events be screened and sampled in order to select the minimum number of events required to accurately compute extremal water-level distributions throughout a lake.

A storm sampling approach for computing water-level probabilities in the Great Lakes was first developed and recommended by Melby et al. (2012) based on an analysis of 27 years of concurrent measured water-level data and hindcast wave data for Lake Michigan. The approach was validated and confirmed by Nadal-Caraballo et al. (2012) for the 50-year period, 1960-2010. The storm sampling method recognizes that total water level along the coast is typically comprised of lake level, storm surge and wave-induced components. Wave runup can dominate for steep shorelines or structures, or in deeper lakes where storm surge generation is limited. If significant shore-fast ice is present, then wave-induced storm responses will not occur. Thus selecting storms is not as simple as selecting the highest storm surge or wave runup values. The analysis must properly weight the influence of surge, waves, and ice in order to rank and select the storms.

It is important to note that a storm event that produces extreme surge and/or wave conditions at one location on the lake shoreline does not produce extreme conditions everywhere else in the lake. Melby et al. (2012) showed that for Lake Michigan there was no significant statistical correlation between water levels measured at one NOAA gage site and a second adjacent gage site. Thus selection of a storm set that accurately describes extremal water-level statistics everywhere within the lake must be done judiciously in order to minimize the storm sample size but accurately reflect the extremal water-level distributions everywhere along the lake shoreline. Selection of an appropriate but smaller composite storm sample can be achieved through analysis and comparison of water-level probability distributions derived from various composite storm sets and a much larger full storm set.

It should also be noted that a surge event at one end of a lake may actually draw down the water level at the other end of the lake. An example of this phenomenon is shown in Figures D.3.3-3a and D.3.3-3b during a major storm that occurred in February, 1987 on Lake Huron. Figure D3.3-3a shows the measured and modeled water levels from Lakeport, which is in the southern end of the lake, and displays the significant surge event. Figure D.3.3-3b shows the water-level measurements and modeling results from Mackinaw City at the northern end of the lake, in which the lake level was clearly drawn down during the height of the storm. During this time period, the small bump in the water-level record at Mackinaw City that preceded the storm would be tagged as the ‘peak surge’ at this location for this event. However, this bump is not even picked up in a POT analysis of the entire water-level record from Mackinaw City. Nevertheless, this ‘anti-storm’ becomes included in the composite storm record for the entire lake, and as such populates the high-frequency segment of the return period curve for Mackinaw City with artificially low values. This issue is particularly germane at each end of an elongated lake, but less so in the middle (nodal) region of such a lake.

The ramification of this issue associated with the composite storm data set is that in curve-fitting a GPD, the lower end (high-frequency) segment of the return period data must be avoided so as not to contaminate the fit at the upper extremes. The Q-Q optimization approach of Nadal-Caraballo, et al. (2012), which tunes the curve fit to the interquartile range of the composite storm set, accomplishes this goal to a significant degree.

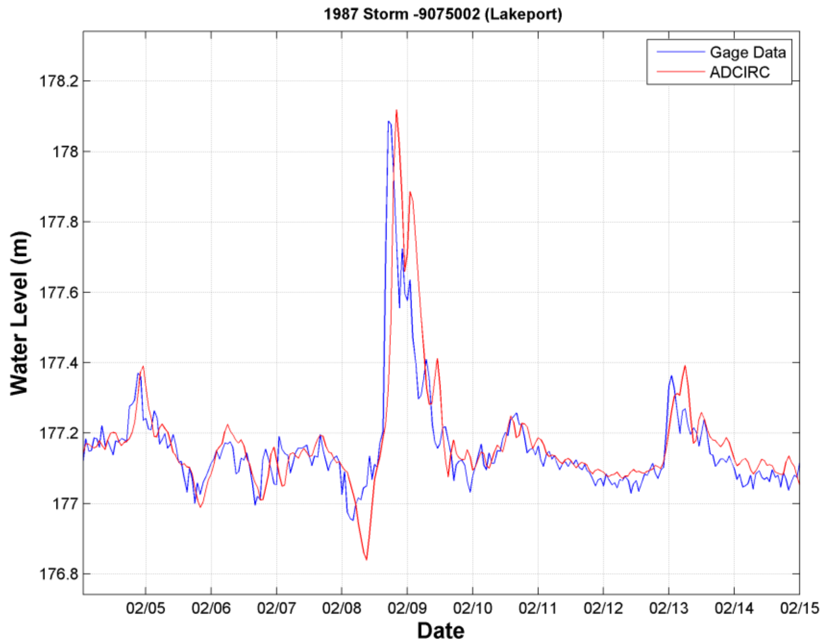


Figure D.3.3-3a– Observed water-level data (and ADCIRC results) at Lakeport (southern end of Lake Huron) for the February 1987 storm.

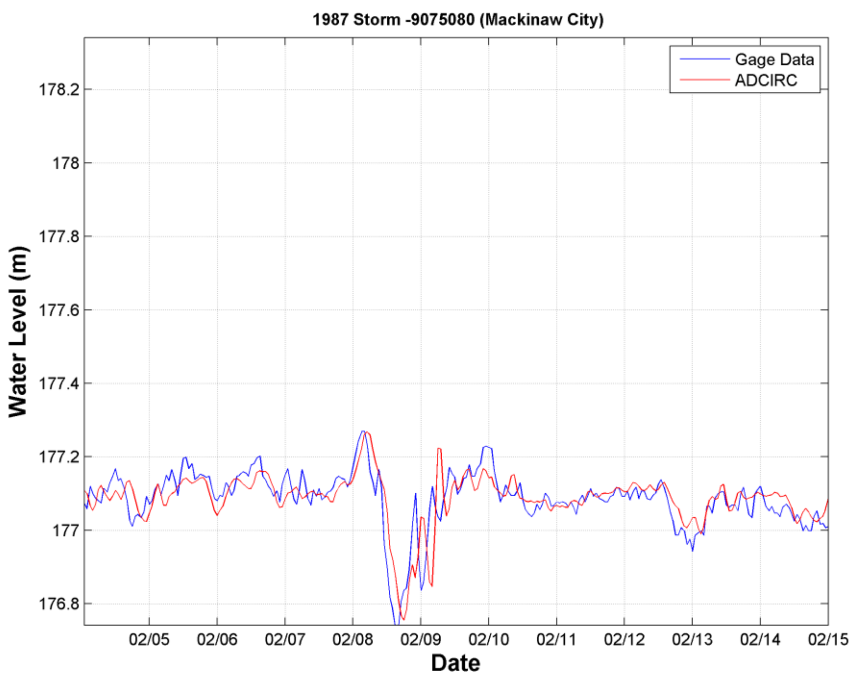


Figure D.3.3-3b – Observed water-level data (and ADCIRC results) at Mackinaw City (northern end of Lake Huron) for the February 1987 storm, showing 'anti-storm' behavior in water level.

When screening and sampling events, storm surge can be computed using long-term NOAA measured water levels. Wave conditions for events can be estimated from a wind/wave-surrogate analysis and from hindcast wave data). Wind-wave surrogate analysis refers to the use of simple methods to estimate wave conditions associated with recorded wind events. Nadal-Caraballo et al., 2012 describes in great detail the process for developing and evaluating the composite storm set for Lake Michigan. An event that produces a large storm surge is not always an event that also produces high wave energy and the coincidence of elevation water levels and high wave conditions will vary by lake and location around a given lake. The Mapping Partner should conduct a lake-specific analysis to determine the appropriate ratio of water level to wave events within the composite storm set to adequately represent the storm conditions that cause extreme flood hazard responses.

The following steps for storm selection are recommended. The objective in the sampling procedure is to select the most significant events for waves and water levels. Although the technical details are important in arriving at the final storm selection, it is important to keep this objective in mind.

1. Determine the number of storms necessary for the composite storm set to adequately represent storm conditions and responses throughout the lake (See Nadal-Caraballo, 2012).

2. Identify storms having the highest peak storm surge. This would typically be done using NOAA water-level measurements from all available sites and over the full record length. Rank storms based on magnitude of peak surge at each site.

3. Identify storms having highest peak wave height. This would typically be done using measured data, hindcasts, or surrogate wave calculations at spatially distributed sites. Rank storms based on magnitude of peak significant wave height.

4. Select a sufficient number of the highest ranked surge and wave events at each location such that the total number of storms is greater than the previously determined composite storm set size. This sample will be further reduced through screening to achieve the target composite storm set size.

5. The screening process first eliminates duplicate storms. If storms are duplicates, reject duplicate event (or events) with the lowest site-specific rank and include the next largest event at that site.

6. Balance the number of storms selected for each site to maximize consistency in geographical and temporal coverage of selected storms; define an initial set with a sufficient number of storms as previously determined with an appropriate ratio of wave height dominated events and storm surge events. For Lake Michigan, the appropriate ratio was determined to be roughly 50 percent based on maximum wave height, and 50 percent based on peak storm surge.

7. Ice screening may then be required if ice processes prevented flooding for the selected events. Data and basic process knowledge of ice cover influences on flooding are limited. Therefore, the ice screening must be done with caution. Using regional ice maps nearest the time of each storm, determine if shore fast ice has the potential to block waves. If a storm in the initial set is a low-ranked surge event and waves are blocked due to ice coverage, consider it as a candidate to remove.

D.3.3.6 Storm Sampling Across Long-Term Lake Levels

A critical issue in developing a storm sampling approach is whether the events that end up being sampled as part of the composite storm set are representative of the entire record length, in terms of the distribution of mean lake levels. In the case of the Great Lakes, a key question is whether or not the sampling is actually being done properly across both high and low lake levels. If storm sampling is done solely on high lake levels, for example, this would result in ill-shaped exceedance distributions and could bias the 1-percent and 0.2-percent-annual-chance flood elevations. In order to avoid introducing bias into BFE probabilities it is important that in the storm sampling, the lake levels associated with the storms in the composite storm set properly reflect the full distribution of lake levels.

This issue of sampling across lake levels was assessed for Lake Michigan by Nadal-Caraballo et al. (2012) through a re-sampling analysis. Different re-sampling methods were utilized, and it was determined that the Nearest Neighbor Re-sampling (NNR) method provided the optimal results. The observations used in this analysis were the monthly maximum lakewide levels (from BOC still water-level data at NOAA gage sites) associated with each of the sampled storms. These monthly maximum levels were used because they account for both lake level and storm surge elevations. Statistics were computed to determine the minimum number of storms that needed to be sampled per NOAA gage site in Lake Michigan in order to assure adequate sampling across all lake levels; computed statistics included: variance, standard deviation, and model error tests using root mean square deviation. The statistics are computed from all the monthly maximum lake-level values for the entire required period of record. From the Lake Michigan re-sampling analysis it was concluded that at least 15 storms per NOAA water-level gage site (or 135 total storms) are necessary for adequate emulation of the long-term water-level signal and its distribution. This was in agreement with the storm sample size arrived at based on other analyses, which suggested a composite storm sample of 150 storms was necessary.

The distribution of lake levels that corresponds to all storms contained in the 150-storm composite storm set for Lake Michigan is shown in Figure D.3.3-4 for Ludington, MI. Also illustrated in the figure are the long-term maximum monthly water levels (red lines) and the NNR fits (blue lines). The figure shows how well all three high water-level periods (i.e., mid 1970’s, mid 1980’s, and late 1990’s) and all low water periods (mid 1960’s, late 1970’s, early 1990’s and 2000’s) are represented by the sampled storms. It can be seen that the 150-storm set not only captures the decadal variation in lake levels but also much of the higher frequency variation in lakewide water levels.

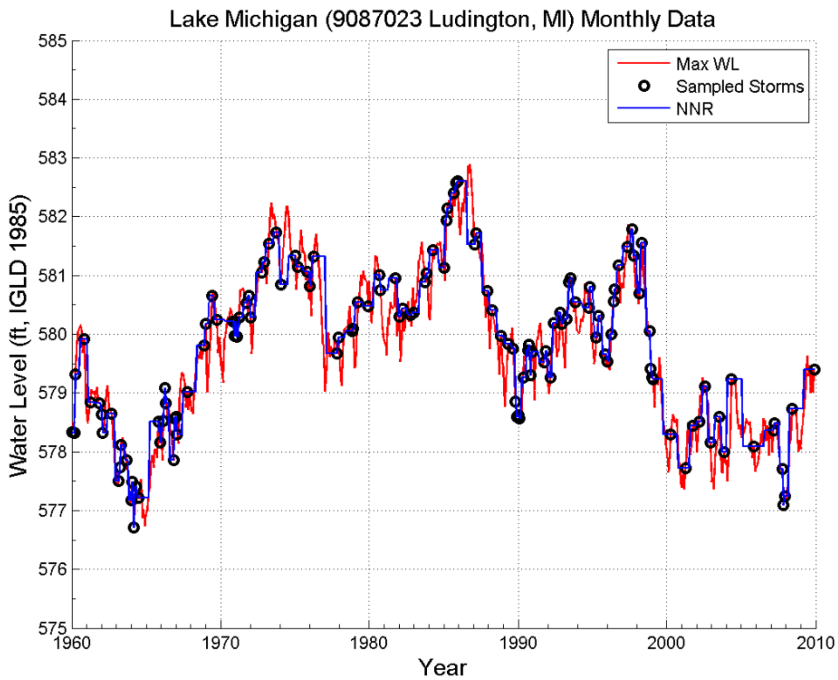


Figure D.3.3-4. Distribution of storms in the 150-storm Composite Storm Set across lake levels for Ludington, MI.

D.3.3.7 Estimating Extremal Response Probabilities

Extreme value theory suggests that the Partial Duration Series (PDS) determined from the Peaks-Over-Threshold (POT) method should conform to the Generalized Pareto Distribution (GPD). Therefore the GPD is adopted in these guidelines for deriving probability distributions for storm responses such as still water level or total water level (which includes wave runup). Melby et al. (2012) and Nadal-Caraballo et al. (2012) describe application of the POT/GPD method to Great Lakes storm responses in detail. Nadal-Caraballo et al. also describes the adopted method for maximizing the fit of a GPD distribution to a set of storm responses. The goodness of fit is evaluated using both the cumulative distribution function (CDF) and a quantile-quantile (Q-Q) analysis of storm responses.

Figures D.3.3.-5 through D.3.3-7 illustrate application of the POT/GPD response-based statistical method to measured still water level for Ludington, MI, using the 150-storm Lake Michigan composite storm set. In Figure D.3.3-5, the Q-Q plots display the quantiles of the modeled water level versus the theoretical quantiles of the GPD that was fit to the model results. Superimposed in the Q-Q plots are a *robust linear fit* of the Q-Q data and a 45-degree line to help evaluate linearity. The "robust linear fit" is deemed so because it is actually a line that joins the first quartile (Q1) and the third quartile (Q3) of both the water-level data and the GPD fit. The segment between Q1 and Q3 is known as the interquartile range (IQR = Q3 – Q1) and it is defined as a robust order statistic. In theory, the closer the robust QQ linear fit (red dashed line shown in Q-Q plots) is to the 45-degree line, the better the GPD fit.

A metric can be established to identify, through iteration, the lower-bound threshold, RP_{th}, that minimizes the difference between the slopes of both lines. Optimization of this metric is then used to determine the optimal GPD fit. This optimization process effectively reduces the value of lambda (average number of storms per year) used for the GPD fitting. Details of the GPD fitting process are described by Nadal-Caraballo et al. (2012).

Figure D.3.3-6 shows the non-exceedance probability distribution, i.e., the CDF, for the optimized fit. Figure D.3.3-7 shows a return period plot for still water level based on the optimally fit GPD distribution as well as the raw response data. This example illustrates application of the response-based storm sampling and statistical analysis approach using the POT/GPD methods to compute probabilities for storm responses.

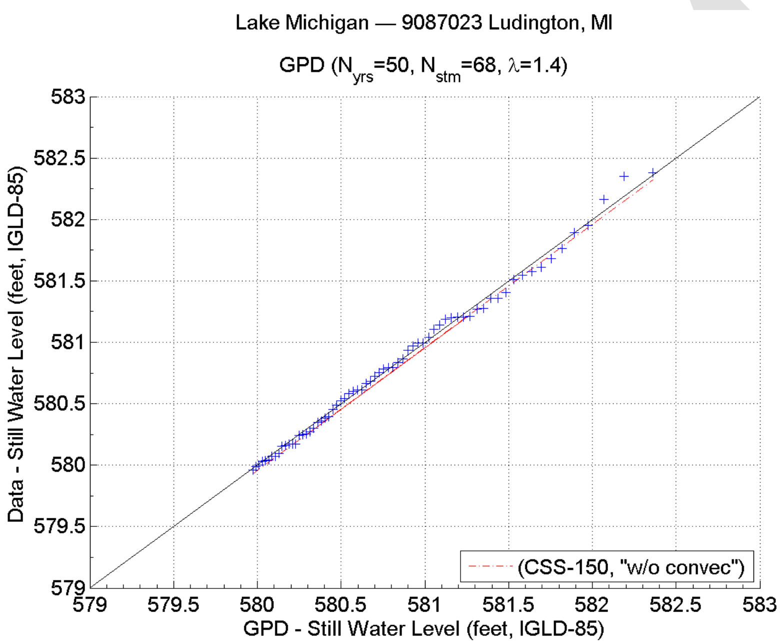


Figure D.3.3-5. SWL Q-Q Plot from Composite Storm Set (w/o convective storms) for Ludington, MI.

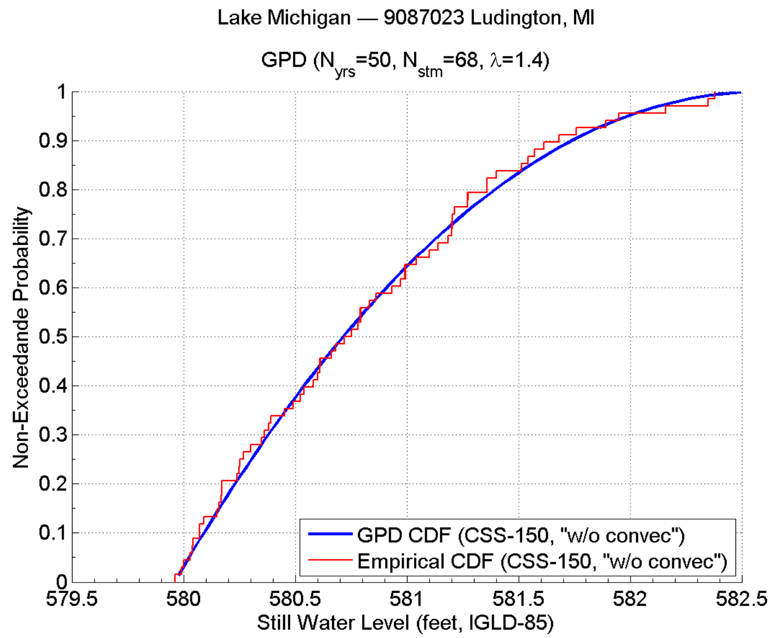


Figure D.3.3-6. SWL CDF Plot from the Composite Storm Set (w/o convective storms) for Ludington, MI.

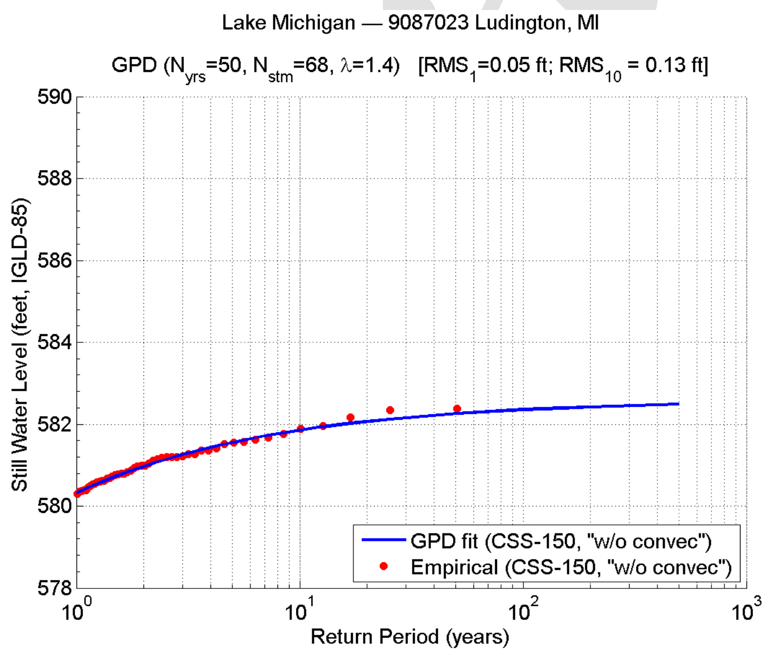


Figure D.3.3-7. SWL Return Period Plot from the Composite Storm Set (w/o convective storms) for Ludington, MI.

D.3.4 Water Levels and Waves

This section provides guidance for two study components: the determination of offshore waves and their transformation into the surf zone, and nearshore water levels. Guidance on special considerations for the presence of ice is provided for both of these components.

D.3.4.1 Water Levels

Storm surge is the rise of the ocean surface that occurs in response to barometric pressure variations (the inverse barometer effect) and to the stress of the wind acting over the water surface (the wind setup component). Wave setup is excluded by this definition and must be taken into account separately as discussed in Subsection D.3.5.2.

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

Some of the factors that must be considered in selection and application of a model are enumerated below. Specific guidance regarding each factor is not given here. Instead, guidance for complex 2-D modeling is best obtained from the user's manual for a particular model, and from review of prior studies which have successfully used that model.

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

- The governing equations of the model, typically the nonlinear long wave equations accounting for conservation of mass and momentum, with surface wind and barometric pressure terms representing the influence of the storm
- The numerical scheme used by the model, whether finite differences computed on a grid of rectangular cells (commonly of fixed size) or in curvilinear coordinates, or finite elements represented by triangular or quadrilateral cells (of varying sizes). The numerical scheme may also be explicit or implicit, affecting time step constraints, and so affecting study cost
- The flooding / drying treatment of cells as the surge and tides advance onto land and then recedes
- The storm representation, from large-scale synoptic-scale storm events (the types of events that are being considered in flood hazard mapping), meso-scale systems like frontal boundaries to micro-scale systems synonymous with the development of thunderstorm cells; the storm representation will be quite different although the modeling

principles remain the same in each case; on-land filling will be significant for sheltered waters; winds and pressure representations must be appropriate 10 meter elevation, averaged winds

- The wind stress coefficient which relates the windspeed at the surface to the stress felt by the fluid; consideration must be given to the possibility that the wind stress is capped under the most extreme conditions
- The sheltering treatment, adjusting the effective wind stress to account for partial reduction by tall vegetation, terrain, and structures (especially significant for sheltered waters)
- The offshore bottom friction treatment over the relatively smooth ocean or bay bottom, which retards the flow
- The onshore flow resistance treatment accounting for bottom friction and resistance offered by tall vegetation and structures; critical for sheltered waters
- The source and quality of bathymetric data, defining the varying depths at the site
- The source and quality of topographic data, such as traditional quad sheets or newer LIDAR data
- The manner in which normal storm erosion alters the topography used in the model
- The manner in which catastrophic erosion might affect the modeling assumptions, in the event of loss of a major barrier to inland flooding
- The representation of the bathymetry and topography in the model grid system, which depends upon the numerical scheme
- The faithfulness of the grid to the irregular bathymetry and terrain, including conformance to boundary shapes and inclusion of small sub-grid barriers which may control the local variation of overland flow
- The resolution of the grid, whether fixed or varying through the study area
- The boundary conditions which impose approximate rules along the edges of the model area, both offshore and onshore, permitting termination of the calculations at the expense of accuracy
- The types and limits of calibration which might be done,
- The role of verification hindcasts to confirm the apparent reasonableness of the final model when compared with historical surge records
- The role of wave setup (a separate topic in these guidelines), especially in the interpretation of high-water marks used for hindcast verification

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

D.3.4.1.1 Scales of Water-Level Variability

The water surface elevation called the still water level (SWL), or the still water elevation (SWEL), is the water level upon which storm-generated waves "ride." SWL consists of multiple components including the long-term mean lake level and its fluctuations, seasonal variation of the mean lake level, antecedent seiche conditions at the time of a storm occurrence, and short-term storm-driven water-level variations (see Section 3.1.1). These three sources of water-level variability, at different time scales, are all reflected in the available gage data, and all must be accounted for in a flood study. In the response-based approach adopted in these guidelines, the seasonal and long-term fluctuations are treated appropriately by simulating storms at their synoptic mean lake levels. This approach requires that the lake levels associated with the storms represent the present-day distribution of possible lake levels.

In general, FIS are intended to be based on existing conditions. Strictly speaking, then, a study could ignore long term variability, adopting the current mean annual level for the analysis. However, the data shows that significant variability can occur over a period of just a few years, and it is recognized that both flood maps and new construction have lifetimes during which such variations may be significant.

D.3.4.1.2 Measured Water-Level Data

Measured water levels are an important data source for coastal flood analyses. These data are essential for characterizing long-term and seasonal-scale lake level changes, for characterizing storm event-scale water-level changes, for use in storm surge model validation, for identifying storms and developing a composite storm set for use in detailed modeling, and for developing and validating the storm sampling and statistical analysis methods that are adopted for a lake. The very long water-level records that are available at some locations provide excellent data sources with which to validate estimates of the 1-percent-annual-chance stillwater levels that are calculated using the adopted statistical analysis approach; and they should be used for that purpose whenever possible. Melby et al. (2012) and Nadal-Caraballo et al. (2012) further describe use of measured water-level data in validating the adopted approaches for storm sampling and statistical analysis of flood elevations.

For the Great Lakes, over sixty water-level gaging stations are in operation and each report levels hourly. Some gages have been in existence for very long periods of some. Hourly data acquired since the 1970s are readily available; hourly data acquired prior to 1970 less so. Monthly maxima and monthly mean data are readily available and are very useful for evaluating long-term trends in lake levels. The utility of the monthly maxima and monthly means for estimating historical storm surges has varied from lake to lake (Melby et al., 2012 and Baird, 2012). These data are available from the NOAA and USACE. NOAA provides access to its data through the CO-OPS National Water Level Observation Network (NWLON) database. The USACE, Detroit District, provides water-level data on its web site. Other sources of water-level data may also be available in particular locations, and these should be sought by the Mapping Partner as part of the study scoping effort.

D.3.4.1.3 **2-D Lakewide Storm Surge Modeling**

This section describes the process to develop and apply a storm surge model for estimating event-scale water-level changes in the Great Lakes. Modeling storm surge for the entire lake provides a means for developing consistent, high quality and highly-defensible storm surge information for use in local county-scale mapping, taking advantage of economies-of-scale in doing so.

The Great Lakes have complex shapes and bathymetric characteristics. Some lakes have a series of interconnected bays of different sizes and shapes. All are characterized by highly irregular coastlines. Some lakes are characterized by the presence of multiple islands, and by multiple small bays and harbors that are situated along the coast and which have constricted connections with the larger lake. The lakes differ in their water depths, and in the size and shape of deep and shallow water regions, which also influences the generation and evolution of storm surge within each lake. Modeling of the storm surge within the entire lake complex, as a single system including connectivity between different water bodies, enables more accurate treatment of the complex hydrodynamic interactions that occur in response to meteorological forcing. Modeling the system as a whole eliminates the need to specify approximate boundary conditions at open-water boundaries (which can be problematic), that might otherwise be needed to model the interconnected water bodies separately.

Storm surge models must solve the 2-D depth-averaged, shallow water, long-wave equations. The Mapping Partner should seek FEMA approval before finalizing selection of the particular model to be used.

The general approach to storm surge modeling consists of the following steps:

1. Developing the bathymetric dataset and model grid mesh for the lake system;
2. Assembling input files for atmospheric forcing (wind and pressure fields) and surface ice fields;
3. Testing and refining the initial model setup;
4. Validating the model for a number of historical extreme storm events using objective measures of predictive skill; and
5. Assessing model sensitivity to various factors and adjustable parameters such as bottom friction and presence/treatment of ice.

In the implementation of these guidelines, storm surge is simulated for all historic storms that are contained in the composite storm set. All storms are simulated using the mean lake level that existed at the time of each storm, as the initial water-level condition. This is done to properly treat the effect of varying lake level in the statistical approach that was adopted for developing Great Lakes BFEs.

D.3.4.1.3.1 **Grid and Mesh Development**

Development of a surge model grid or mesh that accurately characterizes the irregular shape and variability in water depth throughout a lake is an extremely important step, in order to properly simulate wind and pressure induced water-level changes and seiche motions within a lake. Model grid meshes that best resolve and represent the physical characteristics of the lake (shoreline irregularities and topography/bathymetry variations) will result in the best predictions of storm surge.

The NOAA Electronic Navigation Charts (ENC), together with NOAA-published 3 and 9 arc-sec bathymetry data files from NOAA's National Environmental Satellite Data and Information Service (NESDIS) digital bathymetry data base can be used to facilitate development of the storm surge model grid mesh and subsequent specification of bathymetry for all nodes of the grid mesh. Bathymetry data are usually processed to a consistent IGLD 1985 vertical water-level datum, which can serve as a consistent vertical reference for all bathymetry, topography, lake level and storm-induced water-level data, or data can be converted to the mapping vertical datum, NAVD88. In addition, NOAA's IGLD 1985 zero-depth coastline file can be incorporated into the bathymetry data set to facilitate accurate specification of the irregular lake shoreline during development of the grid mesh.

In floodplain areas that can become inundated during severe storms and which are to be included in the storm surge grid mesh, topography data such as those obtained from LIDAR surveys can be used in the mesh development process. Grid mesh development should carefully consider and resolve all significant features that either act to retard storm surge penetration into the floodplain (elevated roadways, levees or other natural landscape features), or that might facilitate its movement into backshore areas (small rivers and streams, navigation channels, underpasses, drainage canals, etc.). Grid mesh development can be aided through use of geo-rectified photography and images to aid in establishing and resolving the shoreline or landscape features that influence surge propagation.

After generating an initial grid mesh, the mesh builder must perform the important task of checking and refining the grid, optimizing agreement between the grid and the shoreline and coastal features such as breakwaters and jetties or other landscape and infrastructure features in the floodplain. Arc spacing between grid vertices in high-resolution regional-scale lakewide storm surge modeling generally varies from 30 m, specified for the shoreline in critical areas to be mapped, in small bays and harbors and connections between small bays and harbors and the larger lake, and to resolve small but important landscape features, to spacing between grid nodes of thousands of meters in deeper offshore regions of the grid mesh.

Land cover type data bases, such as those developed by the USGS or NOAA, can be used to specify the friction resistance characteristics of different portions of the model grid domain. This can be done in order to maximize the accuracy with which landscapes of various types influence the propagation of the storm surge into an inundated floodplain. Frictional effects of the landscape can be important if the inundated flood plain is large in extent or if the storm surge wave must propagate a significant distance to reach a particular location.

D.3.4.1.3.2 **Wind and Atmospheric Pressure Forcing**

Storm events in the Great Lakes can vary from large-scale synoptic-scale storm events (the types of events that are being considered in flood hazard mapping), meso-scale systems like frontal boundaries to micro-scale systems synonymous with the development of thunderstorm cells. If the meteorology of these events can be accurately quantified, the associated impact of the surge and waves on a coastal reach also can be quantified. Jensen et al. (2012) describe in detail the development of storm wind and atmospheric pressure fields for use in storm surge and wave modeling, as applied to support flood hazard mapping in Lake Michigan.

The NOAA NCEP Climate Forecast System Reanalysis (CFSR) wind and pressure fields (Saha, et al., 2010), variable on a 0.5-deg longitude/latitude grid with global coverage, with meteorological variables (wind speed and direction, surface barometric pressure fields) provided at hourly time intervals, are one of the preferred and recommended sources of meteorological input to the wave and storm surge modeling. These latter data are available for all storms contained within the reanalysis period (1979 to 2009). The Natural Neighbor Method developed by NOAA GLERL, Schwab (1978 and 1989) and Schwab et al. (1984 and 1998), can be used in developing wind and pressure fields for storms prior to 1979. In the future, climatological data should be evaluated in a manner similar to Jensen et al. (2012) and the best available source adopted.

The quality of wind and atmospheric pressure field input is of the utmost importance in storm surge modeling, in light of the strong nonlinear dependence of surface wind stress on wind speed, and the importance of atmospheric pressure induced water-level changes in the Great Lakes. The quality of water-level predictions is only as good as the quality of the meteorological forcing. Both wind speed and directional accuracy are important in these irregularly-shaped, sometimes elongated lakes, where wind fetch is highly sensitive to wind direction.

Surge modeling must be able to utilize time-varying wind and pressure fields from the CFSR data base, or fields developed using the NNM, in order to properly simulate the water-level response to rapidly changing meteorological forcing. The frequency of data needed to develop these fields limits the ability to capture certain storm events that very quickly traverse the lake, such as squall lines and fat-moving fronts. The spatial and temporal resolution of wind and pressure fields derived using either the NNM or the CFSR database can only really represent well the forcing associated with large-scale non-convective storm systems, not smaller rapidly moving frontal passages associated with thunderstorms or other convective events.

In these guidelines only the flood hazard associated with large non-convective systems is considered. For Lake Michigan, large non-convective storms were found to be the most important source of high waves and high water levels that dictate the flood hazard. Melby et al. (2012) and Nadal-Caraballo et al. (2012) confirmed that neglecting convective storms in determination of BFEs was reasonable for Lake Michigan. This same type of analysis should be performed for each lake to examine the suitability of this assumption.

D.3.4.1.3.3 **Ice Cover**

Storm surge modeling also requires ice fields as input. Using ice fields developed from the ice data produced by NOAA GLERL, the effective wind stress applied to the water surface in the surge modeling can be influenced by the concentration of the ice and by the horizontal extent of ice cover.

An additional physical process that has been examined in ice-covered regions such as the Great Lakes is the influence of sea ice as a source of aerodynamic roughness. Many storm surge models use the wind drag coefficient formulation of Garratt (1977) in the calculation of surface wind stresses. This is a widely-used formulation and it has been found to work well for storm surge applications. Macklin (1983) and Pease et al. (1983) found that measurements over first year sea ice typically yielded wind drag coefficient values that were significantly larger, and varied less with wind speed, than that those predicted for open water. More recent work (Birnbaum and Lupkes, 2002, and Garbrecht et al., 2002) formalized the effect of form drag associated with ice on the specification of wind drag coefficients within marginal ice zones. From their work, Chapman et al. (2005 and 2009) utilized an empirical fit to the range of field data for the air-ice-water effective wind drag coefficient, C_{DF}, and suggested:

$$C_{DF} = [0.125 + 0.5\, IC\,(1.0 - IC)]\, 10^{-3} \qquad \text{(D.3.4-12)}$$

in which IC is the ice concentration varying from 0.0 to 1.0 (corresponding to 0 percent to 100 percent) for open water and complete ice cover conditions, respectively. Inspection of this air-ice-water-wind drag coefficient formula shows that a maximum value of 0.0025 occurs with 50-percent ice coverage. This value is very close to the Macklin (1983) measurement of 0.0028 for first year ice. Furthermore, it is seen that the value of the drag coefficient is symmetrical at about 50-percent ice coverage suggesting that the drag coefficient needed to represent 75-percent ice coverage is close to that of 25-percent ice coverage. An alternative linear fit dependence on ice concentration has been applied by Danard et al. (1989). These notions regarding variation of wind drag coefficient with ice cover have been supported by a number of Chukchi and Beaufort Sea storm surge simulations (Henry and Heaps, 1976; Kowalik, 1984; and Schafer, 1966) in which, wind drag coefficients greater than or equal to 0.0025 where utilized. The interactions of wind, ice, and the water column are not well understood, however. Testing and validation of the approach for treating ice cover in the modeling is recommended where possible.

If ice cover is present and the increased drag coefficient, calculated with equation D.3.4-12, exceeds the value calculated using the standard Garratt (1997) formulation, it is recommended to replace the standard Garratt wind drag coefficient with the increased value associated with the presence of ice cover.

D.3.4.1.3.4 **Model Validation and Skill Assessment**

Validation of the water-level modeling approach is critical to the success of the mapping project, to defensibility of the technical approaches that are taken, and ultimately to acceptance of mapping results. Comprehensive model validation shall be performed.

Various parts of a lake responds differently to any one particular storm, and the storm that produces extreme water levels in one part of the lake might not, and probably does not, produce extreme levels in other parts. Therefore the number of validation storms must be large enough to assess model prediction skill along all parts of the lake shoreline that are to be mapped. Measured water-level data are used to perform the validation, through comparisons between measured and modeled water levels. To the extent possible, the treatment of ice should be validated by appropriate selection of validation storms.

In assessing model predictive skill, objective measures, or metrics, should be used to perform the comparisons between measured wave parameters and simulated parameters. The following model skill metrics should be examined: bias, standard deviation of error, root mean square error, scatter index, summary performance score, regression analysis, providing the slope and intercept, and correlation for the significant wave height (H_{mo}), peak wave period (T_p) and mean wave periods (T_m). In the following, rms refers to root mean square, *p* = predicted, *m* = measured, and *n* is number of data points.

Dimensional RMS of Measurements: $m_{rms} = \sqrt{\frac{1}{n}\sum_{i=1}^{n} m_i^2}$ (D.3.4-1)

Dimensional RMS Error: $E_{rms} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(p_i - m_i)^2}$ (D.3.4-2)

Non-dimensional RMS Error: $e_{rms} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(\frac{p_i}{m_i} - 1\right)^2}$ (D.3.4-3)

Bias: $B = \frac{1}{n}\sum_{i=1}^{n}(p_i - m_i)$ (D.3.4-4)

Standard Deviation of Errors: $\sigma_d = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(p_i - m_i - b)^2}$ (D.3.4-5)

Mean of Measurements: $\bar{m} = \frac{1}{n}\sum_{i=1}^{n} m_i$ (D.3.4-6)

Scatter Index : $SI = \frac{\sigma_d}{\bar{m}}$ (D.3.4-7)

Normalized RMS Error Performance: $\hat{E}_{rms} = \left(1 - \frac{E_{rms}}{m_{rms}}\right)$ (D.3.4-8)

Normalized Bias Error Performance: $$\hat{b} = \left(1 - \frac{|B|}{m_{rms}}\right) \qquad \text{(D.3.4-9)}$$

Normalized SI Performance: $$\widehat{SI} = (1 - SI) \qquad \text{(D.3.4-10)}$$

Summary Performance Score: $$P_s = \frac{\hat{E}_{rms} + \hat{b} + \widehat{SI}}{3} \qquad \text{(D.3.4-11)}$$

In addition to the aforementioned metrics, Q-Q plots can be developed to examine model performance over the full distribution of wave conditions. Peak-to-peak comparisons should also be made to quantify model skill in predicting the maximum wave conditions during severe events.

D.3.4.2 Waves

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

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

D.3.4.2.1 Lakewide Wave Modeling

Wave fields generated by the moving Great Lakes storm systems can be quite complex, exhibiting considerable spatial and temporal variation in wave conditions around the periphery of each lake. Use of a two-dimensional, time-dependent, spectral wave model is recommended to develop a consistent set of high quality information with which to characterize the incident waves. If sufficient resolution is adopted, this same class of modeling can also provide storm wave information for the shoaling and portions of the surf zone. The selected wave model must be able to treat the following processes: 1) quantification of the temporal and spatial variation of the two-dimensional wave spectra, 2) complete source term specification of the atmospheric wind input, nonlinear wave-wave interactions, wave dissipation in the form of white-capping, 3) shallow water mechanisms including, refraction, shoaling, wave-bottom effects and depth-induced wave breaking, and 4) time and spatial varying specification of wind and ice fields. Jensen et al. (2012) provide a detailed description of the application of lakewide wave modeling to Lake Michigan, using the WAM Cycle 4.5.1C model. WAM or any other well-tested, validated and approved model of this class of models (such as WAVEWATCH III or SWAN) can be used. The Mapping Partner should seek FEMA approval before finalizing selection of model to use.

D.3.4.2.1.1 **Grid Mesh Development**

Wave model grid meshes can be generated using the NOAA GLERL digital bathymetry data base. The resolution of these bathymetry data sets is 3 arc seconds (about 90 m). LIDAR data or some other source of bathymetry data, such as NOAA digital electronic navigation charts, can be used to supplement these data in characterizing water depths in the shoaling zone. LIDAR data or data from other beach surveys can be used to characterize the surf zone if the desire is to resolve the surf zone with this regional-scale modeling. Grid resolution must be sufficient to properly simulate the wave transformation processes, particularly refraction. The resolution required will depend on the water depth of the locations where wave information is saved for input to other facets of the coastal process analysis, and on the irregularity of bathymetric contours seaward of this location. Wave model resolution to properly treat refraction is usually in the range from 30 to 500 meters or more, for the shoaling zone; it is dependent upon the degree of contour irregularity. Lower resolution can be used in deep water and in areas with slowly varying contours; higher resolution is needed where the changes in depth are more irregular and complex and where beach slopes are greater. Resolution required to treat wave transformation and breaking over barred beaches is on the order of meters. High computational requirements will generally preclude resolving the surf zone at sufficiently high resolution in the context of lake-scale modeling.

D.3.4.2.1.2 **Frequency and Direction Resolution**

Lakewide wave modeling also must adopt sufficient resolution of the frequency and directional energy spectrum, to properly resolve shallow water transformation processes. Each of the Great Lakes can be considered an enclosed body of water. Storm waves generated in the Great Lakes have, for the most part, shorter-periods compared to those along the Atlantic and Pacific Oceans. Therefore, the active frequency domain of the spectral wave model needs to be adjusted for these conditions.

Based on the nearly 30-yr records of NOAA's two NDBC buoys in Lake Michigan, the frequency range was selected for WAM modeling by Jensen et al. (2012) according to the following specifications. The starting frequency band was set at 0.06116 Hz which corresponds to the longest wave period considered, 16.5 sec. Setting the starting frequency band to this value will assure there is a reasonable lower limit for frequency downshifting during an extreme storm event. The discrete frequency range limit of the model also needs to be consistent with the required range in WAM to assure the Discrete Interaction Approximation (nonlinear wave-wave interaction source term) is properly defined. To minimize the approximations for initial wind-wave growth, the number of discrete frequency bands was set to 28, and the last frequency was set equal to 0.8018 Hz or a wave period of approximately 1.2 sec. Neither WAM nor any other discrete spectral wave model was developed to accurately estimate wind-generated waves for period conditions down to 1.2 sec. However, the relaxation time for initial wind-wave growth is relatively short (on the order of minutes) and the amount of wave energy contained in these higher frequency bands (lower period bands) will be, at their maximum, an order of magnitude less than that contained at the spectral peak. Selection of this high frequency limit reduces the approximations made by the model in the parametric region of the estimated spectrum, minimizing most sources of error; and selection of this value does not increase computational requirements inordinately. Selection of appropriate wave-model frequency bands for each lake should be examined independently.

Sensitivity testing should be performed to determine the optimal wave model grid resolution and directional resolution, and both should be defined by considering the computational requirements versus the value-added of the higher resolution wave modeling. Value-added was assessed for Lake Michigan by Jensen et al. (2012) via comparisons for a series of model tests. Based on results of testing, a spatial wave model grid resolution of 0.02-deg was adopted, and a 5-deg directional resolution was selected for all storm wave simulations. Sensitivity tests are recommended to define the wave model grid resolution and the directional resolution required for adequately resolving the wave energy spectrum in each lake. Sensitivity to the two different wind input sources, NNM and CFSR, should also be examined and evaluated using measured wave data.

D.3.4.2.1.3 **Ice Cover**

During each year the Great Lakes become ice covered, some completely, some partially. In general, the formation of ice develops from the shoreline (shore-fast ice) toward the offshore. The presence of shore-fast ice presents a natural impediment for storm generated waves to reach the shoreline under certain times and conditions, and at certain locations. If ice cover is not considered in the wave and storm surge modeling, the quality of the long-term wave and surge climatology might suffer, with potential for introducing biases into the flooding analysis. Neglecting ice cover could overstate the frequency and severity wave and surge conditions at the shoreline in the winter, therefore ice cover should be accounted for in the wave modeling.

Assel (2005), NOAA GLERL, produced digital weekly ice atlases for the Great Lakes. Synoptic ice chart observations for the Great Lakes began in 1960. A synoptic ice chart usually covers only a portion of one or more of the Great Lakes. Synoptic ice charts for a 20-winter period (1960 to 1979) were digitized (Assel, 1983), and a multi-winter statistical analysis of ice concentration patterns over nine half-month periods (December 16-31 to April 16-30) was published as a NOAA Great Lakes Ice Atlas (Assel, 1983) thirty years ago. Composite ice charts were produced starting in the 1970's, based upon a blend of observations from different data sources (ships, shore, aircraft, and satellite). These charts cover the entire area of the Great Lakes for a given date, and may contain some estimated ice cover data. A 30-winter (1973-2002) set of composite ice charts was digitized, and a multi-winter statistical analysis of the climatology of the ice cover concentration was completed more recently (Assel, 2003). There are three primary ice cover data bases available. Note there is overlap in time between two of the data bases. For development of ice field input to Great Lakes wave and storm surge modeling, the Digital Ice Atlas for the period 1973-2002 (Assel, 2003) is the primary source. For storms prior to 1973, the ice concentration data base (Assel, 1983) is used. Recent data for the period 2003 to present were provided directly by NOAA GLERL for use in Great Lakes Mapping studies.

Each database has its own unique characteristics, and these differences complicate the generation of one consistent set of ice cover fields. The observation period varies from daily (and generally interpolated, 1973-2002), to weekly, to bi-weekly. Historically, these products were based on mean monthly distributions of ice. More recently the digital maps have been constructed based on mean weekly analysis techniques. In general, digital ice information is in the form of a longitude, latitude and ice concentration level. The concentration level is estimated from either photographs or based on the return pulse from satellite-based remote sensing methods. One approach for treating ice cover was developed and implemented by Jensen et al. (2012).

Wave model implementation by Jensen et al. (2012) treated ice as a land-water mask that is delineated based on the ice concentration that was chosen to represent open water conditions. This "threshold" ice concentration needs to be pre-selected for implementation in the numerical wave modeling. As the ice cover increases, the open water points in the wave model domain are set to "land" at the locations where ice concentration exceeds the threshold value. In the spring, as the ice-edge disappears, those locations are then set back to water points in the wave model calculations. An ice concentration of 70 percent was adopted by Jensen et al. (2012) as the value to delineate ice-covered versus open-water conditions. Application of ice fields in the modeling efforts and selection of an appropriate concentration level are important. Unfortunately, for the Great Lakes all wave measurement buoys are generally removed in winter so that the ability to examine model results, and validate them, as a function of ice concentration threshold level is limited. Where buoy data enable evaluation of the choice of ice concentration threshold, they should be used to determine the appropriate value. The threshold value of 70 percent was developed based on prior USACE modeling of Lake Michigan and validated using shallow water wave gauges deployed during the winter months. These results provided valuable information to assess the reliability and consistency using the 70-percent threshold value for ice concentration levels (Jensen et al. 2012).

D.3.4.2.1.4 **Model Validation and Skill Assessment**

Validation of the wave modeling approach is critical to the success of the mapping project, to defensibility of the technical approaches that are taken, and ultimately to acceptance of mapping results. Comprehensive model validation shall be performed using objective metrics such as those listed in section D.3.4.1.3.4. If directional information is available from buoys or gages, comparisons for wave direction should also be made. Jensen et al. (2012) describe rigorous wave model validation for Lake Michigan and the results provide information about the skill that can be expected for lakewide regional-scale modeling of Great Lakes storm waves.

D.3.4.2.1.5 **Selecting Model Output Locations**

Incident wave information from the lakewide modeling can be saved at many locations along the shoreline to facilitate eventual analysis of the wave shoaling and surf zones at those same locations. Generally it is desirable to save the information just seaward of the breaking zone for the most energetic incident wave conditions expected at a particular location. However, the decision on where to save information from the lakewide modeling depends on how well the lakewide modeling approach resolves transformation processes in the shoaling and surf zones, how it is to be used as input to methods for transforming the wave conditions through the shoaling and surf zones and then for determination of wave run-up, overtopping, and overland wave propagation. Decisions also should consider the potential for storm surge to influence wave transformation and local wave characteristics, whether or not wave setup is negligible at the save point, and whether or not wave setup is fully accounted for at the save point

D.3.4.2.2 Sheltered Waters

There are alternative methods for estimating incident wave conditions in more isolated areas that are substantially sheltered from significant open-lake wave energy. The USACE Coastal Engineering Manual (USACE, 2003) outlines approaches for making wave estimates in certain idealized wind and restricted-fetch situations. Detailed guidance for treating waves in sheltered areas is provided in *Guidance for Coastal Flood Hazard Analyses and Mapping in Sheltered Waters* (FEMA, 2008). The same wind data sources used in detailed wave modeling, derived using either the NNM method or extracted from the CFSR data archive, can be used to make the wave estimates, as can measured wind data from a nearby land station or interpolated from several land stations.

Nadal-Caraballo et al. (2012) describe step-by-step application of a simple wave calculation method, which they used as a surrogate approach for computing characteristics of wind waves in lieu of more rigorous wave modeling. The simpler method was applied to support selection of a composite storm set for Lake Michigan; for which more detailed lakewide wave and surge modeling was performed for each storm. This type of method is known as the restricted-fetch method and it is described in much greater detail in Nadal-Caraballo et al. (2012). The resulting wave heights and wave periods were estimated under assumptions of offshore and deepwater conditions, in addition to the restricted-fetch assumption. This approach made use of the land-based meteorological station winds for the region, with wind conditions at each site of interest segregated into 15-degree bins. All the necessary meteorological data corrections were performed, including (1) over-land to over-lake winds adjustment, (2) equivalent neutral wind speed adjustment, and (3) wind speed averaging duration adjustment. The adjusted winds were then used to check for duration- or fetch-limited wave conditions, and then appropriate sets of equations were used to compute wave heights and periods.

By utilizing the simple methods outlined above, the Mapping Partner can determine a reasonable wave height and period estimate for use with observed water levels (or those derived from storm surge modeling) in the overland wave height or beach runup analysis in sheltered areas, where more detailed wave and surge modeling is not computationally feasible or where limitations in data availability preclude application of a more rigorous analysis method. The Mapping Partner must discuss use of simpler wave estimation methods with the FEMA Study Representative and obtain agreement for their use.

D.3.4.3 Coupled Storm Surge and Wave Modeling

In shallow regions of a lake or bay, the change in water level that is forced by wind and atmospheric pressure gradients can influence local wave transformation, through the influence of changing water depth on wave refraction, shoaling and energy dissipation due to breaking. In turn breaking waves can contribute to the storm surge, increasing the water depth, through the wave setup contribution. In the deep-water regions of the bay, these interactions between storm surge and waves are negligible. In shallow water the interactions can be more important. Both the contribution of wave setup to storm surge, and the influence of water depth changes on wave transformation, can be treated using coupled wave and surge modeling, as necessary.

This interaction can be handled in the 2-D lakewide wave and surge modeling, provided that sufficient grid resolution is adopted to satisfy the requirements for accurately resolving both wind wave processes and the long-wave surge propagation process in shallow water. Usually the wind wave processes, because they are the phenomenon having a much shorter length scale, dictate grid resolution requirements. The Mapping Partner needs to be aware of limitations in coupled wave and surge modeling applied at a lakewide scale. In many Great Lakes open coast situations, even the adoption of high resolution in lakewide modeling (30 to 50 m is considered very high resolution for regional-scaled wave and surge modeling) might not be sufficient to fully and accurately simulate the effect of wave setup on storm surge (see Figures D.3.5-2, D.3.5-3, and D.3.5-4, for example, and the related discussion). This will be especially true for situations with lower incident wave heights and/or steeper beach slopes, which together create a narrow surf zone, where much or most of the setup contribution is realized at and very near the shoreline. For very gently sloping beaches and inundated floodplains, and large incident wave conditions, which together result in a very wide surf zone, a grid resolution of 30 to 50 m in coupled wave-surge modeling might be adequate for estimating the maximum wave setup at the shoreline.

Alternatives to treating wave setup at the lakewide 2-D modeling scale include: 1) use of sub-regional scale 2-D modeling over smaller domains covering the shoaling and perhaps the surf zone; or 2) use of 1-D surf zone dynamics modeling performed using grid resolution on the order of meters and the Direct Integration Method (DIM). Treatment of wave setup is discussed in detail in Section D.3.5.

D.3.5 Wave Setup, Runup, and Overtopping

This section provides methodology for establishing wave and water-level characteristics in the surf zone including wave setup, runup, and overtopping of sandy beaches and natural or constructed barriers.

D.3.5.1 Overview of Response-Based Approach

The following general steps should be used to conduct a response-based flood frequency analysis in areas where the dominant flood hazard is wave runup. The transect will likely be a runup-dominated shoreline if the calculated still water levels at the landward end of the modeled transect are below the eroded dune or bluff elevation.

1. Extract time-paired values of still water level and incident wave conditions for each of the storms in the composite storm set. Time series data can come directly from the 2-D lakewide modeling, if it properly resolves the shoaling zone, or from an intermediate wave transformation step of the shoaling zone using output from the lakewide modeling as input. The multiple-day time series data for each storm can be pared to capture the build-up and peak stages of each storm. Process these data as necessary for input to the following steps.

2. Apply either a 1-D surf zone dynamics model or an empirical runup formula to estimate the maximum runup elevation for each storm, using results from step 1 as input. If erosion is to be considered, the eroded profile should be generated prior to calculating wave runup since changes in water depth and slope may affect the runup results.

3. Using the set of runup elevation maxima for the entire set of storms, conduct a statistical extreme value analysis on the runup elevations to determine the 1-percent-annual-chance runup elevation, or total water-level (TWL) values. See Section D.3.3.7.

4. If the runup at a particular transect produces the overtopping of a barrier, and the overtopping produces a potential flooding hazard, overtopping can be assessed using predictive methods described in section D.3.5.4.

D.3.5.2 Wave Setup

D.3.5.2.1 Description of Wave Setup

Waves can affect the mean water level at the shoreline during severe storms through the transfer of momentum from waves to the water column during the shoaling and breaking processes. As waves break on a beach, wave heights decrease and the flux of wave momentum in the onshore direction is reduced. This creates a compensating force that is exerted on the water column, as shown in Figure D.3.5-1. The water-level increases to produce the compensating force, an increase called wave setup. The magnitude of wave setup is greatest at the shoreline, where the maximum value is roughly 10 to 20 percent of the incident wave height at the seaward edge of the surf zone, i.e., the breaking wave height.

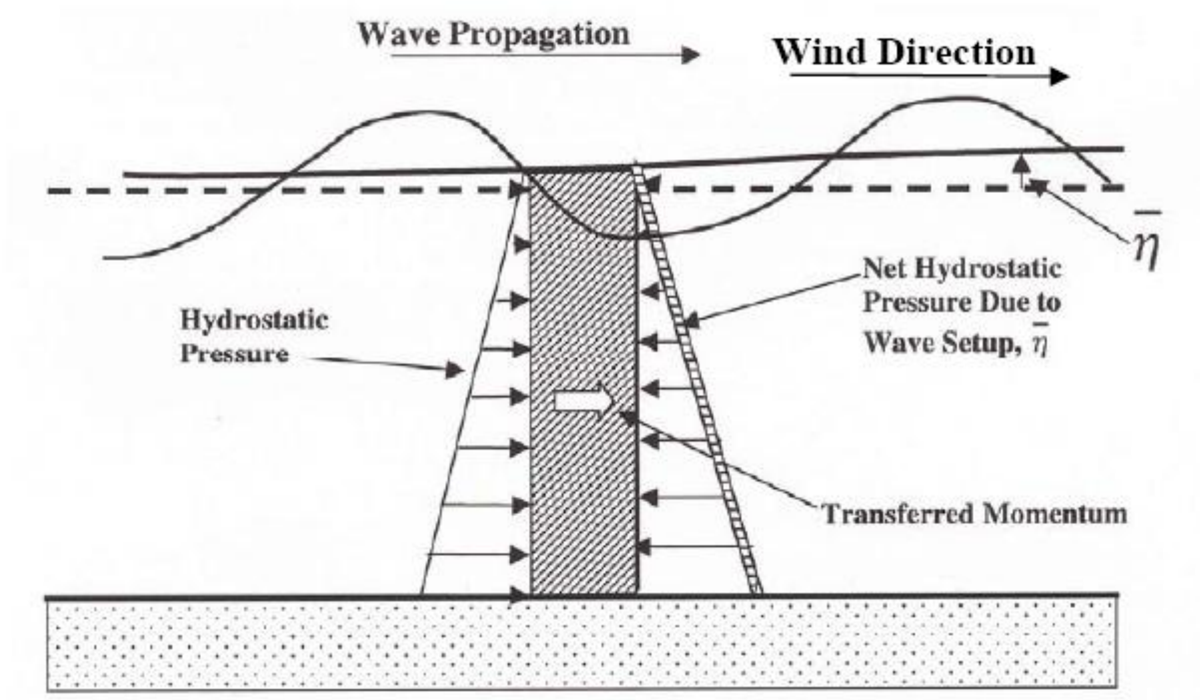


Figure D.3.5-1. Wave Setup Due to Transfer of Momentum

This is a "static" wave setup, which remains approximately constant as long as the storm tide and incident wave conditions remain unchanged. Static wave setup is treated as a mean quantity in time. Factors that affect wave setup include wave nonlinearity, wave breaking characteristics, beach slope and changes in slope, and wave propagation through vegetation.

Oscillations in the wave setup will also occur in nature, and this oscillation is known as "dynamic" wave setup. These oscillations will typically occur with periods of 10 to 20 times the mean wave period. The dynamic wave setup increases with narrow frequency spectra and narrow directional spectra, both uncharacteristic of storm conditions in the Great Lakes. Therefore, the dynamic setup component is considered to be small by comparison with the static component for the Great Lake applications, and should not be included at present in the calculations for the Great Lake water levels.

D.3.5.2.2 Wave Setup Implications for Flood Mapping

Wave setup can be a significant contributor to the still water level (as much as several feet for Great Lakes conditions) and should be included in the determination of coastal BFEs.

For the vast majority of Great Lakes coastal settings and situations, storm surge and wave setup are to be treated concurrently, either through dynamically coupled 2-D surge and wave models or through application of a 1-D surf zone dynamics model (with incident wave and storm surge as inputs) that inherently computes wave transformation and setup, or through the use of empirical methods for predicting wave runup that implicitly include the effects of wave setup.

The recommended method for calculating wave setup in Great Lakes FISs is either the use of a 1-D surf zone dynamics model applied along a transect at a sufficient cross-shore resolution (order of meters) in which storm surge and wave transformation are coupled in the modeling, or the use of coupled 2-D wave and surge models, provided sufficient resolution is adopted in the surf zone to be able to compute wave setup accurately.

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

D.3.5.2.3 Wave Setup Using a 1-D Surf Zone Model

Use of one-dimensional surf zone dynamics models for transects, applied at a cross-shore resolution on the order of meters, represents a more accurate approach for treating the following important coastal processes in a single calculation step: 1) surf zone breaking and wave energy dissipation that accounts for the influence of irregular morphology, 2) beach erosion which creates a steeper foreshore slope during storms which in turn increases the wave runup, 3) possible erosion of dunes that have been created during the low lake levels and subsequent increase in flood hazard that can arise from dune degradation at higher lake levels, and 4) wave setup and runup at the shoreline where the maximum value of wave setup occurs. Accurate calculation of wave setup for Great Lakes beach settings using modeling must adequately resolve and represent the inner surf zone where beach slopes are greatest and much of the wave setup is forced. This generally requires grid resolution that is on the order of meters.

D.3.5.2.4 Parametric Representation for Estimating Wave Setup

A simple method for calculating the effect of wave setup separately is the Direct Integration Method (DIM). The DIM was developed in conjunction with the FEMA-sponsored development of the Pacific Coast *Guidelines* (FEMA 2004). This method can be applied in situations where the application of more rigorous surf zone modeling is not warranted in light of input data limitations, or in conjunction with application of simple wave estimation techniques that to do implicitly treat wave setup. DIM yields wave setup estimates at any point along a shore-normal transect.

The Pacific Guidelines technical working group compared wave setup results calculated with the DIM method to those calculated using the method suggested by the USACE Shore Protection Manual, SPM, (USACE, 1984) and those calculated using the method developed by Goda (2000). The working group found that the DIM methodology yielded wave setup values ranging from 60 to 100 percent larger than those from the SPM method. However, the DIM methodology values were less than 16 percent greater than those predicted by Goda. It was concluded by the working group that the DIM method provides a better estimate of wave setup than the SPM methodology. A reduction of up to 16 percent (based on the comparison with the Goda methodology) may be applied to the DIM results if evidence[2] suggests a reduction is appropriate.

The DIM methodology can be written as follows for the maximum wave setup ($\bar{\eta}$) at the shoreline, which allows direct calculation of the effect of the profile slope (*m*) and deepwater wave steepness (H_o/L_o) on wave setup.

$$\bar{\eta}/H_o' = 0.160\frac{m^{0.2}}{(H_o'/L_o)^{0.2}} \qquad \text{(D.3.5-1)}$$

An estimate must be made of the beach slope to apply the DIM method, and there is some subjectivity in that choice. In a modeling approach to calculate wave setup, the beach slope is calculated implicitly based on actual bathymetric data in the surf zone, and it can vary through the surf zone.

Note that the SPM and Goda methods provide the wave setup at the landward limit of flooding. Thus, in some cases a method might be required to determine the wave setup value at the still water shoreline in the absence of wave setup for later transect applications. It is recommended that the Mapping Partner proportion the maximum wave setup as determined by the SPM or Goda method to determine the approximate wave setup at this alternate location. Denoting the wave setup at the no-wave-setup shoreline as $\overline{\eta_o}$ and the maximum setup as $\overline{\eta_{\max}}$, $\overline{\eta_o}$ can be approximated as:

$$\overline{\eta_o} = \left[1 - \frac{3\kappa^2}{8}\frac{1}{\left(1 + \frac{3\kappa^2}{8}\right)}\right]\overline{\eta_{\max}} \qquad \text{(D.3.5-2a)}$$

which simplifies to:

$$\overline{\eta_0} = \left[\frac{8}{\left(8 + 3\kappa^2\right)}\right]\overline{\eta_{\max}} \qquad \text{(D.3.5-2b)}$$

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

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

$$\overline{\eta_o} = 0.88 \text{ to } 0.94\ \overline{\eta_{max}} \approx 0.9\ \overline{\eta_{max}} \qquad \text{(D.3.5-2c)}$$

The Atlantic and Gulf coast guidelines provide additional details on wave setup including considerations for wave/structure interactions, dissipation over vegetation, and island and backshore situations which might also be suitable for application in the Great Lakes. Working examples of analysis in these situations are also included. The Mapping Partner should refer to section D.2.6.3 for more information.

D.3.5.3 Wave Runup

D.3.5.3.1 Description of Wave Runup

Wave runup is the uprush of water from wave action on a beach or shore barrier such as a steep dune, bluff or coastal structure. The wedge of water associated with a breaking or broken wave 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 mapping purposes is the wave runup elevation, which is the vertical elevation above the still water level that is ultimately attained by the extremity of the uprushing water, as illustrated in Figure D.3.5-2.

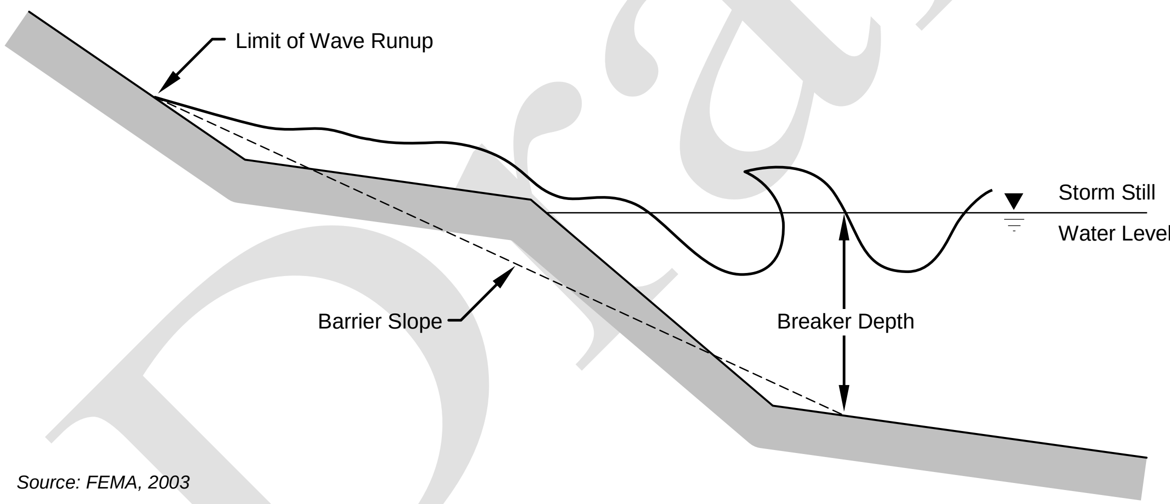


Figure D.3.5-2. Wave Runup Schematic

Runup is a function of nearshore wave transformation and wave breaking across the surf zone, and their influence on wave height, period and direction. Runup on beaches also is influenced by local bathymetry, beach steepness, beach composition, beach permeability, and groundwater elevation. For structures, runup also is influenced by bathymetry seaward of the structure, structure geometry, porosity/roughness, and core permeability. Runup can vary considerably along shore.

Wave runup is an extremely important contributor to BFEs along many sections of Great Lakes coastline. For many locations, wave runup heights are larger than the range of long-term or seasonal-scale lake level changes, wave setup, and the storm surge.

Summaries of different methods for predicting wave runup have been compiled in various publications (e.g. Kobayashi, 1999; CEM, 2003; and the EurOtop Manual, 2007). Melby (2012) provides a review of runup methods for FIS studies. As noted by Kobayashi, wave runup on coastal structures has been studied mostly by engineers using hydraulic physical models whereas wave runup on beaches has been studied mostly by oceanographers using field measurements.

Use of alternative treatments of runup to those presented in these guidelines must be approved by the FEMA study representative.

D.3.5.3.2 Definition of the Limit of Wave Runup

The current policy of the NFIP is to define the wave runup elevation as the value exceeded by 2 percent probability of exceedance of runup events[3]. The 2-percent exceedance value was chosen during the development of the Pacific Coast *Guidelines and Specifications for Flood Hazard Mapping Partners* (Section D.4). This runup elevation is a short-term statistic associated with a group of waves or a particular storm. It is a standard definition of runup, commonly denoted as $R_{2\%}$. This 2-percent exceedance designation is different from the 1-percent-annual-chance designation associated with long-term extreme value statistics. The 1-percent-annual-chance condition has a 1-percent annual probability of occurrence, which corresponds approximately to the 100-year condition, while the runup statistic corresponds to a 2-percent probability of exceedance during a half hour or hour of wave action. The 2-percent exceedance runup is denoted as $R_{2\%}$. Unless otherwise indicated, the runup referred to in all sections of D.3 is the 2-percent exceedance runup elevation.

The Mapping Partner must be aware of the relationship between still water level, wave setup, and wave runup. Outputs from many runup and overtopping calculation procedures, including those recommended in these Great Lakes guidelines, implicitly include wave setup effects. The Mapping Partner must also know whether water-level outputs from wave and storm surge models (which will be used as inputs to transect-based wave height, wave runup, wave overtopping and erosion analyses) include or exclude wave setup, and the degree to which wave setup is fully reflected in the model output, particular in the inner surf zone.

D.3.5.3.3 Recommended Methods for Predicting Runup

Melby (2012) examined several popular empirical methods for predicting wave runup on structures and beaches. Included were the Hunt (1959)-based formulations of Holman (1986), Ahrens (1981), Mase (1989), van der Meer and Stam (1992), and van Gent (1999a, b), the momentum flux method of Hughes (2004b), and the formulation of Stockdon et al. (2006). Two computer programs, ACES (USACE 1992) and Runup 2.0 (FEMA 1981, 1991) that are based on

[3] Walton (1992) concluded that both theory and laboratory experiments show that the 2-percent exceedance runup height above the still water level is approximately 2.2 times the mean runup height.

empirical methods and the CSHORE 1-D numerical surf zone dynamics model (Kobayashi et al. 2009, and Johnson et al. 2012) were also evaluated. Recommendations from Melby (2012) are adopted for these guidelines.

D.3.5.3.3.1 **1-D Surf Zone Dynamics Model**

An attractive solution is to numerically model the dynamics of nearshore wave transformation across a transect of the surf zone, through the swash zone, and up to the extent of wave runup, including changing morphology and dune erosion, and overtopping, if necessary. Several hydrodynamic models for modeling surf zone transects are in wide use and they generally fall into two categories: phase-averaged and phase-resolving. Phase-averaged models based on the nonlinear shallow water wave (NLSW) equations, such as CSHORE, have been widely discussed in the literature (Kobayashi 1997, Kobayashi 2009). The primary advantage of NLSW surf zone dynamics models is that they incorporate many of the important physical processes, run very quickly and are very stable. The disadvantage is that they do not model the detailed transformation of each wave in the spectrum so they might miss some physics in some cases. An example is modeling both incident and infragravity components of an incident wave spectrum.

Advanced phase-resolving models based on the Boussinesq equations have also gained recent popularity for practical application. The primary advantage of the Boussinesq-type models is that they capture the wave-to-wave physics so they can, in some cases, model the details of the spectral wave transformation including long wave generation within the surf zone. The primary disadvantage is that they run much slower than the NLSW 1-D surf zone dynamics models and are less stable. For studies where hundreds to thousands of transects are modeled for hundreds of storms, as is the case for flood hazard studies in the Great Lakes, detailed phase-resolving modeling for all transects and all storms may not be practical at the present time; but it might be desirable in areas of high complexity and in areas of dense population and/or critical infrastructure that vulnerable to flooding.

Existing numerical models of the NLSW class can provide consistent prediction of runup from steep to shallow slopes, including structure/beach porosity and roughness, and account for complex nearshore processes on irregular bathymetry. CSHORE has the option of including morphology change, bottom porosity, and many other complex nearshore processes. CSHORE runs extremely fast – a few seconds per storm per transect is typical. It is also very stable. The programs have been validated for a large number of data sets as described in the many references (see Johnson et al., 2012, for validation to storm-induced morphology change data sets and Melby, 2012, for validation to run-up data sets, for both structures and beaches).

For most shoreline conditions, a 1-D surf zone dynamics model, such as CSHORE, should be used to predict wave runup. This includes both beaches and coastal structures, particularly for those structure/beach configurations that are quite different from those considered in the development of the empirical predictors. The CSHORE model provides great flexibility for calculating wave runup for a wide range of beach settings, beach/structure situations and wave conditions. CSHORE also can be used to predict cross-shore beach morphology change, the steepening of beaches during storms, and the resulting influence on runup which is sensitive to beach slope. In addition, CSHORE can be used to predict wave overtopping of structures and dunes, although that was not analyzed by Melby (2012). Melby (2012) found that CSHORE does not predict runup well on very gently-sloping, dissipative beaches where the surf similarity parameter, ξ_{op}<0.3. For these latter cases, it is recommended that the Stockdon empirical equation listed below be used.

D.3.5.3.3.2 **Empirical Formulas**

An alternative to 1-D surf zone dynamics modeling is to use empirical formulas. For those cases where the local coastal setting and wave/water-level conditions are similar to those that were used to derive the empirical prediction methods, such as wave runup on a planar slope or overtopping of a planar sloped rubble-mound coastal structure, these approaches provide an alternate and less computationally intensive method compared to use of a 1-D dynamics model.

D.3.5.3.3.2.1 Runup on Beaches

The most general method is also the simplest and is a simple adjustment of the Mase formulation, modified to fit the Stockdon beach data (Melby, 2012):

$$\frac{R_{2\%}}{H_{m0}} = 1.1\xi_{op}^{0.7}$$

where H_{m0} is defined in deep water, $\xi_{op} = \tan\beta_f/\sqrt{s_{op}}$, β_f is the foreshore beach slope defined as the average slope over a region between $\pm 2\sigma$ of the mean water level, where σ is defined as the standard deviation of the continuous water-level record, $s_{op} = H_{mo}/L_{op}$, and $L_{op} = gT_p^2/2\pi$. For computing β_f, use can be made of the relationship $\sigma = 0.5H_{mo}$ as given in the Coastal Engineering Manual (CEM) (USACE 2002) and H_{mo} is the value used in the runup determination.

Stockdon gave equations for beaches as follows:

All Beaches:

$$R_{2\%} = 1.1\left(0.35\beta_f\,(H_{m0}L_{0p})^{1/2} + \frac{1}{2}\left[H_{m0}L_{0p}(0.563\beta_f^2 + 0.004)\right]^{1/2}\right)$$

Dissipative Beaches:

$$R_{2\%} = 0.043(H_{m0}L_{0p})^{1/2} \qquad \xi_{0p} < 0.3$$

where wave conditions are defined in deep water. The second equation for dissipative beaches is recommended.

D.3.5.3.3.2.2 Runup on Barriers

In this subsection, "barriers" include steep dune features and coastal armoring structures, such as revetments. Runup elevations on barriers depend not only on the height and steepness of the incident wave (and its interaction with the preceding wave), but also on the geometry (and construction) of the structure. Runup on structures can also be affected by antecedent conditions resulting from the previous waves and structure composition. Because of these complexities, runup on structures is best calculated using equations developed with tests on similar structures with similar wave characteristics, with coefficients developed from laboratory or field experiments.

For structures, the van Gent equations should be used:

$$\frac{R_{2\%}}{\gamma H_s} = \begin{cases} c_0 \xi & \xi \le p \\ c_1 - c_2/\xi & \xi > p \end{cases}$$

where H_s is $H_{1/3}$ at the structure toe, the surf similarity parameter $\xi = \tan\alpha/\sqrt{s}$, $s = H_s/L_o$, $L_o = gT^2/2\pi$ and $c_2 = 0.25(c_1)^2/c_0$, $p = 0.5c_1/c_0$, and the wave period T is given by one of the following:

T_p: $c_0 = 1.35$ $c_1 = 4.3$ $c_2 = 3.4$ $p = 1.6$

$T_{m\text{-}1,0}$: $c_0 = 1.35$ $c_1 = 4.7$ $c_2 = 4.1$ $p = 1.7$

where Tp is the peak spectral wave period and the negative first moment wave period is defined as Tm-1,0, where Tm-1,0 = m-1/m0, m-1 is the negative first moment of the wave energy density spectrum and $m_n = \int_0^\infty f^n S(f)df$. Although not widely available, Tm-1,0 provides a more stable parameter than Tp because it is based on the integrated wave energy density spectrum rather than the somewhat uncertain peak of the spectrum. The wave period, $T_{m\text{-}1,0}$, can be approximated as:

$$T_{m-1.0} = \frac{T_p}{1.1}$$

In deepwater, Hmo is approximately the same as Hs, but in shallow water, Hmo is 10 to 15 percent smaller than Hs. If Hmo is used in the above Van Gent equations for Hs , then the following applies:

$T_{m\text{-}1,0}$: $c_0 = 1.45$ $c_1 = 3.8$ $c_2 = 2.5$ $p = 1.3$

The influence coefficient, given as $\gamma = \gamma_f \gamma_\beta$, is a cumulative adjustment for slope roughness (γ_f) and wave directionality (γ_β). For a berm, use an average structure slope of $\tan\alpha = 4H_s/L$, where L is the horizontal distance between points on the structure at $2H_s$ below and $2H_s$ above the still water line. The directionality factor is given as $\gamma_\beta = 1 - 0.0022\beta$ for $\beta < 80^o$ where β is the incident wave angle from shore normal.

Roughness reduction factors, γ_f, are 1.0 for smooth slopes, 0.9 for grass-covered slopes, 0.6 for single layer rock slopes and 0.5 for multi-layer rock slopes. Roughness coefficients for stone and concrete armored structures are given in the EurOTop Manual (2007) and are repeated in Table D.3.5-1. Values are only repeated for armoring types used in the U.S.

Table D.3.5-1. Roughness Factors for Varied Types of Armoring

Type of armor layer	γ_f
Rocks (1 layer, impermeable core)	0.60
Rocks (1 layer, permeable core)	0.45
Rocks (2 layers, impermeable core)	0.55
Rocks (2 layers, permeable core)	0.40
Cubes (1 layer, random positioning)	0.50
Cubes (2 layers, random positioning)	0.47
Antifer cube	0.47
CORE-LOC®	0.44
Tetrapod	0.38
Dolos	0.43

Runup on stepped embankments has been investigated for a number of site-specific cases. Melby et al. (2009) suggested using a roughness coefficient of $\gamma_r = 0.60$ for stepped embankments. This value is less than the values given in the CEM for rectangular blocks on an otherwise smooth impermeable slope. For blocks, it is suggested using $\gamma_r = 0.70 – 0.95$ depending on the geometry of the blocks and how they are distributed on the slope.

D.3.5.3.3.2.3 Vertical Walls

Vertical walls exist as shoreline structures along the Great Lakes coastline. Often a seawall exists at the top of a beach. Sometimes the wall is at the crest of a coastal structure. The walls can be vertical, near vertical, or have a re-curved shape. Many site-specific laboratory studies have been conducted to develop empirical equations for overtopping on walls, however rarely is the runup elevation calculated in the process. The CEM (USACE 2003) gives a variety of predictive overtopping equations specific to vertical and re-curved wall geometries for seawalls and breakwater crest walls. Because the data are mostly site-specific, application of these equations for general use can be difficult and can require significant experience. However, the predictive equations given in the CEM are the state of practice and should be used for flood hazard estimates at this time. If runup elevations are desired for mapping of a BFE at the wall, the recommended method for calculating runup on a vertical wall is taken from the Shore Protection Manual (USACE, 1984) and documented in section D.2.8.1.4 of these guidelines.

D.3.5.3.4 Interpretation of Wave Runup Results

To interpret and apply the calculated wave runup results properly, the Mapping Partner shall examine the results of the analysis carefully. One important consideration is that a 2-percent exceedance runup elevation below the crest of a given barrier does not necessarily imply that the barrier will not occasionally be overtopped by floodwaters. Also, analysis might yield a runup elevation exceeding the maximum barrier elevation; this outcome can occur because the analysis assumes a positive slope to continue indefinitely. For bluffs or eroded dunes with negative landward slopes, a general rule has been used that limits the wave runup elevation to 3 feet above the maximum ground elevation, even when the potential runup along the imaginary slope extension exceeds 3 feet. When the runup overtops a barrier, such as a partially eroded bluff or a structure, the floodwater percolates into the bed and/or runs along the back slope until it reaches another flooding source or a ponding area. The runoff-influenced areas are usually designated as Zone AO, with a depth of 1, 2, or 3 feet. Ponding areas are designated as Zone AH (depth of 3 feet or less), with BFEs shown.

When the potential runup is at least 3 feet above the barrier crest, a VE Zone is delineated landward of the barrier, as shown in Figure D.3.5-3. The BFE for that VE Zone is capped at 3 feet above the crest of the barrier. When the runup depth in excess of the barrier crest is 0.1 to 1.5 feet, the VE Zone BFE is the runup elevation (rounded to the nearest whole foot), and an AO Zone with a depth of 1 foot should be mapped landward until another flooding source is encountered (Zone AE) or the floodplain limit is reached (Zone X). Similarly, for a runup depth of 1.5 to 2.9 feet above the barrier crest, the VE Zone BFE is the runup elevation (rounded to the nearest whole foot). In this case, however, an AO Zone with depth of 2 feet should be mapped, then transitioned landward into an AO Zone with a depth of 1 foot and then into subsequent flood insurance risk zones, if any. Detailed guidance on mapping is contained in Section D.3.9.

A distinct type of overflow situation can occur at low bluffs or banks backed by a nearly level plateau, where calculated wave runup may appreciably exceed the top elevation of the steep barrier. A memorandum entitled *Special Computation Procedure Developed for Wave Runup Analysis for Casco Bay, FIS - Maine, 9700-153* provides a simple procedure to determine realistic runup elevations for such situations, as illustrated in Figure D.3.5-4 (French, 1982). An extension to the bluff face slope permits the computation of a hypothetical runup elevation for the barrier, with the imaginary portion given by the excess height R' = (R-C) between the calculated runup and the bluff crest. Using that height (R') and the plateau slope (m), Figure D.3.5-5 defines the inland limit to a wave runup (X) corresponding to the runup above the bluff crest (mX) or an adjusted runup elevation of R_a = (C + mX). This procedure is based on a Manning's “n” value of 0.04, with some simplifications in the energy grade line, and is meant for application only with positive slopes landward of the bluff crest.

These runup assessment procedures are given for general guidance, but they may not be entirely applicable in certain situations. For example, runup elevations need to be fully consistent with the wave setup and wave overtopping assessments described in the subsections that follow. In problematic cases, the Mapping Partner shall use good judgment and rely on the historical data to reach a solution for the realistic flood hazards associated with a shore barrier. Section D.3.9 considers the integration of separately calculated wave effects into coherent hazard zones for the base flood. When a unique situation is encountered, the Mapping Partner shall consult the FEMA Study Representative.

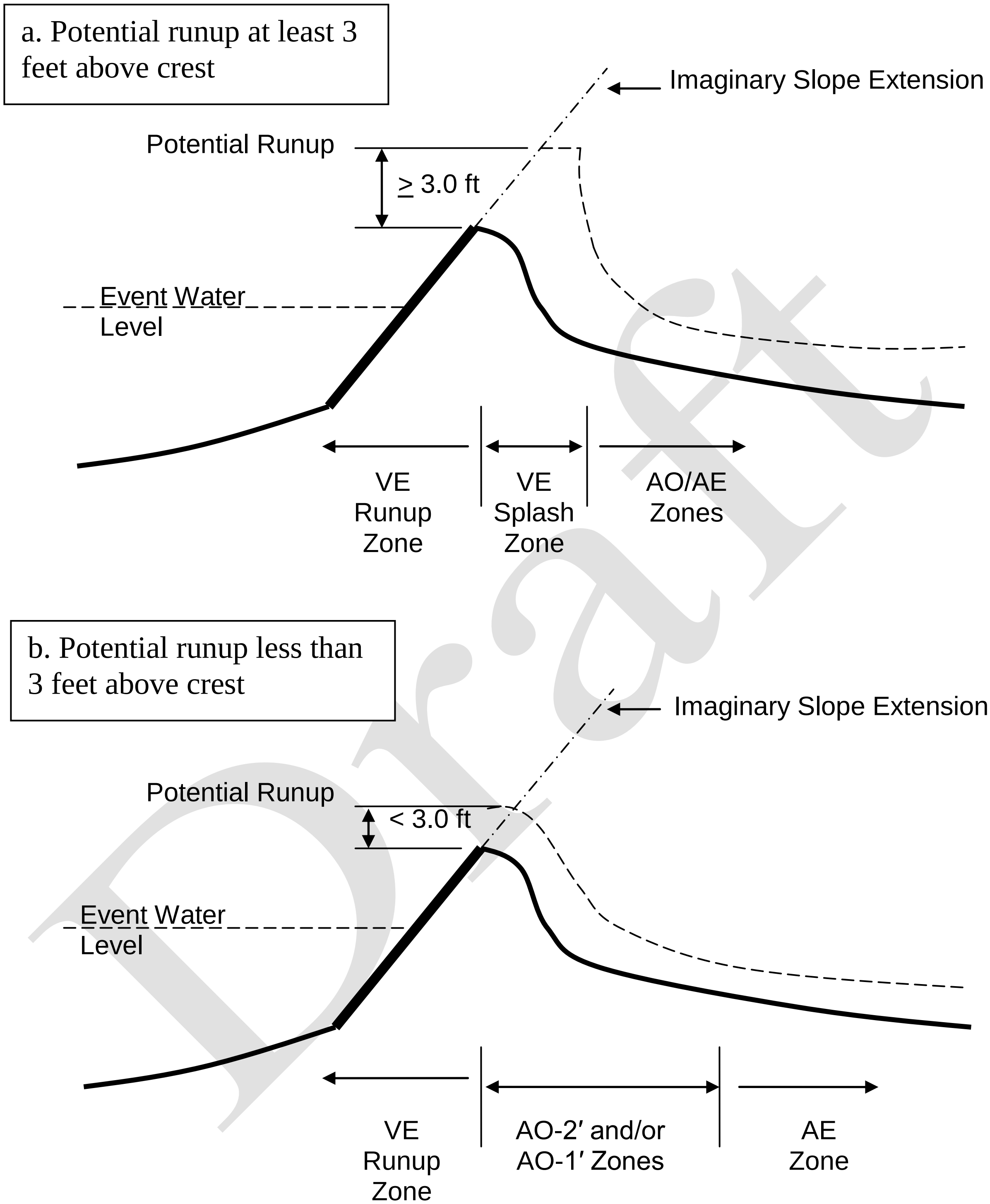


Figure D.3.5-3. Simplified Runoff Procedures (Zone AO)

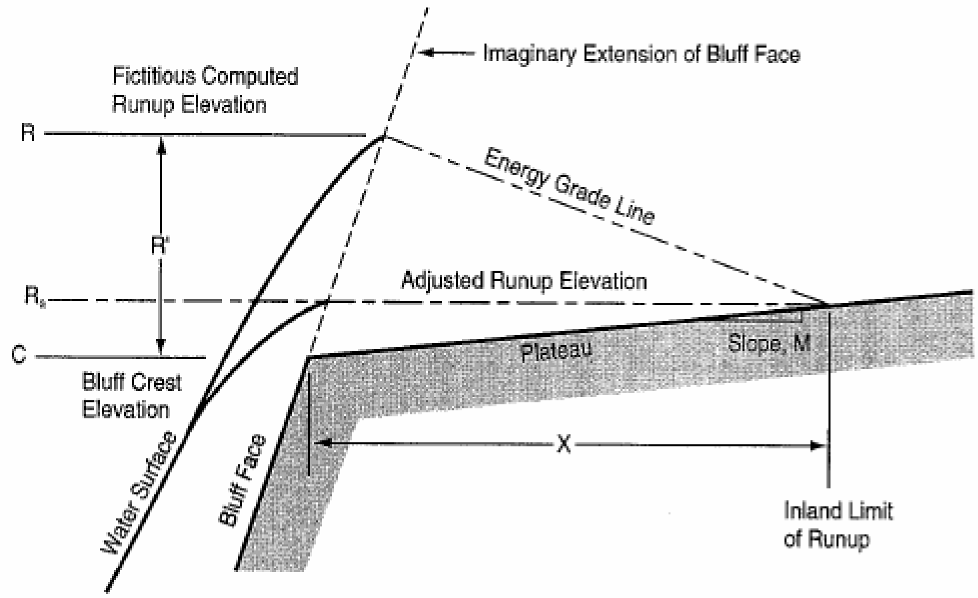


Figure D.3.5-4. Treatment of Runup onto Plateau above Low Bluff

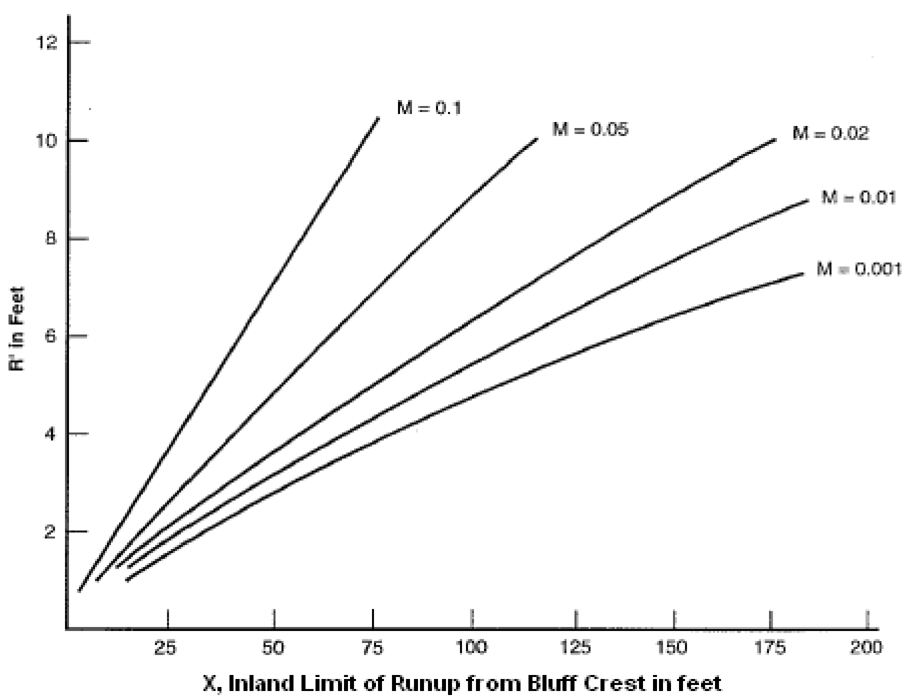


Figure D.3.5-5. Curves for Computation of Runup Inland of Low Bluffs

D.3.5.3.5 Documentation

The Mapping Partner shall document the procedures and values of parameters employed to establish the wave runup on the various transects on natural beaches and barriers, which could include steep dunes and structures. In particular, the basis for establishing the runup reduction factors and their values shall be documented. The documentation shall be especially detailed if the methodology deviates from that described herein. Any measurements, observations and/or anecdotal information regarding previous major storm-induced runup shall be recorded and documented. Any notable difficulties encountered and the approaches to addressing them shall be described clearly. Additional information on required documentation criteria can be found in Section D.3.10.

D.3.5.4 Wave Overtopping

D.3.5.4.1 Introduction

Wave overtopping occurs when a barrier crest height is lower than the potential wave runup level, as shown in Figure D.3.5-6. Waves will flow or splash over the barrier crest, typically to an elevation less than the potential runup elevation (*R*). The extent of the overtopping water surface and overtopping rate will depend on the still water level, incident wave conditions, and the barrier geometry and roughness characteristics. Moreover, overtopping rates can vary over several orders of magnitude, with only subtle changes in hydraulic and barrier characteristics, and are difficult to predict precisely.

The assessment of potential wave overtopping for flood hazard mapping purposes has relied on readily available empirical guidance, historical effects, and engineering judgment. Except for very heavy overtopping, useful guidance has been derived from laboratory tests with irregular waves. Recently, surf zone dynamics numerical models, such as CSHORE, or more rigorous and advanced models based on solutions to the Boussinesq or Reynolds-Averaged Navier Stokes (RANS) equations, provide other options for calculating overtopping rates, particularly for structure/beach configurations that are different from those configurations used in laboratory tests upon which the empirical predictors have been developed. But in general, estimation of overtopping remains highly uncertain. Therefore, the Mapping Partner shall estimate only the order of magnitude of mean overtopping rates, because there are clearly documented thresholds below which wave overtopping may be classified as negligible. While this approach does not account explicitly for highly variable peak overtopping rates and does not offer a complete specification of overtopping hazards, its use is recommended until overtopping rate calculation guidance is improved significantly.

Two publications, *Design of Seawalls Allowing for Wave Overtopping* (Owen, 1980) and *Random Seas and Design of Maritime Structures* (Goda, 1985), provide wide-ranging summaries of mean overtopping rates with storm waves. The former publication addresses smooth-plane or bermed slopes, and the latter publication considers vertical walls with or without a fronting rubble mound. The Coastal Engineering Manual (USACE 2003) and EurOtop (2007) provide authoritative guidance on calculating overtopping rates and on overtopping rate thresholds for damage to different types of coastal barriers. The following provides a summary of pertinent guidance from these publications for FIS studies.

Before applying methods in those primary sources of overtopping guidance, however, some introductory considerations can help to determine whether a detailed wave overtopping assessment is needed for base flood elevation conditions at a specific shore barrier. More information regarding the evaluation of coastal structure stability can be found in section D.3.8.

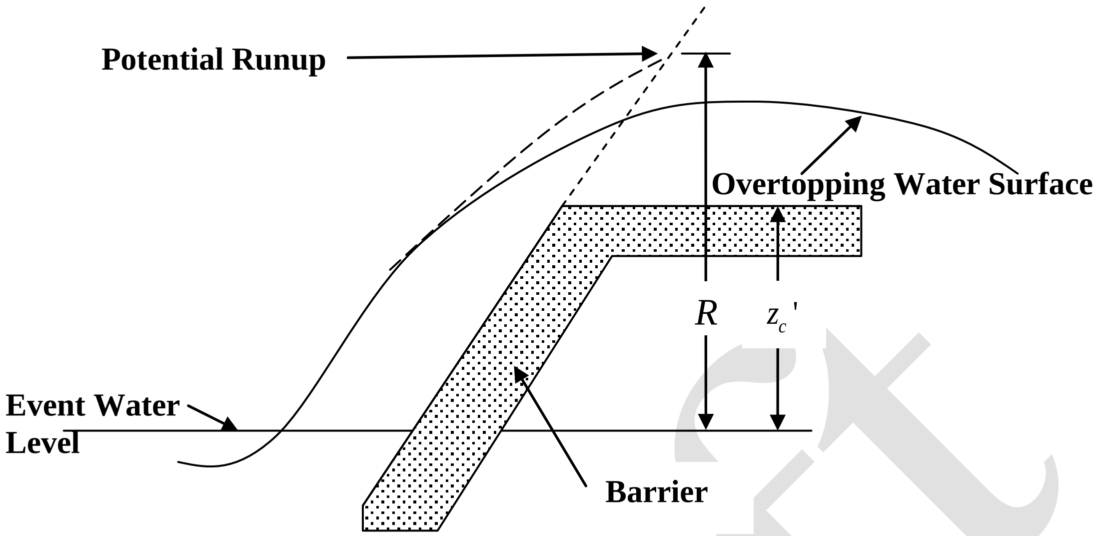


Figure D.3.5-6. Definition Sketch for Wave Overtopping

The initial consideration is an interpretation of the predicted runup elevation. The Mapping Partner should determine if the calculated runup elevation exceeds the crest elevation of the barrier. If the elevation of the barrier crest is exceeded, the Mapping Partner shall assess overtopping rates and potential ponding behind the barrier.

D.3.5.4.2 Mean Overtopping Rates

Once the need for quantitative overtopping assessment is established, wave overtopping estimates for a specified situation must be assessed based on measurements in a similar configuration or using a numerical model that has been verified for similar conditions. Before considering some implications of quantitative guidance for idealized cases, an overview of overtopping magnitudes provides a useful introduction.

Wave overtopping is often specified as a mean discharge: water volume per unit time and per unit alongshore length of the barrier, commonly in cubic feet per second per foot (cfs/ft). By interpreting or visualizing a given mean overtopping rate, the Mapping Partner may take into account actual discharges that are generally intermittent and isolated, being confined to some portion of occasional wave crests at scattered locations. A mean overtopping rate of 0.01 cfs/ft represents a value that should be considered appreciable, and a 1 cfs/ft mean overtopping rate define a threshold where the structural stability of even well-constructed shore barriers becomes threatened by severe overtopping. The 1 cfs/ft mean overtopping rate is well within the range where buildings exposed to overtopping are damaged.

Wave overtopping can be subdivided into distinct regimes illustrated in Figure D.3.5-7:

- Wave only with positive freeboard
- Wave only with zero freeboard
- Surge overflow with negative freeboard

- Combined surge and wave with negative freeboard

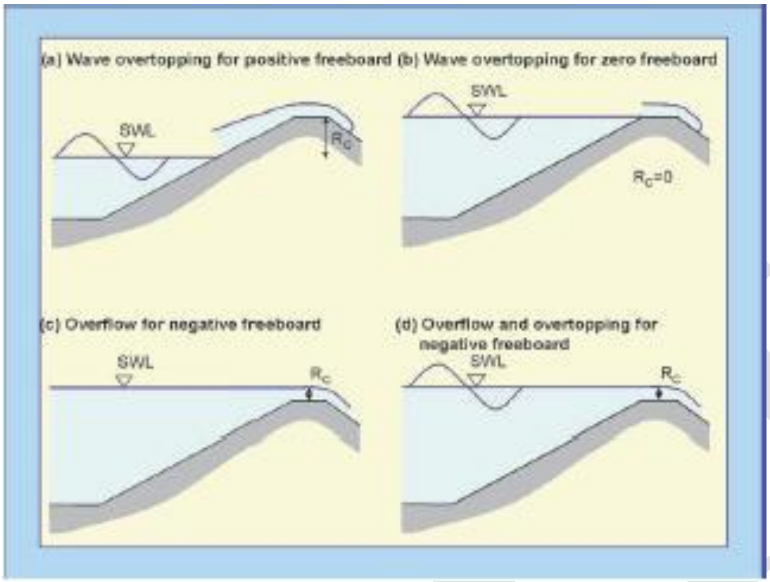


Figure D.3.5-7. Four types of overtopping on levees

(courtesy EurOtop Manual (2007)).

Goda (1985) further subdivided wave-only overtopping into spray, splash, runup wedge, and waveform transmission, in order of increasing intensity. Flood discharges corresponding to the varied regimes naturally depend on the incident wave and water-level conditions. Figure D.3.5-8 shows the association of overtopping volumes with the wave-only regimes noted above illustrating the likely significance of wave overtopping flooding behind a coastal structure. Variables describing the basic situation are cotangent of the front slope for a smooth structure with ideally simple geometry, and freeboard of the structure crest above the still water level (F), as normalized by incident significant wave height (F/H_s). The mean overtopping rate ($\overline{Q}$) is provided in dimensionless form as:

$$Q^* = \overline{Q} / \sqrt{gH_s^3} \qquad \text{(D.3.5-15)}$$

with test results shown for structure slopes of 1:1, 1:2, and 1:4 (Owen, 1980), and for a smooth vertical wall (Goda, 1985). These results pertain to significant wave steepnesses of approximately $2\pi H_s/gT_p^2 = 0.035$, fairly appropriate for extreme extratropical storms; water depth near the structure toe of approximately $d_t = 2H_s$, so that incident waves are not appreciably attenuated; and moderate approach slopes of 1:30 for a vertical wall or 1:20 for other structures. The major feature of the curves is a maximum in overtopping rate for a structure slope of 1:2, corresponding to the gentlest incline producing (at this wave steepness) total reflection rather than breaking, and thus peak waveform elevations (Nagai and Takada, 1972).

These measured results for smooth and simple geometries clearly show severe or "green water" overtopping even for relatively high structures ($F \geq H_s$) for a wide range of common inclinations (cotangents between 0 and 4). Also, for freeboards considered here, a vertical wall (cotangent 0) permits less overtopping than common sloping structures with cotangent less than approximately 3.5. Gentler sloping manmade barriers are uncommon because the construction volume increases with the cotangent squared, so steep coastal flood-protection structures are common and often have porous or roughened faces in order to efficiently attenuate storm waves.

D.3.5.4.2.1 **Wave Only with Positive Freeboard**

Guidance exists for wave overtopping based on modern laboratory model investigations. The following is a summary of present guidance in the CEM and EurOtop Manual (2007). For coastal structures with sloping seaward faces, the normalized mean overtopping discharge rate is given by:

$$Q_w^* = \frac{0.067\gamma_b \xi_{m-1,0}}{\sqrt{\tan\alpha}} \cdot \exp\left(-4.75 \frac{F}{H_{m0}} \frac{1}{\xi_{m-1,0}\gamma_b \gamma_f \gamma_\beta \gamma_v}\right)$$

where

$$Q_w^* \leq 0.2\exp\left(-2.6 \frac{F}{H_{m0}} \frac{1}{\gamma_f \gamma_\beta}\right)$$

Here influence factors are given for berm (γ_b), roughness elements (γ_f), oblique wave attack (γ_β), and wave wall effects (γ_v). The average structure slope including a berm was given in Section D.3.5.3.3.2.2.

The berm influence is computed as

$$\gamma_b = 1 - r_B(1 - r_{db}) \quad \textit{for} \quad 0.6 \leq \gamma_b \leq 1.0$$

$$r_B = \frac{B}{L_{berm}}$$

$$r_{db} = 0.5\left(1 - \cos\left(\frac{\pi d_b}{2H_{m0}}\right)\right) \text{ berm above the SWL}$$

$$r_{db} = 0.5\left(1 - \cos\left(\frac{\pi d_b}{R_{2\%}}\right)\right) \text{ berm below the SWL}$$

where B is the berm width, d_b is the vertical distance between SWL and berm center, and L_{berm} is the influence length of the berm between slope intersecting points one wave height above and one wave height below the berm center. For the case of the berm outside of the influence area, r_{db} = 1.

Roughness coefficients for typical slope coverings were given in Table 3.5-1. Typically, the roughness coefficients are 1 for slope coverings consisting of grass, smooth asphalt, smooth concrete or other smooth surfaces. However, for small wave heights less than about 2.5 feet, grass will influence wave action on the slope and the roughness influence can be computed using the relation:

$$\gamma_f = 1.15\sqrt{H_{m0}}$$

For wave overtopping, the wave obliquity influence factor is given as

$$\gamma_\beta = 1 - 0.0022|\beta| \quad for \quad 0^0 \le \beta \le 80^0$$
$$\gamma_\beta = 0.824 \quad for \quad |\beta| > 80^0$$

where β was described in Section 3.5.3.3.2.2.

If a vertical wave wall is present at the crest of a slope, it will limit wave overtopping. This is accounted for in the influence coefficient γ_v.

In cases where heavy breaking is present on a shallow foreshore (i.e., $\xi_{m-1,0}$ >5.0), long waves influence the predictions leading to underestimation of wave overtopping. In this case when $\xi_{m-1,0}$ >7.0, the following equation should be used for wave only overtopping with positive freeboard:

$$Q_w^* = 10\exp\left(-\frac{F}{\gamma_f \gamma_\beta H_{m0}(0.33 + 0.22\xi_{m-1,0})}\right)$$

Use linear interpolation between these two equations for breaking waves 5< $\xi_{m-1,0}$ <7.

D.3.5.4.2.2 **Wave Only with Zero Freeboard**

Schüttrumpf (2001) and Schüttrumpf et al. (2001) derived equations for average wave overtopping discharge q_w based on model tests with smooth slopes between 1:3 and 1:6. Their results are also presented in the EurOtop manual for overtopping resistant levees when the water level comes close to the crest as:

$$Q_w^* = \begin{cases} 0.0537\,\xi_{m-1,0} & \xi_{m-1,0} < 2.0 \\ 0.136 - \dfrac{0.226}{\xi_{m-1,0}^3} & \xi_{m-1,0} \ge 2.0 \end{cases}$$

D.3.5.4.2.3 **Surge Only Overflow with Negative Freeboard**

If the water level is higher than the crest, then overtopping can be modeled as flow over a broad-crested weir as described for open channel flow (Henderson 1966). The surge only overflow discharge q_s is defined as:

$$\overline{Q_s} = 0.5443\sqrt{g\left|-F^3\right|}$$

where -*F* is the negative relative crest height or overflow depth (ie difference between surge elevation and structure crest elevation).

D.3.5.4.2.4 **Combined Surge and Wave with Negative Freeboard**

An approximation for overtopping with combined wave and surge overtopping is given in the EurOtop Manual (2007) as a superposition of the wave only with zero freeboard and surge only with negative freeboard equations:

$$\overline{Q}_{ws} = \overline{Q}_w + \overline{Q}_s = 0.0537\,\xi_{m-1,0}\sqrt{gH_{m0}^3} + 0.6\sqrt{g\left|-F^3\right|} \qquad \xi_{m-1,0} < 2.0$$

Hughes and Nadal (2008) developed a combined overtopping empirical equation based on small scale laboratory experiments where the normalized overtopping rate is given as a function of freeboard and wave height as

$$Q_{ws}^{*} = 0.034 + 0.53\left(\frac{-F}{H_{m0}}\right)^{1.58} \qquad F < 0$$

Note that *F* must be entered as a negative number to insure that the quantity in brackets is positive.

The incident wave conditions are defined at the structure toe. The defined depth at the toe d_t should always correspond to the scour condition expected due to wave action accompanying the storm still water level.

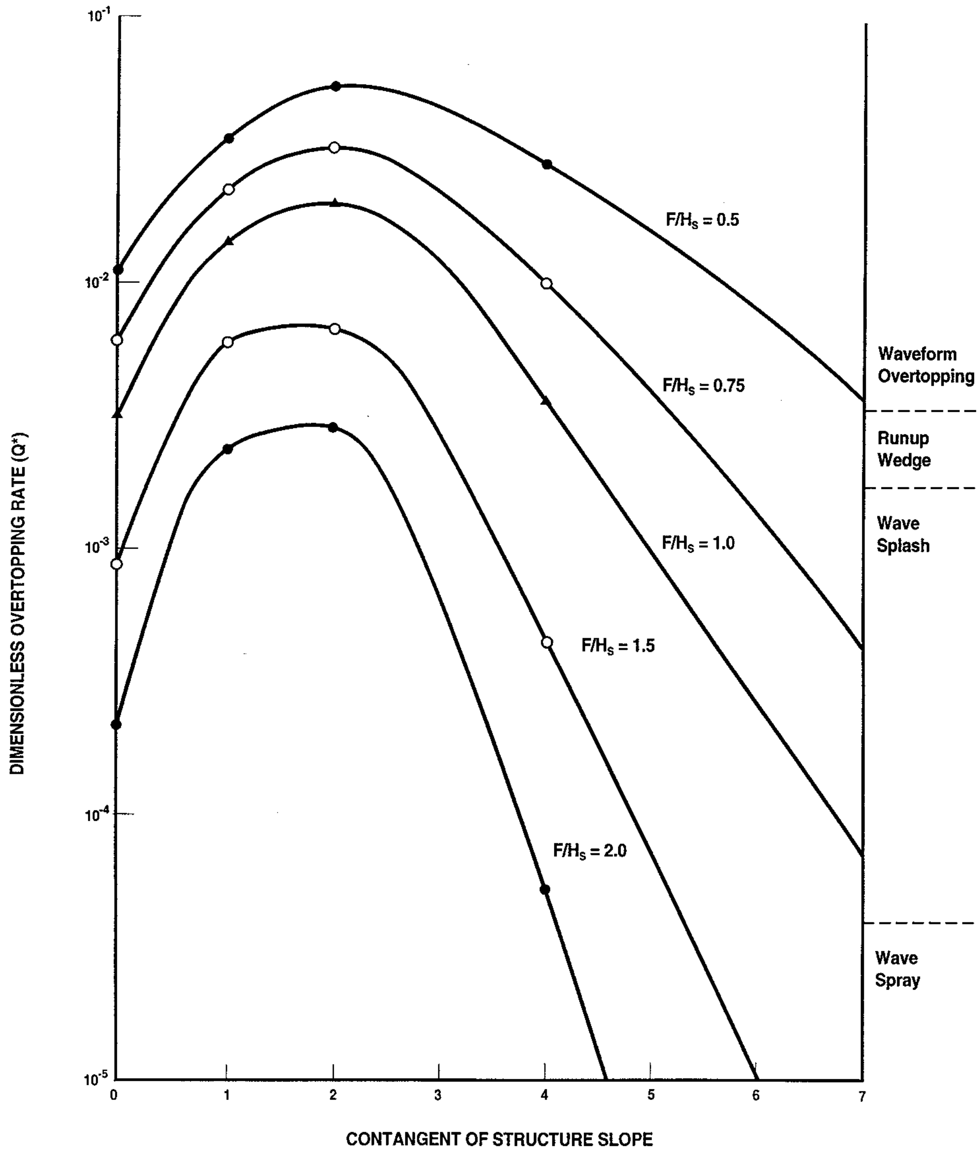


Figure D.3.5-8. Schematic Summary of Storm-Wave Overtopping at Structures of Various Slopes and Freeboards (Goda 1985)

For overtopped vertical walls, the effects of wave attenuation are relatively complex. Random Seas and Design of Maritime Structures (Goda, 1985) provides extensive empirical guidance on various structure situations with incident waves specified for deepwater. Figure D.3.5-9 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. With this information, a specific vertical wall can be categorized as having only modest overtopping ($\overline{Q}$ < 0.01 cfs/ft), intermediate overtopping, or severe overtopping ($\overline{Q}$ > 1 cfs/ft). Runoff or ponding behind the wall may need to be evaluated. Severe overtopping requires a delineation of the landward area susceptible to wave action and velocity hazard. See Section D.3.9 for more detailed information.

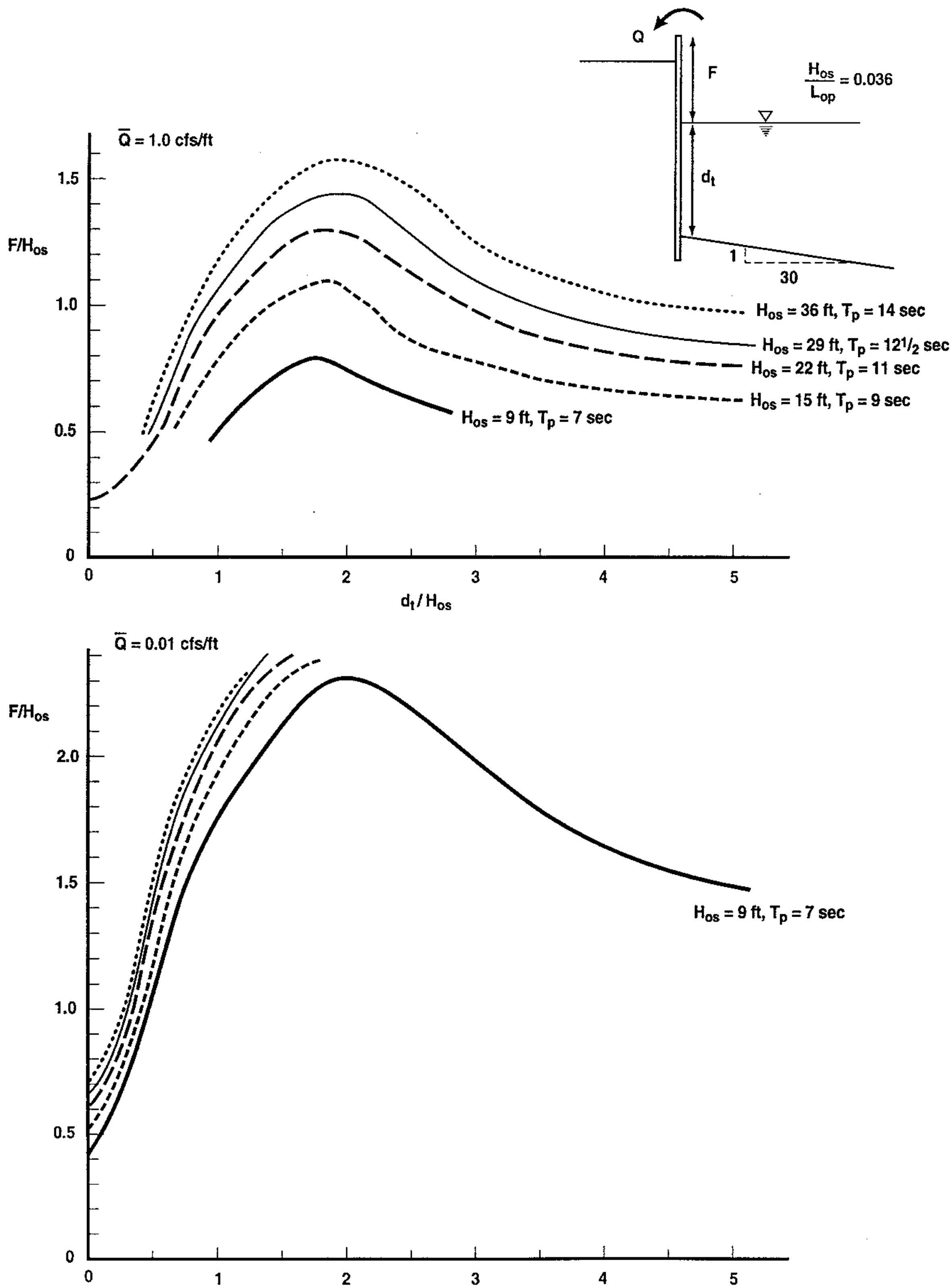


Figure D.3.5-9. Required Freeboard of Vertical Wall to Limit Mean Overtopping Rate to Certain Values. Based on (Goda, 1985)

D.3.5.4.3 Overtopping Rate Considerations for Establishing Flood Insurance Risk Zones

An interpretation of the estimated overtopping rate in terms of flood hazards is complicated by the projected duration of wave effects, the increased discharge possible under storm winds, the varying inland extent of water effects, and the specific topography and drainage landward of the barrier. However, Table D.3.5-2 provides guidance that is potentially applicable to typical coastal situations. Detailed guidance on mapping is provided in Section D.3.9.

Table D.3.5-2. Suggestions for Interpretation of Mean Wave Overtopping Rates

$\overline{Q}$ **Order of Magnitude**	**Flood insurance risk zone Behind Barrier**
<0.0001 cfs/ft	Zone X
0.0001-0.01 cfs/ft	Zone AO (1 foot depth)
0.01-0.1 cfs/ft	Zone AO (2 foot depth)
0.1-1.0 cfs/ft	Zone AO (3 foot depth)
>1.0 cfs/ft*	30-foot width+ of Zone VE (elevation 3 feet above barrier crest), landward Zone AO (3 foot depth)

*With estimated $\overline{Q}$ much greater than 1 cfs/ft, removal of barrier from transect representation may be appropriate.
+Appropriate inland extent of velocity hazards should take into account barrier characteristics, incident wave conditions, overtopping flow depth and velocity, and other factors.

D.3.5.4.4 Ponding Considerations

Once the mean overtopping rate has been estimated for the BFE, determining the resultant flooding landward of the barrier will require the Mapping Partner to evaluate several parameters, including the duration of overtopping, topography, and drainage landward of the overtopped barrier. By integrating the volume of overtopping (mean rate times the duration of the overtopping event) and comparing this to the available storage landward of the barrier, an estimated ponding elevation can be determined. This elevation should be adjusted by the Mapping Partner depending upon rainfall rates associated with the overtopping event, drainage features and systems landward of the barrier, and crest elevations of any features that may allow ponded water to escape. Ponding assumptions and calculations should be reviewed carefully to ensure that overtopping and other potential sources of water trapped behind the barrier are accounted for appropriately.

D.3.6 Overland Wave Propagation

This section provides guidance for estimating overland wave propagation and the associated wave heights and wave crest elevations on flooded land areas. FEMA's WHAFIS model is described.

D.3.6.1 Introduction

The fundamental analysis of overland wave effects for an FIS is provided by the WHAFIS program which uses representative transects to compute heights and wave crest elevations for the study area.

The original basis for the WHAFIS model was the 1977 NAS report *Methodology for Calculating Wave Action Effects Associated with Storm Surges*. The NAS methodology accounted for varying fetch lengths, barriers to wave transmission, and the regeneration of waves over flooded land areas. Since the incorporation of the NAS methodology into the initial version of WHAFIS, periodic upgrades have been made to WHAFIS to incorporate improved or additional wave considerations. Figure D.3.6-1 illustrates the basic factors that WHAFIS considers in its overland wave height and wave crest elevation calculations.

The current WHAFIS 4.0 model is fully documented (Technical Documentation for WHAFIS Program Version 3.0, FEMA, September 1988; Divoky, 2007). WHAFIS 4.0 allows a user-defined wind speed to be input for use in the computation of wave growth along the transect. Past versions of WHAFIS had prescribed a default wind speed of 40 mph for use in all Great Lakes areas. With the adoption of WHAFIS 4.0, this restriction has been removed.

Briefly, the wave action conservation equation governs wave regeneration caused by wind and wave dissipation by marsh plants, tress, and buildings in the model. This equation is supplemented by the conservation of waves equation, which expresses the spatial variation of the wave period at the peak of the wave spectrum. The wave energy (equivalently, wave height) and wave period respond to changes in wind conditions, water depths, and obstructions as a wave propagates. These equations are solved as a function of distance along the wave analysis transect.

The fundamental elements in this wave treatment remain unchanged from the NAS methodology: The controlling wave height[4] (approximately the average height of the highest 1-percent of waves during storm conditions) is limited to 78 percent of the local still water-level depth. Also, the model assumes that 70 percent of the controlling wave height lies above the total SWEL, resulting in the wave crest elevation being 0.55 times the local still water depth above the total SWEL, or 1.55 times the local still water depth above the ground elevation (see Figure D.3.6-1).

The WHAFIS program is available as a stand-alone program, or as a part of FEMA's Coastal Hazard Analysis Modeling Program (CHAMP). CHAMP is a Windows-interfaced Visual Basic

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

program that allows the user to enter data, perform coastal engineering analyses, view and tabulate results, and chart summary information for each representative transect along a coastline, within a user-friendly graphical interface. With CHAMP, the user can import digital elevation data; perform storm-induced erosion treatments, overland wave height analyses, and wave runup analyses; plot summary graphics of the results; and create summary tables and reports in a single environment. The current versions of both programs, WHAFIS 4.0 and CHAMP 2.0 are available for download at FEMA's website, www.fema.gov.

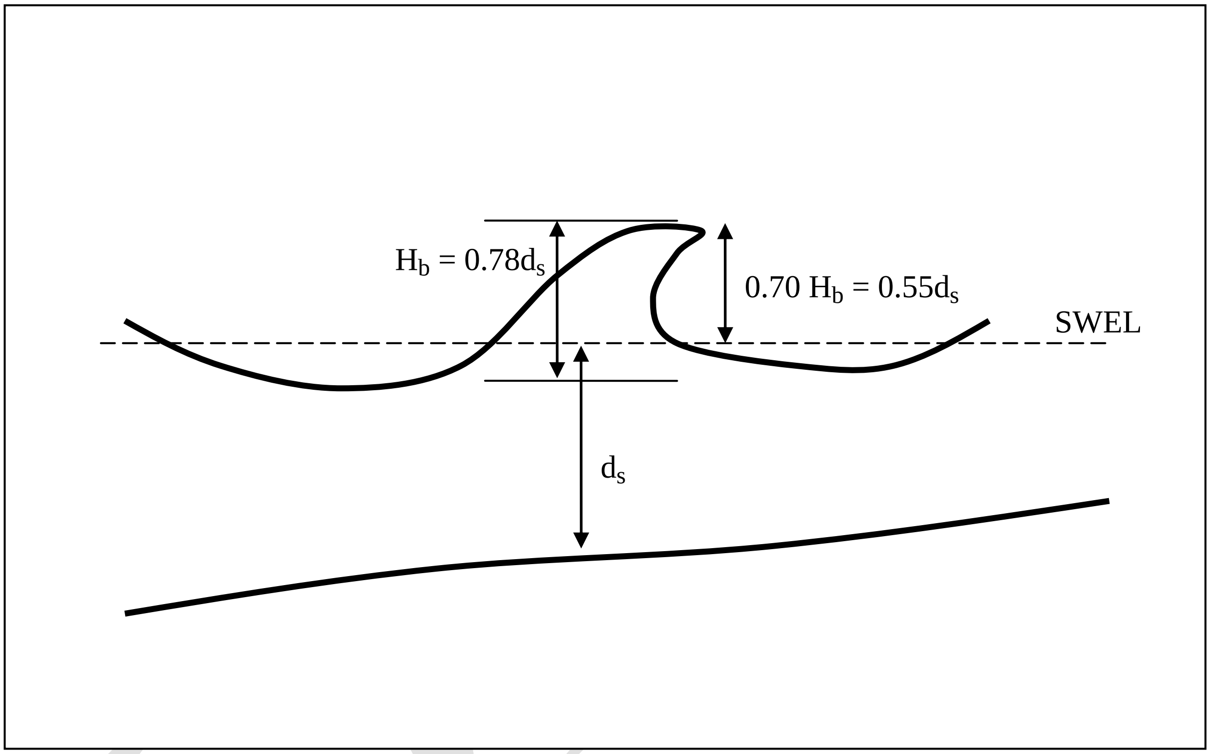


Figure D.3.6-1. WHAFIS relationships between local still water depth, ds, maximum breaking wave height, Hb, and wave crest elevation.

D.3.6.2 Transect Considerations

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

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

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

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

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

D.3.6.3 Water Level and Wave Input

An important consideration is the specification of input water-level and wave conditions for each transect. Within these guidelines, the overall approach is response-based for the flood mapping. However, for regions that are subjected to overland wave propagation, a hybrid approach that incorporates an event-based methodology is recommended.

Unlike the Atlantic and Gulf where a single event defined by 1-percent water level and coincident 1-percent wave height is considered representative of base flood conditions based on the assumption of highly correlated storm surge elevations and wave heights, in the Great Lakes it is necessary to consider multiple events that could result in the 1-percent overland wave propagation flood hazard. These multiple events are likely necessary because of the sometimes rapidly moving storm systems and the relative magnitudes of various contributors to storm surge in the Great Lakes rendering the assumption of elevated water level and wave height coincidence invalid. In addition, the Great Lakes analysis has the added complexity of a different lake level for each storm, so that a different method of WHAFIS application is required from that used in the Atlantic and Gulf.

The 1-percent-annual-chance event actually refers to a flooding event with an annual exceedance probability of 1 percent. In other words, there is a 1-percent chance of such event being equaled or exceeded within any given year. The wave crest elevation defines the BFE in areas where overland wave propagation is the dominant flood hazard. The 1-percent wave crest elevation is commonly going to correspond to very high water levels; however, the combination of wave and water level that creates this critical condition is usually unknown. For example, a high water level and moderate wave height condition might form the 1-percent wave crest elevation, or it could be the result of a slightly lower water level and very high wave height.

The recommended approach for evaluating overland wave propagation hazards with WHAFIS in the Great Lakes utilizes the joint probability method to compute the combination of wave and water-level conditions near the shoreline that are expected to generate the 1-percent-annual chance flood conditions. The method involves first calculating the *1*-percent-annual-chance-exceedance wave crest elevation based on a statistical analysis of the maximum wave crest elevations for all storms in the composite storm set. The same is done for the still water levels. Then a set of effective waves and water levels is defined that creates a *1*-percent chance wave crest elevation for the *1*-percent chance still water level, and that effective wave is transformed across the transect in order to facilitate mapping. Each set of wave heights and water levels will also need an erosion analysis performed to determine the input profile for WHAFIS.

The goal of the event-based approach is to use joint probability distributions in order to compute the limiting state that corresponds to the 1-percent wave crest elevation. The Mapping Partners can choose from a number of methods to compute the critical combination of parameters that generate the 1-percent wave crest elevation. One method is to use Monte Carlo sampling of the joint probability distributions. However, this would generally be too costly to perform in FIS studies.

The recommended method is a simpler joint probability approach (Nadal-Caraballo et al., 2012) that closely resembles the traditional event-based approach used in FIS studies and is as follows:

For wave and water-level model results just outside the surf zone:

> Compute the marginal probability distributions (e.g., GPD) of wave height, wave period, and water level.
>
> Using a bivariate distribution model, compute the joint probability surfaces between wave height and wave period, and between wave height and water level, respectively.

From the joint probability surface for water level and wave height, compute the iso-probability curve corresponding to the 1-percent joint exceedance probability (see Figure D.3.6-2).

$P(SWL \cap HHm0) = 1$ percent

Compute at least three parameter combinations along the iso-probability curve: maximum water level and associated wave height, maximum wave height and associated water level, and at least one combination with intermediate values.

From the water-level marginal distribution, compute the 1-percent annual exceedance water level and the expected value of wave height, E(Hm0) from the conditional probability distribution.

$P(Hm0 \mid SWL1\%)$

From the wave height marginal distribution, compute the 1-percent annual exceedance wave height and the expected value of water level, E(SWL), from the conditional probability distribution.

$P(SWL \mid Hm0{,}1\%)$

For all wave heights, compute the associated wave period as the expected value, E(Tp), from the conditional probability distribution between wave height and wave period.

$P(Tp \mid Hm0{,}1\%)$

Review resultant water-level and wave condition pairings and evaluate whether any pairing can be eliminated to reduce the number of WHAFIS runs necessary. Bases for eliminating pairings include approximate duplication of another pairing, a water level that does not inundate the profile, or some other situation that can be determined a priori to not produce the most hazardous overland wave hazard among the pairings.

Compute the eroded profile for the five or more statistical conditions from the previous steps.

Determine the WHAFIS input for the multiple wave, water level, conditions and eroded profile from step 1. This would typically require running a 1-D surf zone dynamics model to some point near the shoreline for all cases.

Run WHAFIS for all conditions and determine the limiting state that results in the 1-percent annual exceedance flood elevation (BFE).

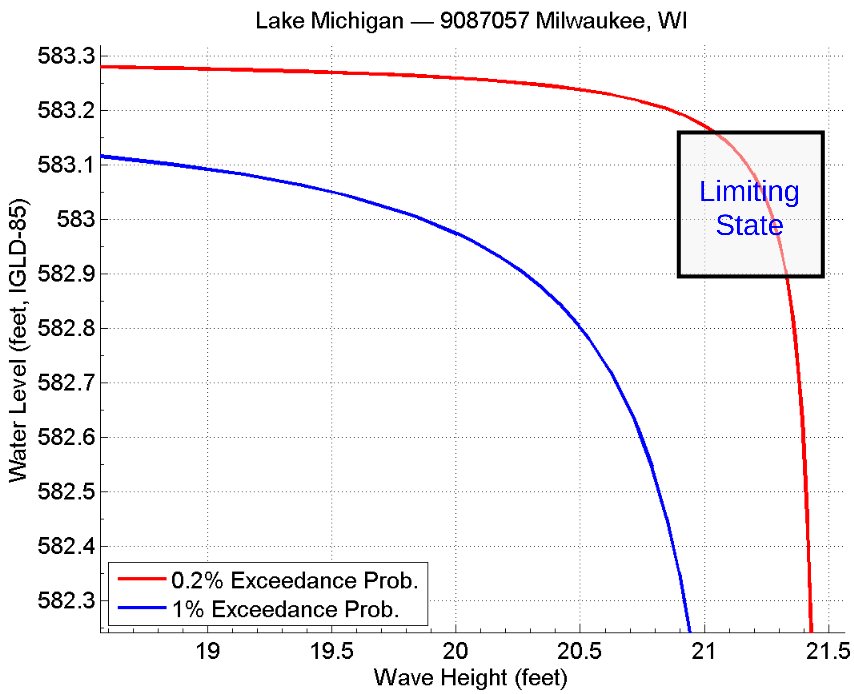


Figure D.3.6-2 Examples of iso-probability curves corresponding to 1- and 0.2-percent annual exceedance probabilities, respectively.

The significant wave height (energy-based H_{m0} or statistically-based H_s) is not directly used in the overland wave propagation analysis in Flood Insurance Studies, but rather the controlling wave height (H_c) is used. H_c is approximately equal to the average of the highest 1 percent of the waves. H_c is related to the significant wave height H_s by H_c = 1.6 Hs where H_s is by definition the average of the highest one-third of all the waves. In deepwater, H_{m0} is approximately the same as H_s, but in shallow water, H_{m0} is 10 to 15 percent smaller than the H_s. This difference in wave height definition must be accounted for where necessary. The process of developing wave and water-level input to the WHAFIS model must be thoroughly documented by the Mapping Partner.

If the Mapping Partner is using a stand-alone version of WHAFIS, the calculation of H_c must be performed by the user. However if WHAFIS is being used within CHAMP, no such adjustment is required. CHAMP converts the significant wave height to the controlling wave height for use in WHAFIS automatically so H_s may be specified directly.

D.3.6.4 Input Considerations

The Mapping Partner should be aware that mapping flood hazards for an area with multiple flood sources or a highly irregular shoreline may involve the mapping of WHAFIS results from multiple transects originating from the different flooding sources or shorelines. This scenario is most likely to occur where an island or peninsula is separated from the mainland by a bay large enough to generate large waves against the back side of the island, and where flooding and waves can strike the island from two directions. A complete analysis of this scenario requires the specification of transects, water levels and wave conditions at both shorelines, and multiple

WHAFIS analyses. At any point on the island, the highest water surface and wave heights from the analyses would control the flood mapping.

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

- Vegetation: WHAFIS has two separate routines for vegetation: One accounts for rigid vegetation that can be represented by an equivalent "stand" of equally spaced circular cylinders (NAS, 1977), and another accounts for marsh vegetation that is flexible and oscillates with wave action (FEMA, 1984). For either type, the Mapping Partner must exercise considerable care in selecting representative parameters and in ruling out the possibility that the vegetation will be intentionally removed or that effects would be markedly reduced during a storm through erosion, uprooting, or breakage. Details on coding vegetation are contained in Section D.2.7 of Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update (FEMA, 2007). Note that marsh vegetation parameters have been built into WHAFIS for characteristic marsh plants along the Atlantic and Gulf coasts, but not for the Great Lakes. The predominant marsh vegetation types in a study area should be identified and compared to default WHAFIS vegetation types to see if it can be properly represented with built-in marsh vegetation options. If the study site's vegetation parameters do not agree with any of the default options, the WHAFIS MG file can be edited to accommodate alternative marsh vegetation.
- Coastal Structures: See section 3.8 for treatment of coastal protection structures in a flood insurance study.
- Buildings: Buildings must be specified on the transect as rows perpendicular to the transect. Because buildings are not always situated in perfect rows, the Mapping Partner must exercise judgment to determine which buildings can be represented by a single row. The required input value for each row of buildings is the ratio of open space to total space. This is simply the sum of distances between buildings in a row, divided by the total length of that row. The Mapping Partner must examine the first several rows of buildings along the shoreline to determine whether they will be obstructions during the base flood – only large, fully engineered buildings with solid, nonbreakaway shearwalls, deep beams, or other horizontal structural elements extending below the BFE should be considered obstructions. It is useful to contact local officials to obtain construction information and the lowest floor elevations of structures before coding buildings as obstructions. If buildings are elevated above the base flood wave crest on pilings, columns, or other open foundations, waves will propagate under the structures with minimal reduction in height. The mapping partner should code these buildings using the BU card (see Section D.2.7) and indicate 100-percent open space. This procedure acknowledges the presence of the pile-elevated buildings and allows others to see that the buildings were considered in the analysis, but recognizes that the presence of the open-

foundation buildings will not lead to wave height reductions or flood insurance risk zone changes.

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

The current version of WHAFIS allows the user to account for wave regeneration over flooded areas, using either a user specified wind speed for the overwater fetch (OF) or inland fetch (IF) transect codes. The Mapping Partner should consult existing local historical wind data or wind data developed during the lakewide modeling of storm surge and waves to derive realistic estimates of wind conditions during the conditions being modeled. If wave regeneration is not a significant issue, or if significant reduction in wind by vegetation canopy is expected, wind input to WHAFIS can be neglected and the transect analysis can be treated as a wave propagation situation only. The process of developing wind input to the WHAFIS model must be thoroughly documented by the Mapping Partner.

The Atlantic and Gulf Coast guidelines provide additional details on the operation of the WHAFIS model including input preparation, operation and model output. Please refer to Section D.2.7 Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update (FEMA, 2007) for more information.

D.3.6.5 Documentation

The Mapping Partner must document all assumptions used to define input waves for WHAFIS analyses, including a brief description of offshore wave conditions, and a description of wave transformation, attenuation or dissipation between the wave source and the shoreline. In sheltered waters, this must include a summary of fetch determination, winds (speeds, directions, and duration), and bathymetry used in hindcasts. The documentation must include the approximations or assumptions used in the analysis. When observational data, such as wave buoy data, are available, the wave height, period, and spectral parameters should be compared to the predicted waves. Documentation should include any field observations or measurements, as well as available historical or anecdotal information regarding overland wave propagation during flooding events.

See Section D.3.10 for additional documentation considerations.

D.3.7 Coastal Erosion

This section reviews erosion processes for the different shoreline types found throughout the Great Lakes. Based on the historical response of these various shore types to storm erosion, guidance is provided for the appropriate approach to consider erosion when evaluating flooding risks in the Great Lakes Basin.

D.3.7.1 Erosion Processes in the Great Lakes

Erosion processes and consequences of erosion can either be "episodic" or "chronic." These two descriptors assign a very important temporal component to erosion processes and their results. *Episodic Erosion* is the shore and backshore adjustment that results from short-duration, high-intensity storm events. This type of event response results in shore adjustment and occurs during a single storm or during a series of closely spaced storm events within a storm season. Shore and backshore profile changes during intense storms can result in dramatic beach and dune erosion, breaching of barrier beaches, or the complete removal of backshore dunes. Bank and high bluff sites can also fail during episodic events, however, the physical processes that cause these type of failures are complex and include a number of variables that are not typically included in a FEMA Coastal FIS investigation, such as details of ground water table, slope stability, soil moisture context, and rainfall (as discussed in Section D.3.7.2.5). *Chronic Erosion* is associated with gradual shoreline adjustments caused by slow, long-term processes such as: (1) erosion forces exceeding the resisting properties of the soils (often consolidated glacial sediments), (2) gradients in longshore sediment transport, (3) lake-level cycles (transition from low to high lake levels), (4) land subsidence, (5) changes in sediment supply due to watershed modifications, dam building, or construction of coastal structures, and (6) decadal-scale changes in wave climate and ice cover associated with climate change.

Current FEMA regulations are limited to risks and losses occurring as the direct result of a severe storm event (episodic erosion). The NFIP does not address long-term chronic erosion, but focuses on episodic, flood-related erosion due to severe coastal storm events.[5] FEMA does not currently map long-term erosion hazard areas. Therefore, the erosion assessment guidelines in this section only include methods for estimating erosion of shore and backshore areas during single, large storm events.

Prior to determining wave runup elevations and overland wave propagation, the Mapping Partner must determine whether the profile being analyzed will be significantly altered during a storm event. Along many Great Lakes shores, storm-induced erosion can change the location and alter the form of an existing shoreline barrier that extends above and below the still water level. Mapping Partners must assess the likely erosion and resultant eroded profile in conjunction with determining flood effects, where erosion influences overland wave propagation, wave runup and overtopping.

[5] Discussions of long-term erosion and the potential consequences of chronic erosion are found in materials listed in the reference section of this document and in many of the support documents referenced herein.

For Great Lakes shores, predicting storm or episodic erosion is subject to various complex and interrelated factors, including the following:

1. Coastal counties can have large variations in wave climate due to the shoreline geometry/orientation and exposure to the prevailing winds. Although water levels determine what part of the profile will erode during an individual storm, wave energy determines the amount of erosion for the episodic event;

2. Erosion is greatly influenced by the mean lake level and by storm surge. Mean lake levels show oscillations, both seasonally and over decades, and the storm surge magnitude varies widely among study sites and with different storms;

3. Late fall, winter and early spring storm winds, surges, and waves are generally the most extreme on the lakes. Ice cover is a complicating factor and it has shown extreme variability in the last decade;

4. Sizeable longshore bars (often multiple bars) are a prevalent feature along sandy shores in the Great Lakes and they modify the wave energy reaching the beach and backshore, which in turn influences runup and overtopping potential. In addition, the volume of sand stored on the beach can vary with lake level cycles and cross-shore transport processes;

5. Previous lake levels (antecedent conditions) can affect erosion susceptibility for a site because of the time lag occurring between lake level change and the resulting beach response (accretion or erosion). Periods of low lake level can lead to the development of wide beaches, berms and small foredunes that will likely survive or afford protection during the next period of high lake level;

6. Beach erodibility at a given site can vary dramatically, as different types of mobile sediment and consolidated materials (e.g. glacial tills) can become exposed during a storm (Dewberry and Davis, 1995). In some locations the beach is sandy, and in other locations a small mobile lens of sand lies over a cohesive sediment substrate that is more erosion-resistant at the storm-event time scale. Some beaches are characterized as mostly cobble or rock, and are typically stable during storms; and

D.3.7.2 Shore Types and Erosion Assessment

Profile erosion and adjustment during storm events is influenced by the type of shoreline being analyzed. The International Joint Commission (IJC) has developed a comprehensive classification that identifies the primary geomorphic shore types found within the Great Lakes (Baird, 2006). For the purpose of evaluating erosion processes for FIS studies across the Great Lakes region, eight generalized shore types are listed below:

- Sandy beaches with dunes and barrier beaches (erosion modeling required)

- Mixed/coarse sediment beaches (erosion estimates may be required)
- Artificial beaches and accretion deposits (erosion modeling likely required, site by site decision)
- Eroding sand bank (erosion estimates may be required, site by site decision)
- Eroding cohesive bank/bluff (no erosion modeling required)
- Consolidated bedrock shores (no erosion modeling required)
- Non-eroding bedrock shores (no erosion modeling required)
- Coastal Wetlands (no erosion modeling required)

The following sections provide a general overview of the geomorphic conditions and erosion processes for these eight general shore types. In particular, guidance is provided on whether erosion modeling is required for FEMA flood studies. This guidance should not supersede historic information, engineering judgment, or local knowledge of a study site.

D.3.7.2.1 Sandy Beaches with Dunes and Barrier Beaches

Sandy beaches and barrier beaches are a common geomorphic shore type found throughout the Great Lakes. Refer to Figure D.3.7-1. These sand beaches are dynamic and respond to fluctuating lake levels and storm events. They are often backed by frontal sand dunes. The dune is defined by relatively steep slopes abutting markedly flatter and lower regions on each side.

Barrier beaches are also found throughout all of the Great Lakes, often sheltering embayments and drowned river valleys from lake waves. These barriers respond dynamically to sediment supply, significant storm events and periods of high water levels. If erosion occurs during a storm event, significant waves can propagate inland and contribute to flooding. Historical breaches should be reviewed in counties with barrier beaches.

The primary factors controlling beach and dune erosion on sandy shorelines are the mean lake level and magnitude of the storm surge, the width and crest height of the beach, size and volume of the dune which controls overtopping and potential for a breach during a storm, sediment grain size, and the wave height and duration during the event. The Mapping Partner should evaluate the long-term stability of the beach, dune and/or barrier systems, including whether the features are eroding, stable or accreting. If erosion of the beach and dune is a concern from the standpoint of flooding, the individual storm response can be modeled using a 1D cross-shore sediment transport model, such as CSHORE. Application of the model can be used to examine whether or not erosion of the beach and dune is likely for each of the storms evaluated.

Figure D.3.7-1 Eastern Lake Ontario Sandy Beach and Dune with Backshore Development

D.3.7.2.2 Mixed/Coarse Sediment Beaches

Mixed or coarse sediment beaches often occur in the Great Lakes when the eroding shore materials have a high concentration of cobbles and pebbles in the glacial sediment (e.g. glacial outwash). Finer sediments such as clays, silts and sands are transported alongshore and offshore, leaving the cobbles and pebbles behind. A picture of a mixed sediment beach is presented in Figure D.3.7-2. A conceptual sketch is also provided (Figure D.3.7-3), highlighting the potentially complex nature of the sediment stratigraphy at these sites.

Figure D.3.7-2. Eastern Lake Ontario Cobble/Shingle Beach

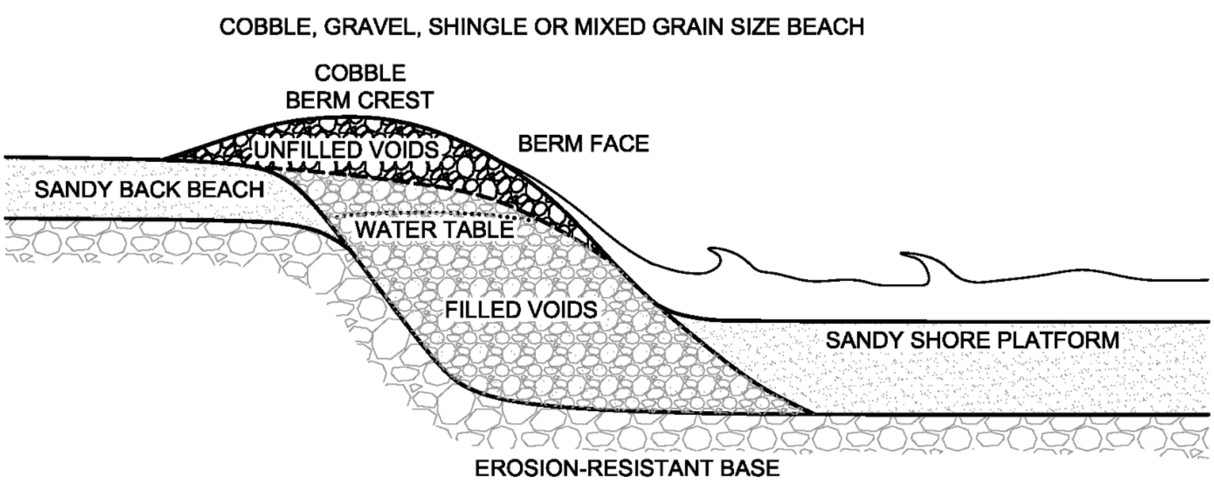


Figure D.3.7-3. Conceptual Sketch of Mixed Sediment Beach

The Mapping Contractor should review historical shoreline change data and/or collect field observations for the mixed sediment beaches to evaluate if they erode significantly during individual storm events. If these beaches are stable, no erosion modeling is required. Conversely, if the beach is dynamic and responds significantly to storm events at various lake levels, the erosion potential should be considered for the response evaluation of individual storms.

D.3.7.2.3 Artificial Beaches and Accretion Deposits

Engineering structures have significantly modified large portions of the Great Lakes shoreline, especially in urban areas. A sample of a large waterfront project that included construction of land-based infrastructure, a new boat launch, offshore breakwaters and beach nourishment is presented in Figure D.3.7-4. The pocket beaches have been very stable for the last 20 years. However, event driven erosion can still alter beach slope and depth for artificial beaches, which in turn influences the flood risks for individual storms. Therefore, storm erosion modeling should be performed when evaluating flood hazards at artificial beaches throughout the Great Lakes. However, if the modeling shows the beaches are stable during storm events, then the Mapping Partner can rely on engineering judgment to determine if all of the events in the composite storm database need to be simulated.

Figure D.3.7-4. Lake Forest Park, North of Chicago, Illinois

Fillet beaches accumulate adjacent to coastal structures, such as harbor and ports, and natural headlands. In general, these beaches are dynamically stable, as the structure holds the sediment in place and limits the potential for the sediment transport alongshore. A history of shoreline accretion at Michigan City since 1834, which is one of the largest fillet beaches in the Great Lakes, is summarized in Figure D.3.7-5. Although the overall trend for these beaches is stable or accreting, storm events can still alter beach width and slope, which in turn can influence the runup analysis and flood risk mapping. Therefore, storm erosion modeling is also recommended for fillet beaches when analyzing flood hazards and engineering judgment should be used to evaluate the number of simulations to be completed based on a site by site analysis.

Figure D.3.7-5. History of Fillet Beach Growth at Michigan City, Indiana

D.3.7.2.4 Eroding Sand Bank

Sandy glacial outwash deposits are a common geomorphic feature in the Great Lakes Basin, deposited during the last ice retreat approximately 10,000 years ago. When these sand sheets were deposited at elevations significantly higher than the present chart datum on each lake, an eroding sand bank often develops along the shore. Refer to the example in Figure D.3.7-6, for Shoreham on the eastern shore of Lake Michigan. Wave attack erodes the sand toe during severe storms at high lake levels and the bank recedes. The amount of retreat is typically small for individual storm events and detailed numerical modeling may not be required when evaluating wave runup. However, the Mapping Partner should review historical shoreline change rates within the county and make a site specific assessment.

Figure D.3.7-6. Shoreham Example of an Eroding Sand Bank (toe of the bank is protected in distance)

In some instances, these eroding sand banks can fail dramatically due to a combination of factors, including heavy rainfall events, unusually high ground water tables and soil moisture levels, wave attack during high lake levels, surcharging of the bank crest (e.g., home construction), and vegetation clearly (to mention a few). It is beyond the scope of a typical FIS to investigate and predict this type of complex slope failure. However, the Mapping Partner is encouraged to identify historical events and mention the potential risk to riparian landowners from slope failures in the FIS report.

Figure D.3.7-7. Massive Slope Failure, St. Glenn Shores, Michigan

D.3.7.2.5 Eroding Cohesive Bank/Bluff

Due to the high percentage of consolidated glacial sediment in the Great Lakes Basin, eroding cohesive banks are a common geomorphic feature. A typical eroding bluff in Wayne County, Lake Ontario is provided in Figure D.3.7-8. When the lakebed consists of consolidated glacial sediment (lacustrine clay or glacial till), erosion and lakebed downcutting is a slow process that is attributed to softening of the surface layer of sediment and erosion due to wave orbital velocities and breaking waves. The banks also erode and retreat landward due to a combination of wave attack at the toe and slope stability factors, such as ground water flows. Typically, bluff recession rates ranges from 1 to 3 feet/year in the Great Lakes Basin. Erosion attributed to any one storm has only minor impacts on the amount of lakebed downcutting and bluff retreat (Baird, 2011). Therefore, in most cases, the Mapping Partner can ignore erosion processes for eroding cohesive banks when evaluating wave runup and overland wave propagation with WHAFIS.

Figure D.3.7-8. Eroding Cohesive Bank, Wayne County, Lake Ontario South Shore

When the cohesive banks feature complex stratigraphy, including alternating bands of impermeable lacustrine clay and sandy lenses, perched water tables may develop well above the mean lake level. These complex groundwater and geologic conditions can lead to large rotational failures and slumps. An example of a large rotational failure on Lake Michigan following the high lake level conditions in the late 1990's is presented in Figure D.3.7-9. Saturated groundwater conditions and large rainfall events in combination with storm events can trigger these large rotational failures. Although these large slope failures can modify the beach and bluff profile, the typical data collected for an FIS does not generate sufficient information to predict these rotational failures, as they are also extremely intermittent. Therefore, it is recommended to document the potential risk of a large rotational failure in the FIS but not to account for it in the flood hazard analysis.

Figure D.3.7-9. Rotational Slope Failure in 1998, 107th Street Allegan County, Michigan

D.3.7.2.6 Consolidated Bedrock Shores

Although bedrock is the foundation of the entire Great Lakes Basin, it is often buried by thick glacial sediments and sand deposits. In some locations, the consolidated bedrock is exposed at the shoreline and on the lake bottom. Typically these shores feature weak mudstones, limestone or shale. A wave-cut terrace or shelf often forms in the nearshore and a steep bank or bluff will develop at the back of the beach. Although these consolidated bedrock shorelines can feature a small long-term recession rate, it is typically very low (a few inches per year) and is not significantly influenced by individual storm events (Episodic Events). Rather erosion is a slow gradual process. As such, in most cases, the Mapping Partner can ignore erosion processes for these consolidated bedrock shores when evaluating flood hazards.

D.3.7.2.7 Non-Eroding Bedrock Shores

Portions of the Great Lakes shorelines are characterized by non-eroding bedrock, such as metamorphic and igneous rocks. This hard rock is very resistant to storm induced erosion and does not erode measurably in a 20 or 30 year period, the typically lifespan of a FEMA FIRM. Therefore erosion assessments are not required for this shore type.

D.3.7.2.8 Open Coast Wetlands

In locations where the regional shoreline orientation shelters the water's edge from large waves generated on the lake, open coastal wetlands will develop if the soil stratigraphy is favorable. These open coast wetlands have typically developed in the embayments and drowned river valleys of the Great Lakes, such as Saginaw Bay on Lake Huron (Figure D.3.7-10). These shorelines typically don't erode as the wave energy is low and the deposition of fine grained material (silts and clays) is the more typical trend. During storm events these wetlands are typically submerged and are not subject to direct wave attack that would induce erosion of the lake bottom. The marsh vegetation also protects the shoreline and upland from erosion. Unless otherwise indicated by historical information, episodic erosion analysis is not necessary for wetlands.

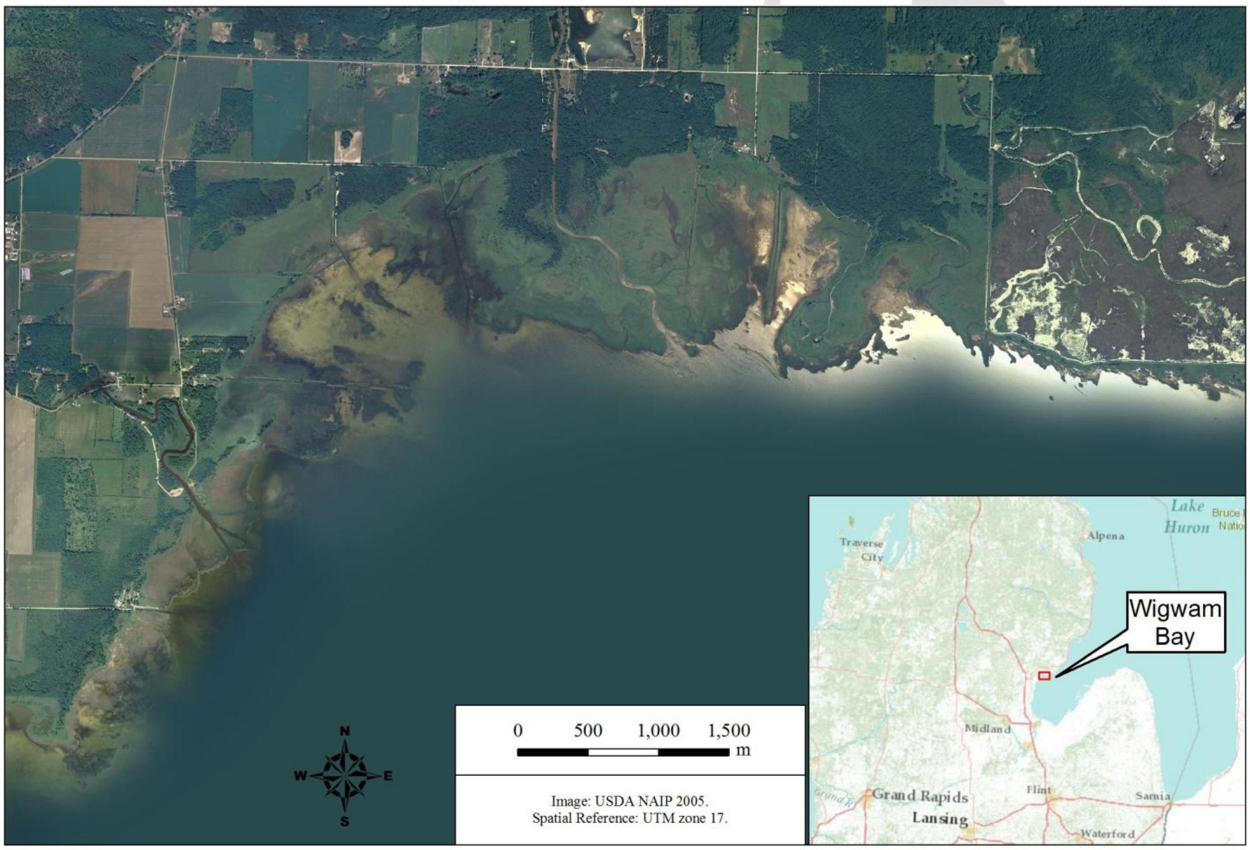


Figure D.3.7-10. Open Coast Wetlands in Wigwam Bay, Saginaw Bay, Lake Huron

D.3.7.3 Erosion Assessment Methods

Use of a 1-D cross-shore hydrodynamic and sediment transport model is recommended for use in estimating beach and dune erosion. A 1-D surf zone dynamics model can simultaneously predict the wave field, water circulation, sediment transport, and nearshore morphology change along a transect, accepting time varying water levels, storm surge and incident wave conditions as input along with an initial beach profile and information about sediment grain size. Use of a 1-D dynamics model is considered a more robust approach that produces more accurate estimates of profile response as compared to simple geometric methods, such as the method previously recommended for use in the Great Lakes by these Guidelines, which have inherent limitations (notably no consideration of storm duration and other known important beach factors such as grain size and site geology).

Johnson (2012) conducted testing of the FEMA geometric erosion method for several sites on Lake Michigan. The investigation examined the appropriateness of the methodology for general use on the Great Lakes. The erosion results obtained using the FEMA method where compared with those calculated using the 1-D surf zone dynamics model CSHORE (Koyabashi 2009, and Johnson et al. 2012). Johnson et al. (2012) describe extensive testing of the CSHORE model's applicability to simulate storm-induced beach erosion. This transect model was found to be robust and efficient, with computation times on the order of several seconds. These findings are consistent with the results from cross-shore erosion modeling completed with SBEACH and the COSMOS model for various dune sites in the Great Lakes for a recent FEMA Pilot Study investigation (Baird, 2011).

D.3.7.3.1 Profile Geometry and Estimating Sediment Grain Size

Geometric models for dune erosion are simple to apply and have minimal data input requirements. For example, the simple geometric method previously recommended by FEMA (FEMA, 2003), is readily used with no requirement for specifying sediment size. Process-based models for nearshore morphology, on the other hand, can provide predictions that are more realistic, but have greater requirements for model initialization (input parameters). Most required data are routine and readily available. The bottom position including bathymetry and topography for the Great Lakes, for instance, is now available from high-resolution LIDAR data in most locations. Likewise, storm water levels and waves are supplied from lake-scale models. However, knowledge of the lakebed substrate (e.g. sand versus bedrock) and sediment grain size is also required for the detailed morphology change models.

In the absence of detailed field data on the lakebed geology, an analysis of profile geometry can be used to estimate lakebed substrate type and transitions from mobile sand and gravel deposits to hard bottom (e.g. bedrock) or consolidated sediment (e.g. glacial till). For example, refer to a map of the lakebed geology for Eastern Lake Ontario in Figure D.3.7-11 (Woodrow et al, 2002) produced from a detailed shallow seismic survey. Stony Point in the north is a bedrock headland (as noted with the hatch pattern), with sand limited to a thin veneer in nearshore bars close to shore. The central portion of the site features a large thick sand deposit. The southern end of the site features exposures of glacial till on the lake bottom. Offshore, laminated silts and clays are present.

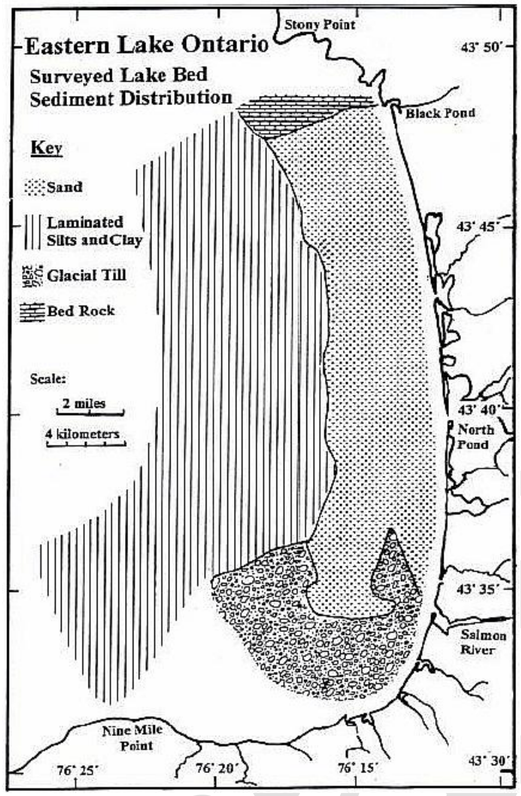


Figure D.3.7-11. Lakebed Substrate for Eastern Lake Ontario Site (Woodrow et al, 2002)

Detailed LIDAR data was collected in 2001 for this area and used to extract the lakebed profiles in Figure D.3.7-12. Profiles 619 and 622 are from the northern bedrock region (just south of Stony Point). They are relatively steep (1:90 V:H) and consistent in their morphology. Profiles 628 and 629 are from the sand region and feature sand bars, are very homogeneous out to a depth of 10 m below Chart Datum, and very flat (1:170 V:H) due to the fine-grained nature of the sand. The glacial till profiles from the southern region of the site are very steep in the nearshore (0 to 100 m on the x-axis), then highly irregular out to a depth of 10 m, which is consistent for an eroding glacial till lakebed in the Great Lakes. The features in deeper water that resemble large sand bars are actually exposures of harder sediment or self-armored cobble-lag deposits. In summary, analyzing the morphology of the lakebed profiles can provide insight into the surficial geology based on unique characteristics for the different sediment types typically found in the Great Lakes. Further, these changes can also help identify the extent of mobile sand deposits (which erode during storms in the models) and hard bottom (such as bedrock) that doesn't erode during storms.

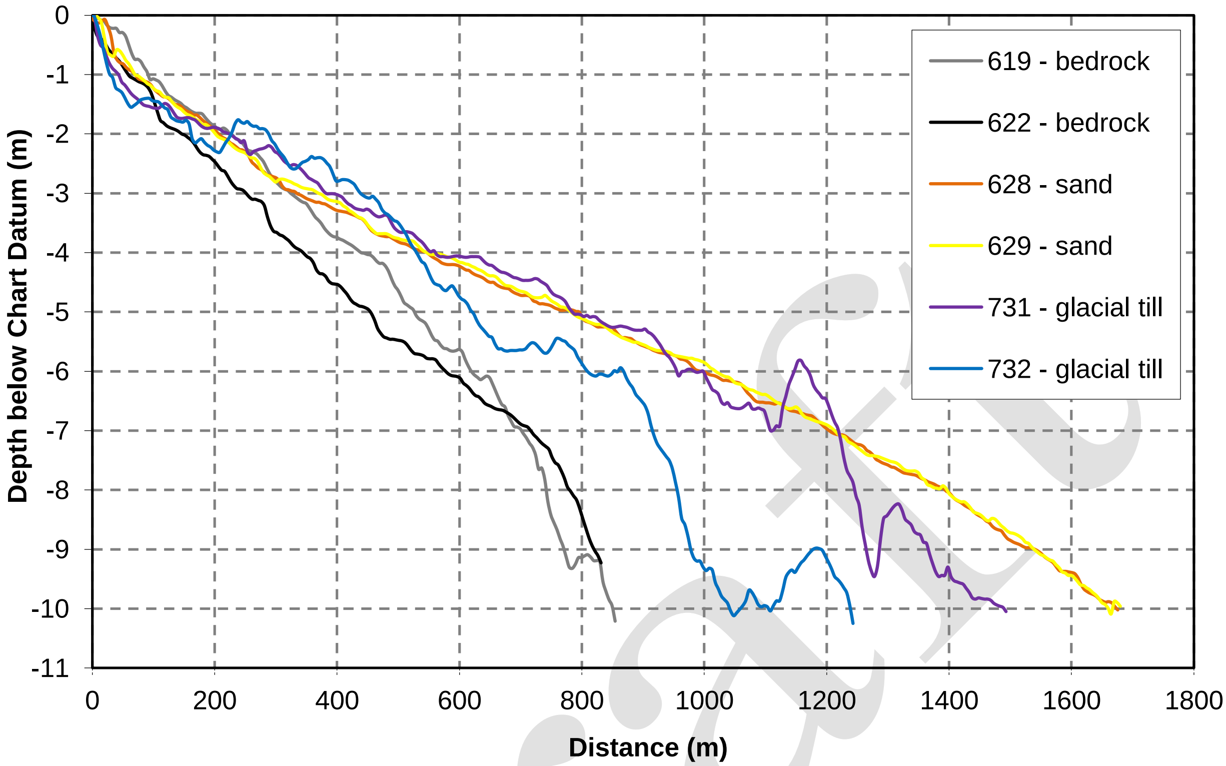


Figure D.3.7-12. Eastern Lake Ontario LIDAR Bathymetry Profiles (200x)

It is also necessary to have information on nearshore and beach sediment grain size for modeling. In the absence of field data, alternative methods may be required for the FIS investigations. The equilibrium beach concept has been used extensively to describe profile shapes over nearshore regions with a wide variation in sediment characteristics. Analyses of many beaches (e.g. Dean 1977) have indicated the applicably of a simple expression for the subaerial profile:

$$d = Ax^{2/3} \qquad (1)$$

where *d* is the water depth, *A* is a shape parameter, and *x* is a cross-shore coordinate, positive offshore with the origin at the still-water shoreline. Dean (1991) provided the theoretical basis for the concave profile shape, Eqn (1), based on the assumptions of linear saturated waves and uniform energy dissipation. Applicability, therefore, is limited to the active surf zone. Available profile data can be used to determine the optimal shape parameter through an error minimization. Consider a single transect comprised of equally spaced discrete points extending from the still water shoreline to the edge of the surf zone. An analysis minimizing the root-mean-squared error between data and the analytical equilibrium beach yields an estimate for the shape parameter

$$A = \frac{\overline{d_i x_i^{2/3}}}{\overline{x_i^{4/3}}} \qquad (2)$$

where the over-line depicts averaging across all points in the surf zone.

In general, it is noted that smaller sand sizes are associated with mildly sloping beaches and a smaller shape parameter. Empirical relations between the shape parameter and sediment characteristics have been developed, and the most widely-cited expressions indirectly relate A to the sediment size through the fall velocity w_f. Dean (1991), for instance, proposed

$$A = 0.067 w_f^{0.44} \quad (3)$$

where the units for A and w_f are $m^{1/3}$ and *cm/s* respectively. On the other hand, Kriebel et al. (1991) proposed

$$A = 2.25 (w_f^2 / g)^{1/3} \quad (4)$$

which is valid for any units. The difference between the two formulas for A is less than 30 percent for sands with w_f = 1–10 cm/s.

Equations that relate the fall speed of natural sediments and grain size are written as explicit expressions for w_f and are not, in general, easily inverted. For example, one widely-used expression due to Soulsby (1997) is given as

$$w_f = \frac{\nu}{d}\left(\sqrt{10.36^2 + 1.049 d^3 g \frac{s-1}{\nu^2}} - 10.36\right) \quad (5)$$

where ν is the kinematic viscosity, d is the grain diameter, g is the acceleration of gravity, and s is the sediment specific gravity. Equation (5) is readily solved for the fall velocity with a given sediment diameter. Solving the inverse relation, however, requires an iterative method for determining d.

Simplified Approach

A practical and accurate method for grain size determination can be developed by approximating the relations with a fitted curve. Figure D.3.7-13 depicts the exact relationship of A and sediment size, making use of (4) and an iterative solution of (5). Also shown is an explicit empirical polynomial curve for sediment size

$$d_{mm} = 14.21A^4 + 57.24A^3 - 10.47A^2 + 2.25A + 0.01 \quad (6)$$

where d_{mm} is the sediment diameter with units of *mm*. Equation (6) is easily applied to determine the characteristic sediment grain size when an optimized shape parameter is determined from measured data. No significant error is introduced by using the provided empirical relationship, but the application should be limited to $A < 0.3\ m^{1/3}$ to remain within the fitted domain.

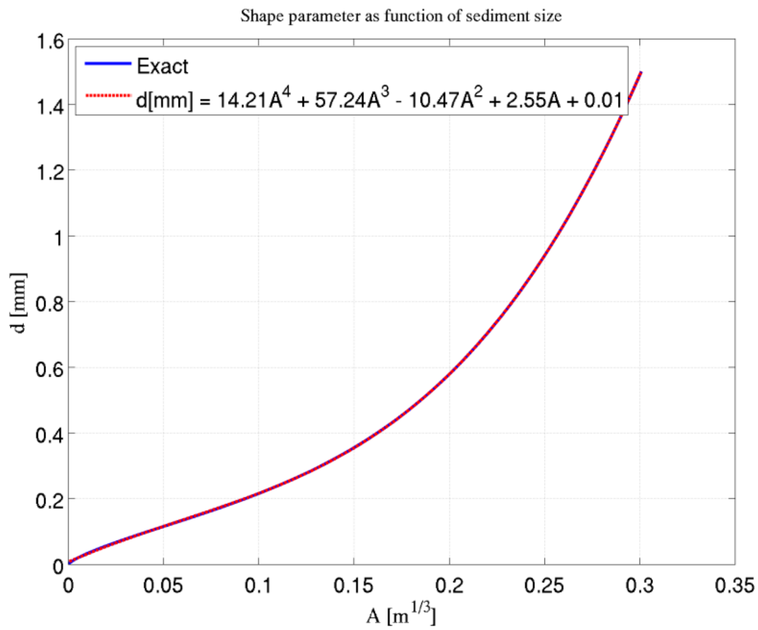


Figure D.3.7-13 Exact and approximate relationship between shape parameter A and characteristic sediment size (solid blue line and dashed red line are equal)

Example Application

A demonstration of this procedure is provided herein, where measured profile data are used to estimate the sediment grain size. North Dunes Nature Preserve is located on Lake Michigan near Zion, IL. The beach profiles in the region are characterized by a large dune and moderately sloping beaches. The measured data are available as a LIDAR data set for Lake Michigan from 2008 US Army Corps of Engineers (USACE) National Coastal Mapping Program Topobathy Lidar: Lake Michigan. Figure D.3.7-14 shows the measured data and the optimized equilibrium profile as a solid blue line. Also depicted are the position of the stillwater-shoreline and the seaward extent of the estimated surf zone as red dots. An active surf zone was assumed for the profile between the breaker line at a depth of 5 m below Chart Datum and the shoreline in this analysis. Application of (2) to find *A* and (6) results in a d=0.2 *mm*. Note that from 300 to 700 m on the x-axis in Figure D.3.7-14, the unusual shape of the lake bottom indicates the substrate changes from sand to glacial till (or another more erosion resident sediment type). Therefore, it would not be appropriate to apply the equilibrium profile concept for this deeper portion of the profile.

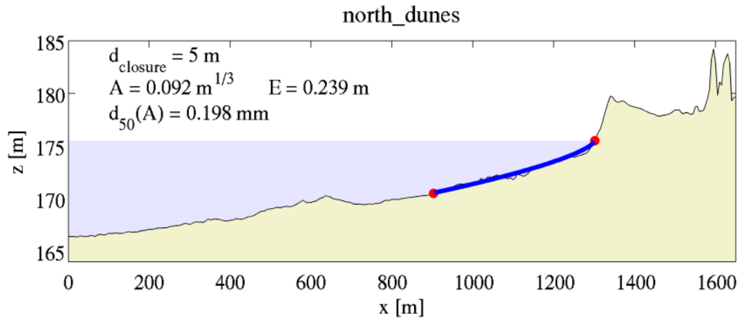


Figure D.3.7-14: Example determination of grain size from measured profile data

D.3.7.3.2 Beach Morphology Change in Response to Lake Level Cycles

The Response approach to establish BFEs for runup- and overtopping-dominated profiles requires the quantification of flooding for a large number of historic storms across the full range of recorded lake levels, as documented in Section D.3.3.2.2. For the sandy shores described in Sections 3.7.2.1 and 3.7.2.2, significant changes in the beach and dune conditions can occur during periods of high to low lake levels. For example, refer to the sandy beach conditions for a site in Berrien County, along the southeast shore of Lake Michigan. In 1985, the beach was completely eroded during a high lake level period, and a vertical wall defined the water's edge. Twenty-four years later in 2009, following a prolonged period of falling lake levels, the beach width had increased by approximately 200 feet. Some of the beach width increase can be attributed to a 2.3-foot drop in lake levels. but as seen in the 2009 aerial photograph, the vertical wall is completely buried and a foredune has now been established lakeward of the treeline. In other words, the overall volume of sediment stored in this beach deposit has also increased.

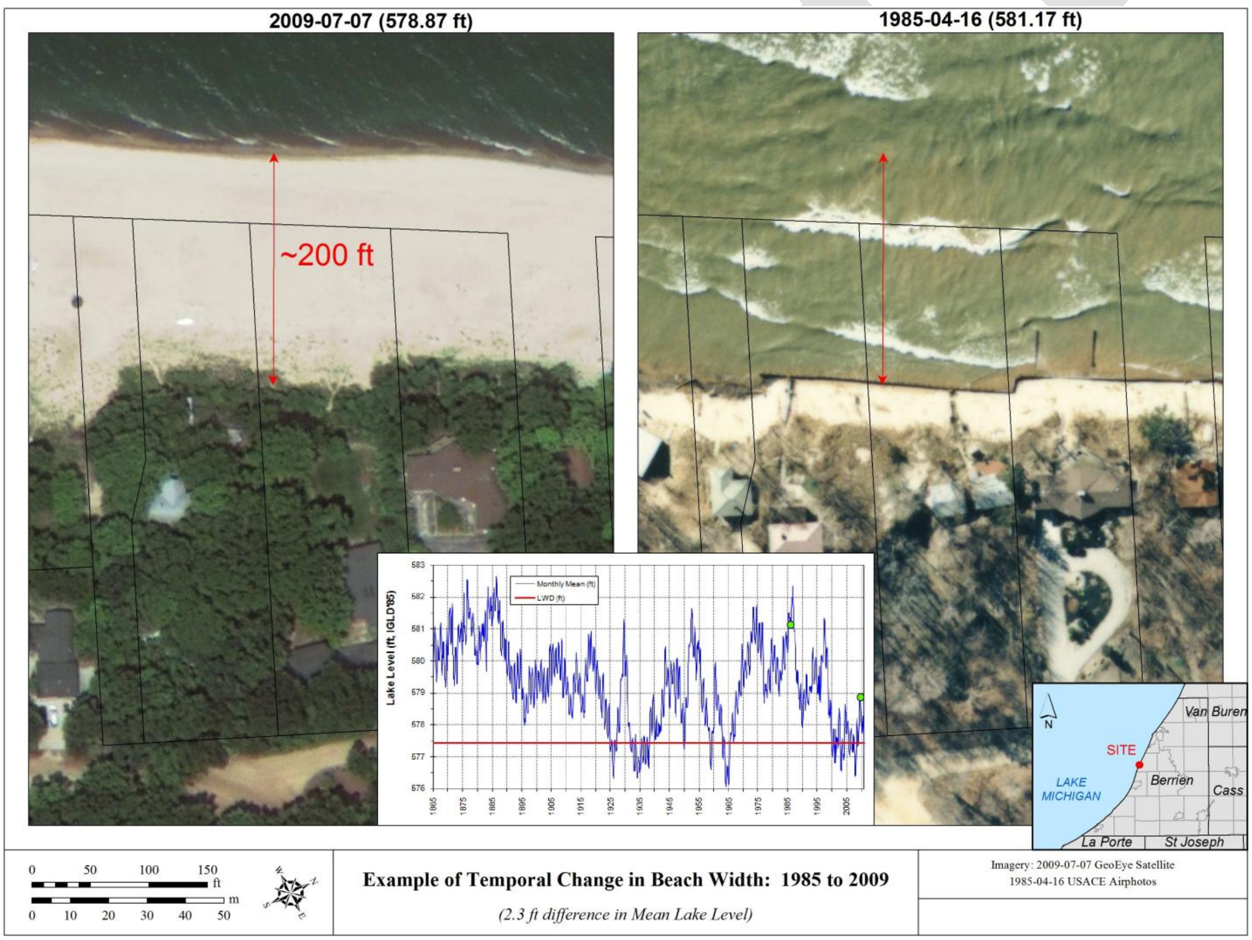


Figure D.3.7-15. Temporal Change in Beach Position for Berrien County Site

When investigating the individual flood response for storm events, the Mapping Partner should investigate the degree of profile change that has occurred historically due to fluctuating lake levels. The type of dramatic changes shown in Figure D.3.7-15 will be limited to sandy beaches and in some cases, mixed sediment beaches. Due to a general lack of mobile coarse grained sediment (sand and gravel) for cohesive and bedrock shorelines, these changes in beach width are not anticipated for most of the bank/bluff sites.

The reliable prediction of storm morphology change with modeling tools and ultimately flood level is dependent on accurate model initialization. An accurate representation of the bathymetry and topography, for example, is of primary importance. Although LIDAR data or acoustic surveys are available for the Great Lakes region, much of the high resolution data collection has occurred during the last decade, when lake levels have been well below long-term average conditions. Therefore, in the sand-dominated regions of the lakes, the beaches and dunes have recovered from the high water conditions that occurred in the 1970s and 1980's, as seen in Figure D.3.7-15. The appropriateness of low water bathymetry and topography for investigating historical storm response during high lake level periods has not been extensively tested and requires investigation on a case by case basis.

For sites where the data collection campaign was conducted at a lake level that is similar to the level required in modeling the historical storms for the Response approach, the data can likely be used without modification. However, if the lake level for the historical storm is significantly different from the conditions during the data collection, it may be necessary to modify the morphology of the bathymetry and beach conditions on the profile before runup calculations are completed. As seen in Figure D.3.1-3, the lake level in Lake Michigan can vary by several feet per year and by 6 feet historically.

For cases where lake level changes are significant, it is advisable to consider alterations to the bathymetry and beach volume used for model initialization. Although advances in process-based modeling have been significant in the last two decades, cross-shore sediment transport models are poorly suited to make long-term predictions, such as multiple years of morphology change. It is therefore advised to use methods based on simple mass balance relationship if changes to the beach and lake bottom position are required. The Bruun Rule (Bruun 1962), for instance, could be used to estimate the lake level difference from the data collection period to the actual storm being simulated.

For example, a profile volume tool was developed for the USACE Detroit District as part of the Lake Michigan Potential Damages Study (Baird, 2003) that integrated the general theories of the Bruun Rule to modify sandy beach profiles for conditions of rising or falling lake levels. The tool requires the following inputs for a beach profile that extends lakeward to the depth of closure (typically 8 to 10 m below Low Water Datum in the Great Lakes):

1. Lake level when the bathymetry was collected;
2. New lake level for the modified profile;
3. Depth of closure for sand and gravel (given as a distance offshore);
4. Toe of dune or back of beach (given as a distance); and
5. Toe of beach expressed as a distance (typically the trough before the first bar).

There are two unique aspects of this tool that make it suitable for modifying beach profile morphology in the Great Lakes. First, the elevation of the backshore dune is integrated into the overall volume solution, so sites that feature large coastal dunes are more resilient to beach erosion during periods of rising lake levels compared to sites with small dunes. Refer to Figure D.3.7-16 for an example of an estimated profile (New Depth in the legend) for a 1.27 m rise in lake level. The beach and dune face erode, with sand transferred to the nearshore bars, which migrate onshore and increase in elevation.

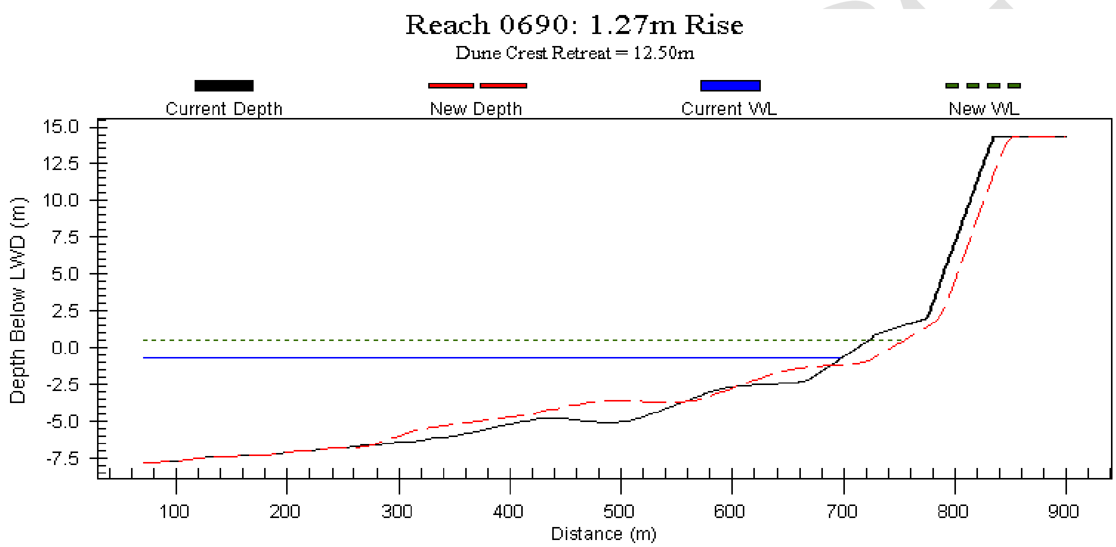


Figure D.3.7-16. Initial and Adjusted Profile Morphology for a 1.27 m (4.2 ft) Rise in Lake Levels

The second scenario is falling lake levels. Refer to Figure D.3.7-17 for an example of the new beach profile for a 0.6-m fall in lake level. In this example, the dune remains stable, the bar elevation decreases and moves offshore, and the beach width increases at the waterline.

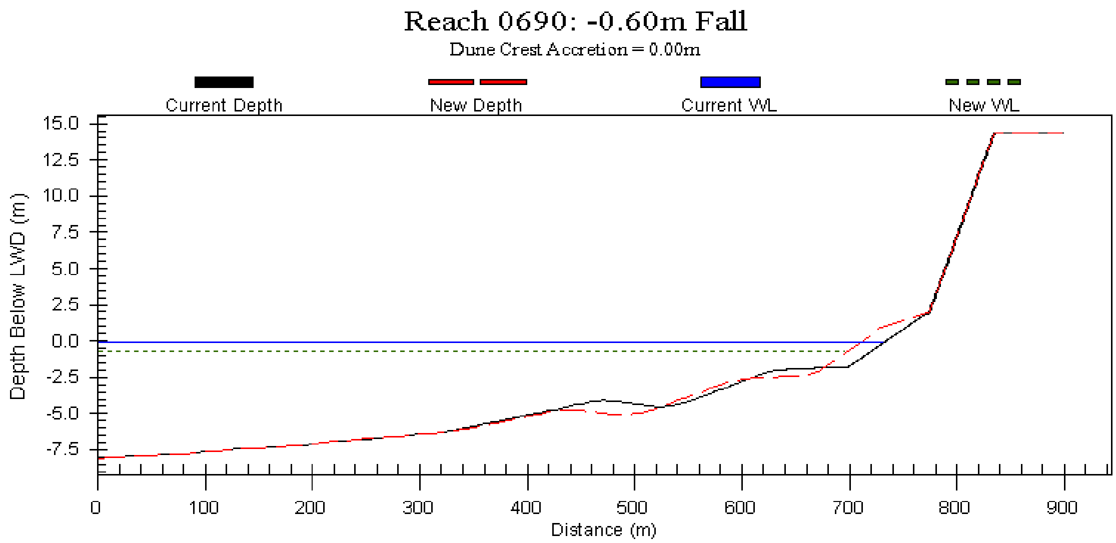


Figure D.3.7-17. Initial and Adjusted Profile Morphology for a 0.60 m (2 ft) Fall in Lake Levels

The Mapping Partner will investigate the potential need for beach profile adjustments during the Response investigation when evaluating flood risks. It may be necessary to apply a profile volume tool similar to the examples presented in Figures D.3.7-16 and D.3.7-17.

D.3.8 Coastal Structures

This section provides guidance for certifying coastal protection structures for use in the NFIP and outlines methods for analyzing the stability and effects of coastal structures during 1-percent-annual-chance flood conditions

D.3.8.1 Purpose and Overview

Because coastal structures can significantly affect local topography and flood hazards, the evaluation of coastal structures is a necessary part of any flood hazard study. The evaluation should, where possible, determine whether a coastal structure will survive the 1-percent-annual-chance flood and provide protection to upland areas.

- If a particular structure is expected to remain intact through the 1-percent-annual-chance flood, the structure geometry shall be used in all ensuing FIS analyses that accompany the flood event (e.g., event-based erosion, wave runup and overtopping, and determination of wave crest elevations).
- If a particular structure is expected to fail during the 1-percent-annual-chance flood, the coastal structure shall either be removed entirely before ensuing analyses, or be replaced by an appropriate failed configuration before ensuing analyses.
- If the performance of a particular structure is uncertain, both intact and failed configurations should be analyzed, and the most hazardous flood conditions should be mapped.

For the purposes of this appendix, coastal structures are classified as follows:

- **Coastal Armoring Structures:** Generally shore-parallel structures constructed to prevent erosion of uplands and mitigate coastal flood effects (e.g., seawalls, revetments, bulkheads, and levees). Please note that coastal levees are classified as armoring structures here, but are often referred to as flood control structures.
- **Beach Stabilization Structures:** Structures intended to stabilize or reduce erosion of the beach, which, by doing so, afford some protection to upland areas (e.g., groins, breakwaters, sills, and reefs).
- **Miscellaneous Structures:** Structures not included above that can affect flood hazards, especially in sheltered waters (e.g., piers, port and navigation structures, bridges, and culverts.

Criteria for evaluating the stability and performance of coastal armoring structures for FIS purposes are well developed and are discussed in detail. Criteria for evaluating beach stabilization structures have not been developed yet, and only basic guidance is provided.

Criteria for evaluating miscellaneous structures are not standardized, and only basic guidance is provided.

D.3.8.2 Evaluation Criteria

Mapping Partners are not required to perform detailed engineering evaluations of all coastal structures within the study area, and, in fact, rarely do so. However, when such an evaluation is performed, specific evaluation criteria must be applied.

D.3.8.2.1 Detailed Engineering Evaluation of Coastal Armoring Structures

Specific criteria for evaluating coastal armoring structures are contained in an April 23, 1990, FEMA memorandum (FEMA, 1990), *Criteria for Evaluating Coastal Flood Protection Structures for National Flood Insurance Program Purposes*.[6] The evaluation criteria from the 1990 memorandum are provided in the *Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update* (FEMA, 2007).

D.3.8.2.2 Coastal Armoring Structure Evaluation Based on Limited Data and Engineering Judgment

For the purposes of an FIS, the Mapping Partner may not have sufficient resources and time to conduct a detailed evaluation of each coastal armoring structure within the study area. In such cases, the Mapping Partner can apply engineering judgment (guided by the FEMA memorandum and USACE CERC Technical Report 89-15, *Criteria for Evaluating Coastal Flood Protections Structures*) to determine the likely stability of each structure during the 1-percent-annual-chance flood. These conclusions may be based largely on available archive information and local observations, including historic evidence of storm damage and maintenance. Note that any data and procedures used in the evaluations shall be documented, and communities and property owners shall be made aware that these evaluations are for mapping purposes only.

If the available information does not clearly point to survival or failure of a coastal structure, the Mapping Partner may either:

1. Conduct a detailed evaluation based on the FEMA criteria (see the previous section).
2. Perform the erosion and wave analyses for both the intact and failed structure cases and map the flood hazards associated with the more hazardous case.

If Option 2 is selected, the Mapping Partner shall clearly document the results of all cases investigated and specify which case is used for mapping purposes. It should be noted that a failed coastal structure may or may not yield the greatest flood hazards. Therefore, coastal flood analyses for both intact and failed conditions should be performed, with the greatest resulting

[6] The criteria discussed in this memorandum are based in large part on Technical Report 89-15, *Criteria for Evaluating Coastal Flood-Protection Structures* (Walton et al., 1989), prepared by the U.S. Army Corps of Engineers, Coastal Engineering Research Center (USACE CERC) for FEMA. The criteria in the memorandum have been adopted as the basis for NFIP accreditation of new or proposed coastal structures to reduce the flood hazard areas and elevations designated on NFIP maps, but they can be applied to existing coastal structures.

flood hazard being mapped. Maintaining results of all analyses may be useful in the event map revisions are requested by property owners based upon certified structures[7].

D.3.8.2.3 Evaluation of Beach Stabilization Structures

Guidance on how to predict the survival or failure of groins, which usually fail by loss of profile (through settlement, displacement, or deterioration) and/or by becoming detached at their landward ends, is not readily available. Guidance on how to predict the failure of breakwaters, sills, and reefs (usually through loss of profile) is not readily available either. Some information on failure modes may be available in technical or historical literature, and these should be consulted by the Mapping Partner.

If a Mapping Partner chooses to evaluate beach stabilization structures during an FIS, the proposed evaluation methods and procedures should be discussed with the FEMA Study Representative, in advance, and approval by FEMA must be obtained before the evaluations are be carried out.

D.3.8.3 FIS Treatment of Coastal Armoring Structures

Technical Report 89-15 identifies four primary functional types of coastal flood protection structures: gravity seawalls, pile-supported seawalls, anchored bulkheads, and dikes or levees. The first three of these are shown below in D.3.8-1a. The fourth and fifth types are shown in Figure D.3.8-1b.

Technical Report 89-15 recommends as a general policy that "FEMA not consider anchored bulkheads as providing flood protection during large storms." Thus, the default assessment should be that open-coast anchored bulkheads are assumed to fail during the 1-percent-annual-chance flood. Mapping Partners may choose to treat some anchored bulkheads as surviving the flood and/or providing some degree of flood protection, but those instances should be limited (e.g., to sheltered waters, where the bulkhead may be stable during 1-percent-annual-chance flood conditions).

Many seawalls and revetments and some bulkheads may be recognized on flood hazard maps if analysis based on the detailed evaluation criteria in *Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update* (FEMA, 2007) shows they will remain intact during the 1-percent-annual-chance storm (in some cases, even if overtopped). These structures may provide total or limited protection against flooding, erosion, and waves, depending upon their location, strength, and dimensions.

D.3.8.3.1 Failure and Removal of Coastal Armoring Structures

In the event that a coastal structure is determined to fail, the Mapping Partner shall remove the structure entirely from the analysis transect, or estimate the partial collapse of the structures where appropriate (see Section D.3.8.3.2). If the failed structure is removed entirely, the remaining soil profile should be altered to achieve its likely slope immediately after structure

[7] Often, property owners request revisions to the FIRM based upon existing, new, or proposed coastal structures. Map revisions based on coastal structures require a detailed evaluation and certification by a professional engineer registered in the subject State. FEMA has distributed the *Coastal Structure Form* (MT-2 Form 5, available at http://www.fema.gov/pdf/fhm/mt2_f5.pdf) to evaluate coastal structures as the basis for map revisions.

failure. Information on slopes behind failed structures is limited. These slopes may vary from 1:100 (v:h) for unconsolidated sands, to 1:1 or steeper for consolidated material landward of the failed structure. The post-failure slope used for analysis should be based on available data and engineering judgment where possible. In the absence of detailed engineering analysis, the slopes adopted should be in the range of 1:1 to 1:1.5 (v:h). The Mapping Partner may propose the use of a different slope based upon field data or engineering analysis, but the value used must be approved by the FEMA Study Representative. Note that the post-failure slope may not necessarily match the long-term stable slope, but will serve as the basis for subsequent site-specific erosion, wave height, wave runup, and wave overtopping analyses.

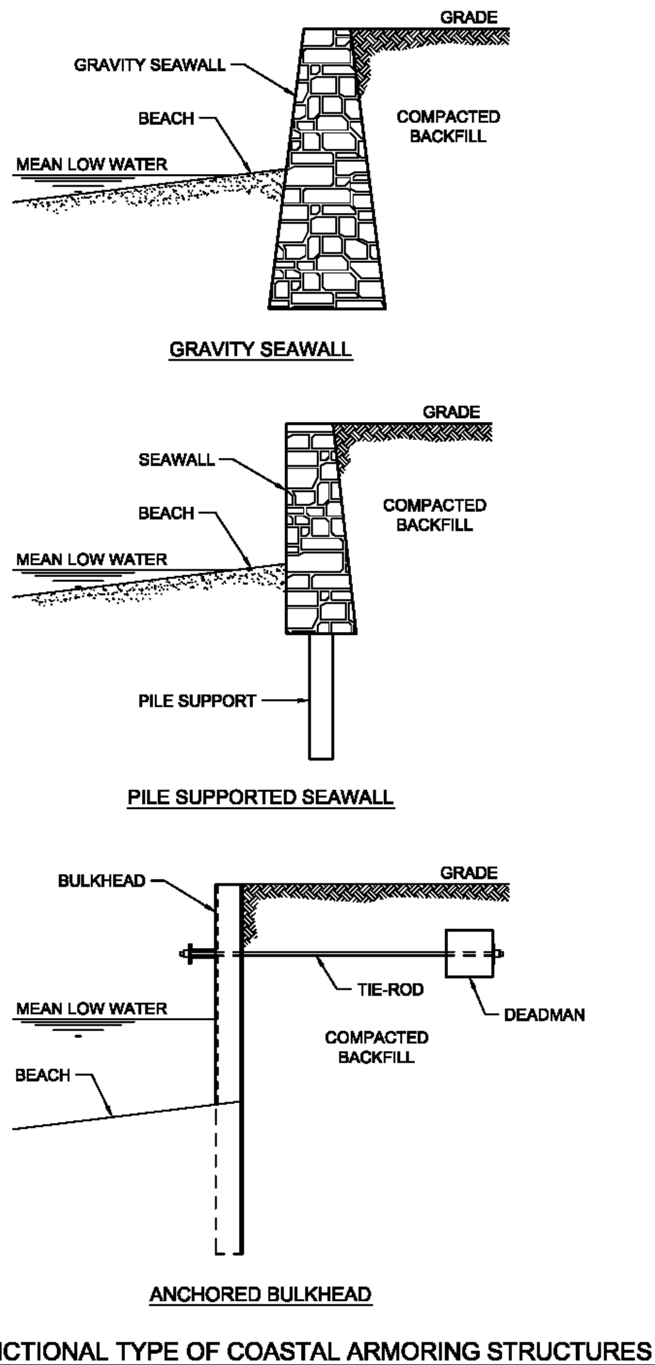


Figure D.3.8-1a. General Classification of Coastal Armoring Structures

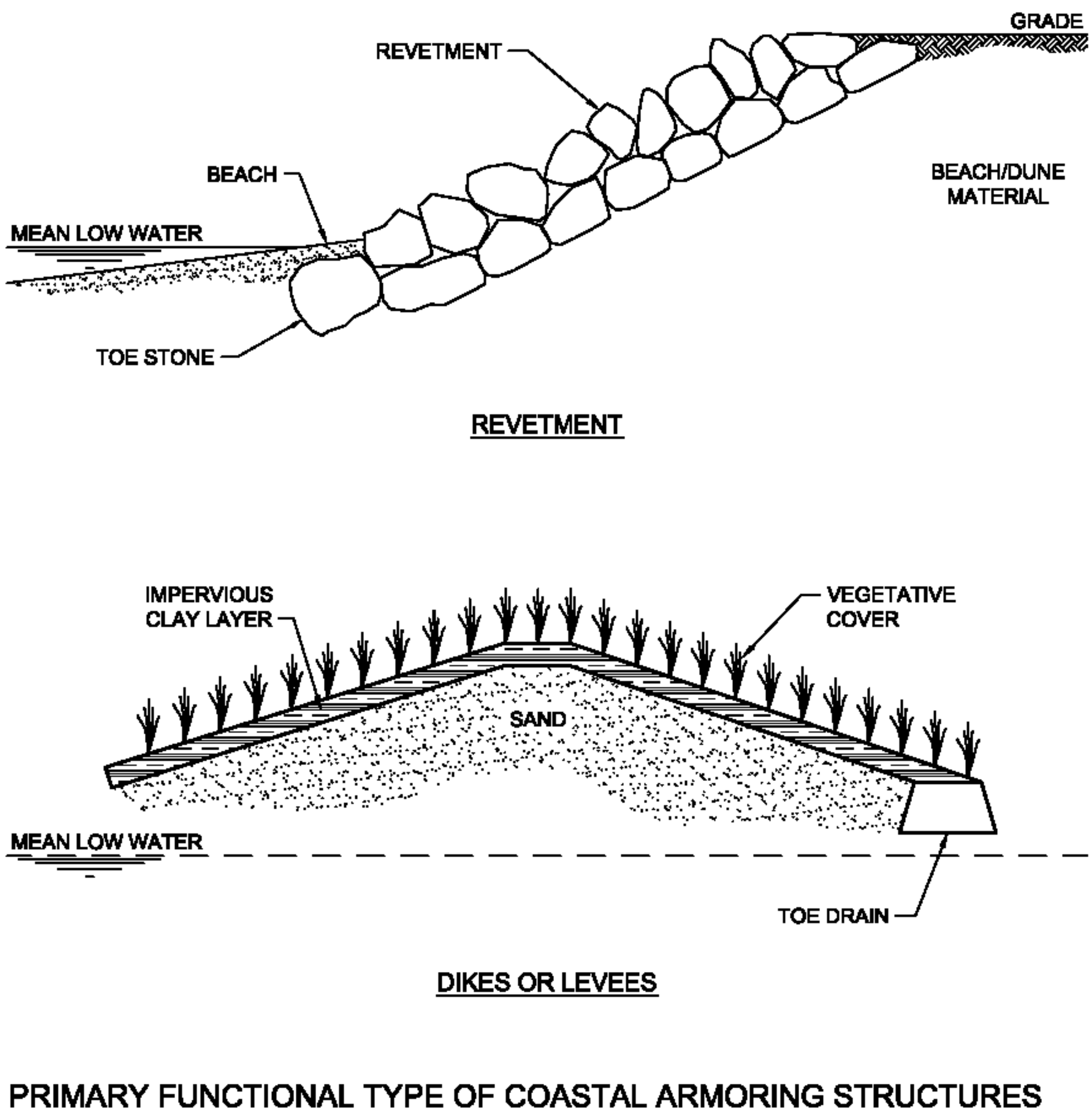


Figure D.3.8-1b. General Classification of Coastal Armoring Structures

D.3.8.3.2 Partial Failure of Coastal Armoring Structures

Coastal structures are frequently constructed of either concrete or large individual armor units. Consequently, it is improbable that the structural components will be completely destroyed or removed from the vicinity during the 1-percent-annual-chance flood. It may be appropriate to assume partial failure of such structures and to model accordingly.

A recommended simple geometric approach for approximating partial failure of a vertical or near-vertical coastal armoring structure is as follows:

1. Estimate toe scour at the subject structure based upon the methods described in the CEM (USACE, 2003).
2. Assume the structure fails and falls into a rough, porous slope at 1:1.5 (v:h).
3. Extend the 1:1.5 failure slope from the depth of scour at the structure toe landward to the point where it intersects the existing grade.

Figure D.3.8-2 provides a graphical depiction of this treatment.

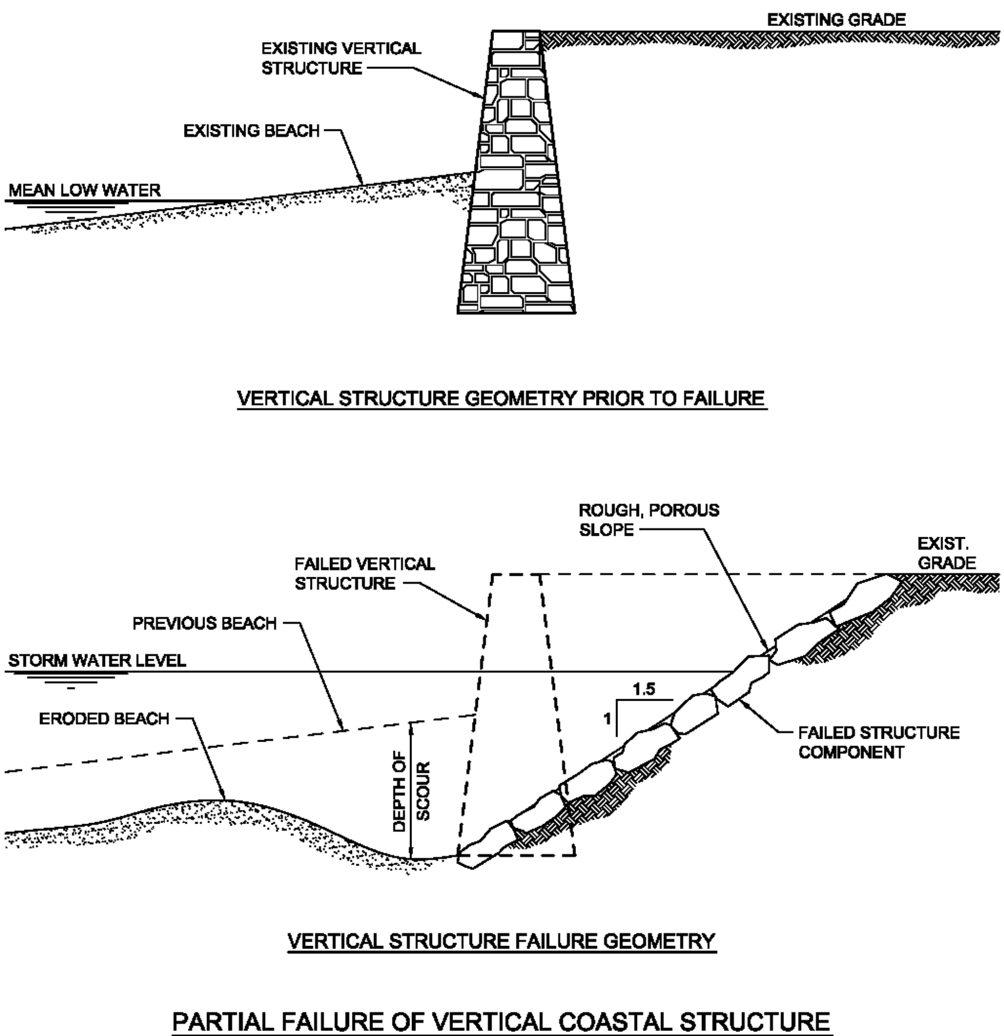


Figure D.3.8-2. Partial Failure of Vertical Coastal Structure

A recommended approach for approximating partial failure of a sloping revetment (due to undermining at the toe, or to collapse at the top due to erosion behind the structure) is as follows:

1. Assume scour at the base of the structure is equal to the depth of the armor layer.
2. Assume the structure will collapse in place into a triangular section throughout the structure footprint, with side slopes equal to the original structure slope.
3. Assume the landward side of the failed configuration will be half exposed and half buried. Approximate the soil slope landward from the failed structure at a slope in the range of 1:1 to 1:1.5 (v:h).

After determining an appropriate failure configuration as shown in Figure D.3.8-2, the Mapping Partner shall conduct overland wave height propagation (Section D.3.6) and wave runup (Section D.3.5) analyses for the failed structure, as discussed in preceding sections. The Mapping Partner shall select an appropriate roughness factor when conducting runup and overtopping analyses on the failed structure.

In some cases, the assumed failed slope may result in the undermining of buildings landward of the coastal structure. If this occurs, the building shall be removed from the analysis transect and not considered during subsequent wave-effects modeling.

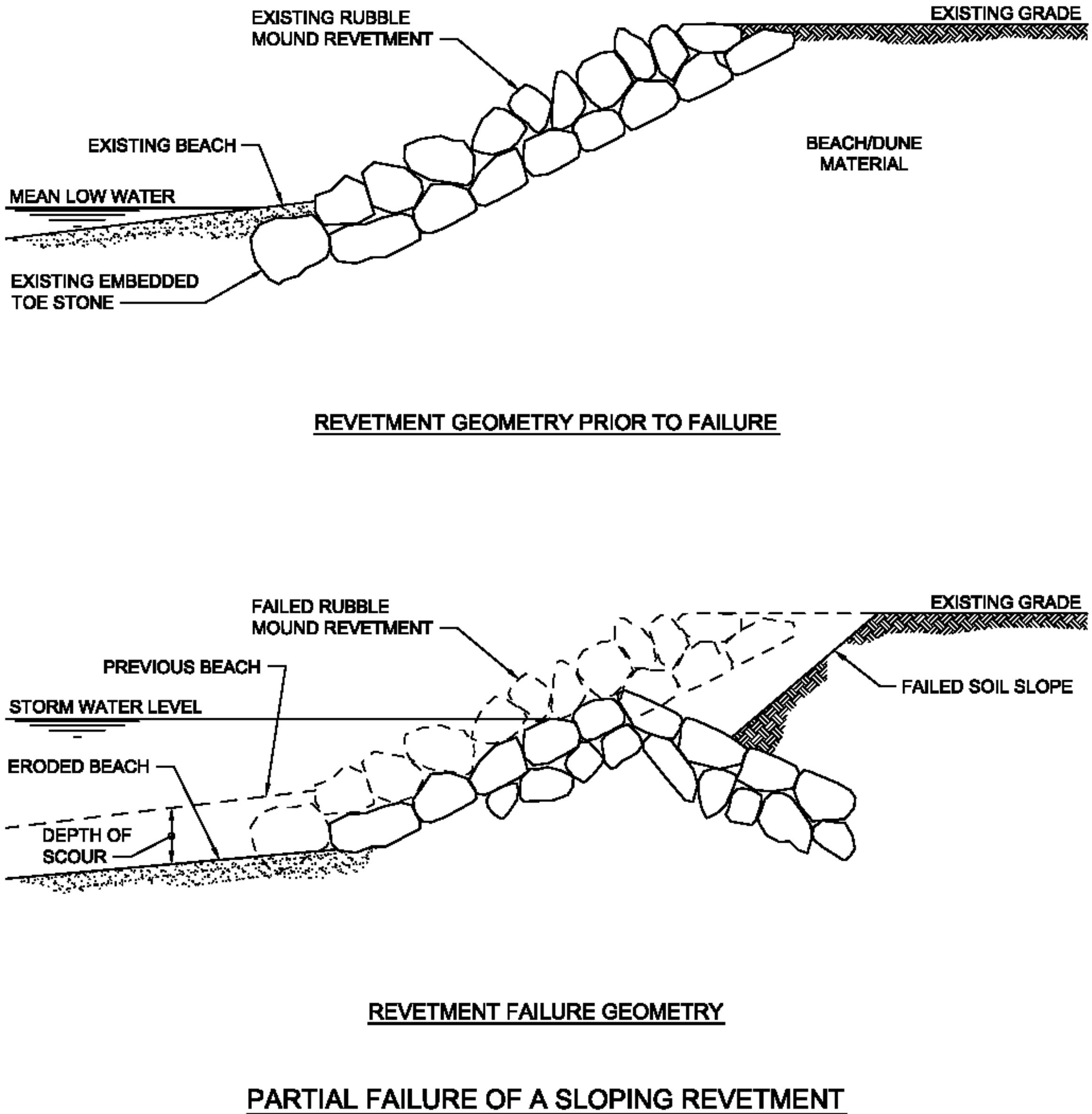


Figure D.3.8-3. Partial Failure of a Sloping Revetment

D.3.8.3.3 Buried Coastal Structures

In some instances, coastal structures may be covered or buried by sediments and not readily observable during an FIS site reconnaissance. Some buried structures are of a size and construction to possibly affect coastal flood hazards, and should—like exposed structures—be considered during the FIS. The Mapping Partner is responsible for determining whether buried coastal structures exist within the study area during the preliminary investigation phase of the

FIS. The Mapping Partner should include information from the community and carefully review aerial photographs of the study area to locate buried structures. For detailed guidance on evaluating buried structures for an FIS, see *Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update* (FEMA, 2007).

D.3.8.3.4 Coastal Levees

Levees are manmade structures (usually earthen embankments that may or may not have their slopes and crest armored) that prevent flooding of low-lying areas. A levee system consists of a levee, several levees, or a floodwall and the associated structures, such as closure and drainage devices, that are constructed and operated to prevent flooding of interior areas. FEMA has issued guidance on levees in Procedure Memorandum No. 34 (PM 34) *Interim Guidance for Studies including Levees,* dated August 22, 2005. The Mapping Partner should consult PM 34 for guidance in any new study or revision in which a levee structure influences the BFEs or hazard mapping.

For coastal levees or levee systems to be recognized as providing protection against the base flood by the NFIP and incorporated into flood hazard maps, they must be designed, constructed, operated, and maintained to resist erosion and prevent any flooding or wave overtopping landward of the levee crest during 1-percent-annual-chance flood conditions. The levee or levee system also must be certified as providing that level of protection. NFIP regulations (44 CFR Part 65.10) detail the requirements for a levee to be recognized as providing protection from flooding.

D.3.8.4 FIS Treatment of Miscellaneous Structures

Current FEMA guidance does not address the effects of miscellaneous structures (e.g., piers, port and navigation structures, bridges, culverts, etc.) on coastal flood hazard analysis and mapping. This section provides general guidance for identifying and analyzing the effects of miscellaneous structures on flooding in sheltered water areas as follows:

1. The Mapping Partner shall identify structures, in addition to the coastal armoring and beach stabilization structures addressed above, that could exert a significant influence on nearshore waves and currents, coastal sediment transport, or ponding in backshore areas during 1-percent-annual-chance flood conditions, particularly in sheltered waters. This should be done during the FIS reconnaissance phase.

2. Once identified, the Mapping Partner shall use historical evidence, other readily available data, and engineering judgment to determine whether the miscellaneous structures are likely to survive the 1-percent-annual-chance flood conditions. If the structures are likely to fail, then they (and their effects on the shoreline and flooding) should be removed from subsequent analyses.

3. The Mapping Partner shall notify the FEMA Study Representative as to how he/she intends to address miscellaneous structures and their effects during the FIS analyses, and obtain FEMA concurrence before proceeding.

D.3.8.4.1 Piers, Navigation Structures, and Port Facilities

The Mapping Partner shall review navigation charts, aerial photographs, and other information relative to piers, navigation structures, and port facilities (including dredged channels) that may affect the propagation and transformation or dissipation of waves within a sheltered water body, or that may affect littoral sediment transport. The Mapping Partner shall consider the range of possible effects of these structures and facilities during 1-percent-annual-chance flood conditions, using readily available data and site characteristics as a guide.

The Mapping Partner shall verify basic structure and facility information with local agencies and communities to determine the location, extent, and influence of these features. If there is any uncertainty concerning major features and their potential effects on upland flood hazards, limited field surveys or additional data collection shall be considered to augment existing data.

D.3.8.4.2 Roads, Bridges, Culverts, Etc.

The shorelines of sheltered waters are often paralleled by roads and railroads in backshore areas. The Mapping Partner shall consider the presence and influence of roadways, railways, embankments and abutment fill, and bridge piers on flood hazards during 1-percent-annual-chance flood conditions.

The Mapping Partner shall identify the location and condition of culverts and other flow-control structures in the vicinity of the study site and evaluate their potential to affect flood elevations. Design calculations and reports for individual culverts and storm drainage master plans for larger drainage systems shall be obtained and reviewed by the Mapping Partner to understand design criteria and provide data for hydraulic calculations and hazard zone delineation.

D.3.9 Mapping of Flood Insurance Risk Zones and Base Flood Elevations

This section provides guidance on the delineation of coastal flood insurance risk zones and BFEs.

D.3.9.1 Review and Evaluation of Basic Results

Before mapping the flood elevations and flood insurance risk zones, the Mapping Partner should review results from the models and assessments from a common-sense viewpoint and compare them to available historical flood data. When using models, there is the potential to forget that transects represent real shorelines being subjected to high water, waves, and winds. Familiarity and experience with the coastal area being modeled, or with similar areas, should provide an idea of a “reasonable” result.

The main point to be emphasized is that the results should not be blindly accepted. There are many uncertainties and variables in coastal processes during an extreme flood, and many possible adjustments to methodologies for treating such an event. The validity of any model is demonstrated by its success in reproducing recorded events. Therefore, the model results must be in basic agreement with past flooding patterns, and historical data must be used to evaluate these results.

It would be very convenient if data from a storm closely approximating the 1-percent-annual-chance flood were available, but this is seldom the case. Although most historical flood data are for storms less intense than a 1-percent-annual-chance flood, these data will still indicate, at a minimum, the areas that should be within a flood zone. For instance, if a storm that produced a flood below the 1-percent-annual-chance flood elevation generally caused structural damage to houses 100 feet from the shoreline, a “reasonable” VE Zone width must be at least 100 feet. Similarly, houses that collected flood insurance claims for the same storm (without building foundation or structural damages) should at least be located in Zone AE, AH, or AO. If the analyses of the 1-percent-annual-chance flood produce flood zones and elevations indicating lesser hazards than those recorded for a more common storm, the analyses should be reevaluated. One possible explanation for changes in flood patterns from those of the historical flood event might be that a new coastal structure now acts to reduce flood hazards in the area.

If there are indications that a reevaluation is needed, the Mapping Partner should determine whether the results of the assessment are appropriate. The Mapping Partner should attempt to compare all aspects of the coastal hazard assessment to past effects, whether in the form of data, profiles, photographs, or anecdotal descriptions. The Mapping Partner should examine other data input to the assessments for wave effects (wave setup, wave height, wave runup, and wave overtopping). This includes checking that the still water levels are correct and the results of wave analyses are consistent with the historical data. The Mapping Partner should use judgment and experience to project previous storm effects onto the 1-percent-annual-chance conditions and to ensure that the coastal assessment results are consistent with previous observed events.

The objective of a coastal study is to provide legible and accurate flood hazard maps with appropriate BFEs including wave contributions. VE zones may also be mapped where the engineering analysis indicates their presence. Both engineering and practical judgment are required for a proper decision in this matter. The typical study finding is a narrow VE zone, making its usefulness uncertain on maps at usual scales. Also, relatively small numbers of existing coastal buildings are likely to be affected by possible VE zone designations along some Great Lakes. Only with prior approval from the FEMA study representative should the VE zones be mapped.

D.3.9.2 Identification of Flood Insurance Risk Zones

The Mapping Partner should identify the flood insurance risk zones and BFEs, including the wave effects, to be identified on each transect plot before delineating the flood insurance risk zones on the work maps. The existing topography, eroded topography, presence of PFDs, effects of coastal structures, and combined wave analyses (wave runup, overtopping, and overland propagation) are all important for the proper identification of flood insurance risk zones. Hazard zones that are generally mapped in coastal areas include Zones VE, AE, AH, AO, and X.[8]

D.3.9.2.1 Zone VE

Zone VE represents coastal high hazard areas where wave action and/or high-velocity water can cause structural damage during the 1-percent-annual-chance flood. Zone VE is identified using one or more of the following criteria for the 1-percent-annual-chance flood conditions:

1. The ***wave runup zone*** occurs where the (eroded) ground profile is 3.0 feet or more below the 2-percent wave runup elevation.

2. The ***wave overtopping splash zone*** is the area landward of the crest of an overtopped barrier, in cases where the potential 2-percent wave runup exceeds the barrier crest elevation by 3.0 feet or more (ΔR>3.0 feet). (See Section D.3.5.3.)

3. The ***breaking wave height zone*** occurs where 3-foot or greater wave heights could occur (this is the area where the wave crest profile is 2.1 feet or more above the total still water level (still water plus wave setup).

4. The ***primary frontal dune zone***, as defined in 44 CFR 59.1 (see Section D.3.1.2.3 of this document for more details).

The actual Zone VE boundary shown on the FIRM is defined as the farthest inland extent of any of the four criteria listed above. Zone VE is subdivided into elevation zones, and whole-foot BFEs should be assigned (see Section D.3.9.5).

[8] For a complete list of flood insurance risk zones, refer to Volume 1, Section 1.4.2.7, of the *Guidelines and Specifications*.

When the potential runup is at least 3.0 feet above the barrier crest (criterion 2), Zone VE is delineated landward of the barrier. The BFE for that zone is capped at 3 feet above the crest of the barrier. Landward of the Zone VE area, Zone AE is mapped if the ground is flat or slopes seaward, and Zone AO is mapped if the ground slopes landward.

Zone VE criterion 3, the designation of a 30-foot splash zone, should be applied to both vertical walls and sloping barriers upon the identification of wave overtopping hazards (D.3.5.4).

Delineation of the landward limit of Zone VE based on the PFD (criterion 4) requires detailed topographic data and engineering judgment. Identifying the PFD heel, "the point where there is a distinct change from a relatively steep slope to a relatively mild slope" (per Section 59.1 of the NFIP regulations) can be particularly challenging when there are inadequate topographic data and/or encroachments into the dune ridge system that obscure this slope change.

The Mapping Partner should review the available topographic data and, if necessary, conduct field verification to delineate PFDs in the study area. Previous FISs may have identified PFDs and used these features as the basis for the Zone VE designation on the effective FIRM; such information should be reviewed to aid in locating PFDs that exist at the time of the restudy. The Mapping Partner is cautioned to carefully evaluate any preexisting methods for PFD heel delineation to ensure that a reasonable approach is applied to the study area.

It is possible that a PFD may be identified landward of a shore protection structure. If the structure can be certified by the criteria in the April 23, 1990, FEMA memorandum (FEMA, 1990), *Criteria for Evaluating Coastal Flood Protection Structures for National Flood Insurance Program Purposes* the Zone VE area should be delineated based on the wave analyses for that transect (criteria 1-3, as applicable), not on the PFD heel. If the structure cannot be certified and will partially or completely fail during the base flood, Zone VE should be mapped to the PFD landward heel. Certified structures with a crest at or below the dune toe or the 10-year flood level will provide little more than protection from toe scour to a dune and will not protect inland areas or dunes from hazardous flood conditions. Low-crested structures would warrant PFD Zone VE determinations landward if deemed appropriate based on wave runup and wave height propagation analysis.

In all cases where the PFD is the basis of Zone VE, the BFE to be applied will be the wave height or wave runup elevation encountered at the dune face; see Examples 1 and 2 in Section D.3.9.6 (Figures D.3.9-2 and D.3.9-3) for more information.

D.3.9.2.2 Zone AE

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

D.3.9.2.3 Zone AH

Zone AH is used for areas of shallow flooding or ponding, with average water depths between 1.0 foot and 3.0 feet. These areas are usually not subdivided, and a BFE is assigned.

D.3.9.2.4 Zone AO

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

D.3.9.2.5 Zone X

Zone X designates areas above the 1-percent-annual-chance flood level. On the FIRM, a shaded Zone X area is subject to inundation by the 0.2-percent-annual-chance flood, and an unshaded Zone X area is above the 0.2-percent-annual-chance flood.

D.3.9.3 Shoreline

An important but potentially ambiguous map feature is the shoreline depicted in the study area. Great Lakes shorelines are subject to large position changes, due to shore erosion or accretion along with the considerable range in mean lake levels. The shoreline location may vary among the transects analyzed because of historical erosion or accretion not shown or accounted for on existing maps, but some clearly designated shoreline should be used for the FIRM. For Great Lakes studies, the Mapping Partner shall ensure the depicted shoreline corresponds to the land intercept of Low Water Datum (LWD), as given in Table D.3.9-1 and usually shown on USGS maps.

D.3.9.4 Wave Envelope

The seaward portion of the wave envelope is a combination of the potential wave runup elevation and the controlling wave crest elevation profile. The wave crest elevation profile is plotted along a transect (from the shoreline landward) based on the results of the WHAFIS model or other methodology. A horizontal line is extended seaward from the potential wave runup elevation to its intersection with the wave crest profile to obtain the wave envelope, as shown in Figure D.3.9-1. If the runup elevation is greater than the maximum wave crest elevation, the wave envelope will be represented as a horizontal line (extending to the shoreline location on the transect) at the runup elevation, and the BFE for mapping purposes will be based on that elevation. Conversely, if the wave runup is negligible, the wave crest elevation profile becomes the wave envelope.

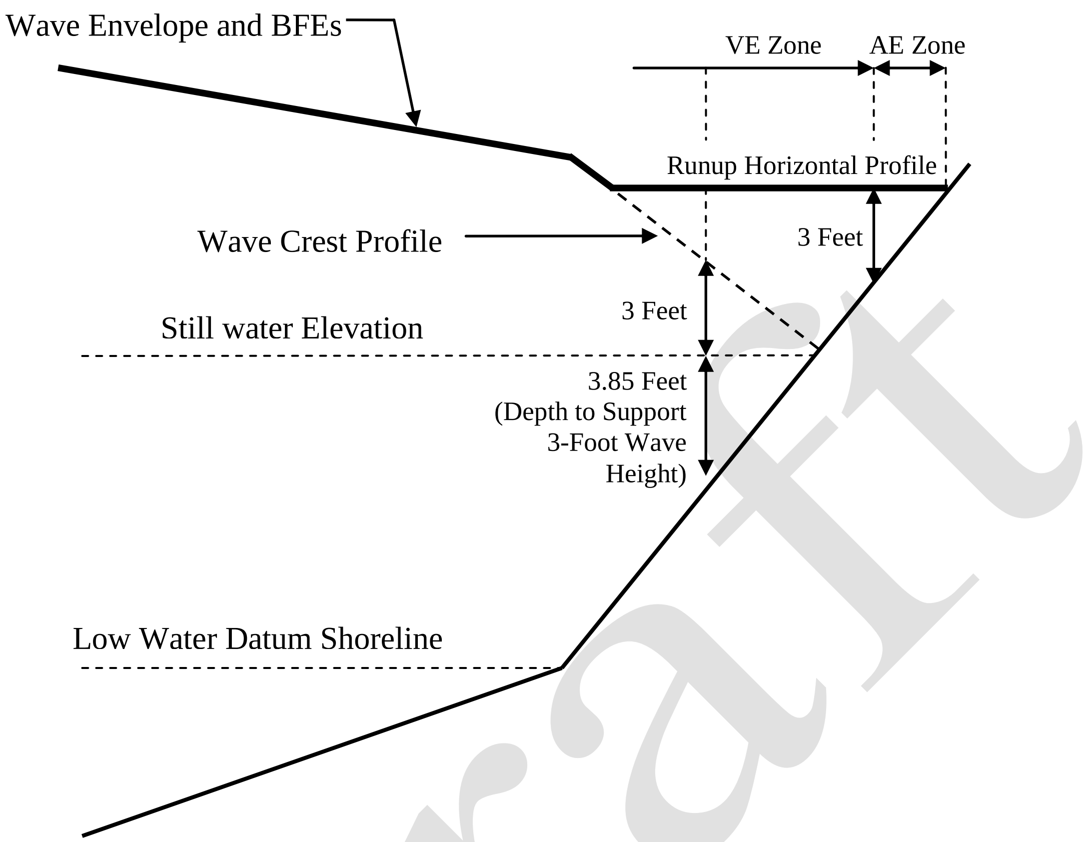


Figure D.3.9-1. Seaward Portion of Wave Envelope Based on Combination of Nearshore Crest Elevations and Shore Runup Elevation (figure not to scale)

D.3.9.5 Criteria for Flood Boundary and Hazard Zone Mapping

The first step in identifying the flood insurance risk zones along a transect is locating the inland extent of Zone VE, also known as the VE/AE boundary. The mapped Zone VE/AE boundary is based on the most landward limit of the four criteria outlined in Section D.3.9.2. The Mapping Partner should extend Zone AE from the VE/AE boundary to the inland limit of 1-percent-annual-chance inundation, which is a ground elevation equal to the potential runup elevation, or the 1-percent-annual-chance SWEL if runup is negligible. The Mapping Partner may designate additional areas of 1-percent-annual-chance flooding caused by wave overtopping sheet flow and shallow flooding or ponding as Zone AO and/or Zone AH. The Mapping Partner should label all areas above 1-percent-annual-chance inundation as Zone X (shaded for areas affected by the 0.2-percent-annual-chance flood and unshaded for areas above the 0.2-percent-annual-chance flood level).

The Mapping Partner should then subdivide the Zone VE and AE areas into elevation zones, with whole-foot BFEs assigned according to the wave envelope. Generally, Zone VE is subdivided first. Initially, the Mapping Partner should mark the location of all elevation zone boundaries on a transect. Because whole-foot BFEs are being used, these should always be mapped at the location of the half-foot elevation on the wave envelope. However, the Mapping Partner should not subdivide the horizontal runup portion of the seaward wave envelope (see Figure D.3.9-1). The BFE should simply be the runup elevation, rounded to the nearest whole foot. When the potential runup is at 3 feet or greater above the barrier crest, a VE Zone is delineated landward of the barrier and the BFE is capped at 3 feet above the crest of the barrier as outlined in Section D.3.9.2.

Ideally, the Mapping Partner would establish an elevation zone for every BFE in the wave envelope; however, because these zones are mapped on the FIRM so that buildings or property can be located in a flood insurance risk zone, the Mapping Partner should use a minimum width for the mapped zone to provide a usable FIRM. For coastal areas, the general guidance is to have a minimum zone width of 0.2 inch on the FIRM. The mapping criteria and the ability to map all coastal BFE and hazard zone changes is dependent upon the scale of the FIRM. The minimum zone width is 0.2 times the final FIRM scale; for example, a width of 80 feet for a FIRM at a scale of 1 inch equals 400 feet, or a width of 100 feet for a FIRM at a scale of 1 inch equals 500 feet. Because digital FIRM data can easily be enlarged, the map scale limitations should be reviewed by the Mapping Partner with the FEMA Study Representative and community officials.

The Mapping Partner should combine elevation zones that do not meet the minimum width requirement with an adjacent zone or zones to yield an elevation zone equal to or wider than the minimum width. The BFE for this combined zone is a weighted average of the combined zones, rounded to the nearest whole foot. When combining Zone VE areas, the Mapping Partner should not reduce the maximum BFE at the shoreline by averaging.

Zone AE, if the area is wide enough, should be subdivided in the same manner. If the total Zone AE width is less than the minimum width requirement, the VE zone with the lowest elevation is usually assigned to that area. This situation typically occurs for steep or rapidly rising ground profiles, and it is not unreasonable to designate the entire area of inundation as Zone VE. In some cases, however, it may be appropriate for the Mapping Partner to extend the AE zone slightly into the next zone seaward to satisfy the minimum width requirement.

Relatively low areas landward of zones subject to wave effects may be subject to shallow flooding or the ponding of floodwater; the Mapping Partner should designate these areas as Zones AO or AH. Such designations can be relatively common landward of coastal structures, bluffs, ridges, and dunes, where wave overtopping occurs.

Identifying appropriate zones and elevations may require particular care for dunes, given that the entire PFD is defined as a coastal high hazard area. Although the analyses may have determined that a dune will not completely erode and that the wave action should stop at the retreated dune face with only overtopping possibly propagating inland, the Mapping Partner should designate the entire dune as Zone VE, as defined in the NFIP regulations. The Mapping Partner should assign the last calculated BFE at the open-coast dune face (whether Zone VE or AE) to be the dominant Zone VE BFE for the entire PFD and should extend this value to the landward limit of the PFD. It may seem unusual to use a BFE lower than the ground elevation, but this is fairly common. Most of the BFEs for areas where the dune was assumed to be eroded are also below existing ground elevations. In these cases, it is the Zone VE designation that is most important to the NFIP because, under current regulations, structures in Zone VE must be built on pilings, and alterations to the dunes are prohibited.

Section D.2.11.2 provides mapping examples depicting common flood hazard mapping for idealized transects for the following beach settings.

1. Sandy beach backed by a low sand dune or sand berm
2. Sandy beach backed by high sand dune or berm
3. Beach backed by shore protection structure (e.g., seawall)
4. Erodible coastal bluffs
5. Non-erodible coastal bluffs or cliffs

D.3.9.6 Mapping Procedures

This section presents guidance for mapping newly studied coastal zones and remapping or redelineating coastal flood insurance risk zones. In redelineation, effective SWELs and BFEs are remapped using new or more detailed topographic data and base maps, or to implement a vertical datum conversion. Included below are the requirements for reviewing the initial model results and identifying flood insurance risk zones, guidance and examples for determining transects, and guidance for depicting the analysis on the FIRM.

D.3.9.6.1 Newly Studied Coastal Zones

A properly integrated delineation of the results of flooding analyses involves judgment and skill in reading topographic and land-cover maps. The time and effort put forth to determine the flood elevations and flood zone extents will be negated if the results of these analyses are not properly delineated on the FIRM. Provided below is a description of the general process by which the coastal analyses are to be transformed from a series of flood zones and BFEs calculated along numerous transects to a mapped product consistent with these mapping guidelines and specifications.

The preliminary FIRM is usually produced from engineering work maps based on the coastal analyses. Therefore, the Mapping Partner must transfer the flood zones and elevations identified on each transect's wave profile to the work maps and interpolate boundaries between transects. To do so, the Mapping Partner will set up the work maps with contour lines, buildings, structures, vegetation, and transect lines clearly located. Because roads are often the only fixed physical features shown on the FIRM, the Mapping Partner should ensure that other features and the flood zone boundaries are properly located on the work maps in relation to the centerline of the roads as they will appear on the FIRM. The starting point (shoreline) for each transect should be clearly annotated on the work maps.

The Mapping Partner must transfer the identified elevation zones from the wave profile to the work maps, marking the location of the flood zone boundaries along the transect line so that boundary lines can be interpolated between transects. The Mapping Partner will ensure that boundaries are marked at the correct location. Because of erosion assumptions, the location of the LWD elevation can change on the transect, but the 0 Station, the point from which the flood zone changes from the wave profile are referenced, must remain fixed on the work map. As discussed in Section D.3.9.5, some flood zones on the wave envelope may be too narrow to map at map scale. Thus, some zones must be eliminated, and elevations must be averaged. The Mapping Partner should measure the widths of the resulting flood zones carefully; zones that narrow to less than 0.2 inch at map scale may need to be tapered to an end. Likewise, if the averaged flood zone becomes much wider, it may be possible to break the averaged zone back into two (or more) separate elevation zones. However, because digital FIRM data can easily be enlarged, the map scale limitations should be reviewed by the Mapping Partner with the FEMA Study Representative and community officials.

With final elevations from the wave profile plotted on the work maps and any zone averaging completed, the Mapping Partner should determine the location of each flood zone change in relation to a physical feature (e.g., ground contour, back side of a row of houses, 50 feet into a vegetated area, etc.) and delineate the boundary for the area represented by that transect along this feature. For example, if the BFE for Zone VE decreases from 14 feet to 13 feet coincident with change from a residential area to a forest, the Mapping Partner should examine the land use data and follow the boundary of the forest to the left and right of the transect line to extend the delineation of the flood zone change.

One of the more difficult steps in delineating coastal flood zones and elevations is the transition between transects. Good judgment and an understanding of typical flooding patterns are vital to performing this work accurately. Initially, the Mapping Partner should locate the area of transition (an area not exactly represented by either transect) on the work maps. The Mapping Partner should then delineate the floodplain boundaries for each transect up to this transition area. The Mapping Partner should examine how a transition can be made across this area to connect matching zones and still have the boundaries follow logical physical features. Other transects similar to this area could give an indication of flooding. Sometimes the elevation zones for the two contiguous transects are not the same; in such cases, the Mapping Partner may have to taper the zones to an end or enlarge the zones and subdivide them in the transition area. Additional transects may be required to assist with transitions which prove problematic.

Areas with significant flooding hazards from wave runup may have one transect representing multiple alongshore reaches because the areas have similar shore slopes. In this case, the Mapping Partner should identify the different areas and delineate the results of the typical transect in each area. Transition zones may be necessary between areas with high runup elevations to avoid large differences in BFEs, and to smooth the change in flood zone boundaries. These zones should be fairly short and cover the shore segment with a slope not exactly typical of either area. The Mapping Partner should determine the transition elevation using judgment in examining runup transects with similar slopes. The Mapping Partner should not use transition zones if there is a very abrupt change in topography, such as at the end of a coastal structure.

Lastly, after plotting flood zones and BFEs and interpolating results between transects, the Mapping Partner should map the Zone X areas. The Mapping Partner should show areas below the 0.2-percent-annual-chance SWEL that are not covered by any other flood zone as Zone X (shaded) on the FIRM. Often, the maximum runup elevation associated with the base flood is higher than the 0.2-percent-annual-chance SWEL. In such cases, the Zone X (shaded) designation will not be used in that area. All other areas are designated Zone X without shading.

Although BFEs are mapped to the whole foot, the SFHA boundary should be delineated using the SWEL or runup elevation to the tenth of a foot. Mapping of the SFHA boundary must conform to FEMA's floodplain boundary standards (See FEMA Procedure Memorandum No. 38). In preparing the FIRM, the Mapping Partner should ensure that the mapped results are technically correct and that the FIRM is easy for the community official, engineer, surveyor, and insurance agent to use.

D.3.9.6.2 Redelineation of Coastal Zones

During the project scoping phase, coastal reaches may be identified where new surge modeling and detailed wave analyses are not required. In these cases, the Mapping Partner will be responsible for remapping or redelineating the effective coastal flood hazard data onto the new FIRM. For detailed guidance on coastal redelineation please see *Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update* (FEMA, 2007).

D.3.9.6.3 Limit of Moderate Wave Action (LiMWA)

Flood hazard identification under the National Flood Insurance Program (NFIP) divides coastal flood hazard areas into two flood zones: Zone VE and Zone AE. Present NFIP regulations make no distinction between the design and construction requirements for coastal AE Zones and riverine AE Zones. However, evidence suggests that design and construction requirements in some portions of coastal AE zones should be more like VE Zone requirements. Post-storm investigations have shown that typical AE Zone construction techniques (e.g., woodframe, light gauge steel, or masonry walls on shallow footings or slabs, etc.) are subject to damage when exposed to waves less than 3-feet in height. One of the hazard identification criteria for VE Zone designation is where wave heights are estimated to be equal to or greater than 3 feet. Laboratory tests and field investigations confirm that wave heights as small as 1.5 feet can cause failure of the above-listed wall types. Other flood hazards associated with coastal waves (e.g., floating debris, high velocity flow, erosion, and scour) also damage AE Zone-type construction in these coastal areas.

For all new detailed coastal study starts in Fiscal Year 2009, the landward limit of waves 1.5 feet in height will be delineated on the FIRMs and included in the DFIRM database as an informational layer with no NFIP floodplain management requirements or special insurance ratings. Communities are encouraged but not required to adopt higher standards than the minimum NFIP requirements in these areas. The limit will be included on the preliminary FIRM; however, if a community does not want to delineate the limit on its final FIRMs, the community may provide a written request to their FEMA Study Representative with justification for such a request. See FEMA's *Procedure Memorandum No. 50-- Policy and Procedures for Identifying and Mapping Areas Subject to Wave Heights Greater than 1.5 feet as an Informational Layer on Flood Insurance Rate Maps (FIRMs)* for more information.

D.3.10 Study Documentation

This section summarizes the reporting requirements for coastal Flood Insurance Studies (FISs) on the Great Lakes coasts, with emphasis on the intermediate data submissions that document the basis and results of coastal flooding analyses during the course of the FIS.

The Mapping Partner must fully document the coastal flood hazard determination for each affected community. FIS reports and FIRMs form the basis of Federal, State, and local regulatory and statutory enforcement mechanisms and are subject to administrative appeal. Mapping Partners must ensure that all technical processes and decisions are recorded and documented. Such documentation will provide detailed data needed by FEMA or the community to reconstruct or defend the study results on technical grounds.

Reporting requirements for coastal studies must include the following:

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

The data capture standards for these requirements are described in Appendix M: Data Capture Standards of these Guidelines. The information must be submitted to the Mapping Information Platform (MIP) via the internet. All documentation must be dated. At a minimum, mapping data must contain a descriptive label, source reference, compilation date, projection, and if elevation data are included, a vertical datum. Appendix M is not intended for drafts, preliminary, or interim submittals. The final data that Mapping Partners upload to the MIP should be the final deliverable required by the Mapping Activity Statement (MAS) and must comply with the DCS. However, certain "raw" data that is to be submitted with an intermediate report should be submitted in the format described in Appendix M. These data might include: storm climatological and meteorological event selections, still water elevations, wave and wind data, coastal structures, and model input files.

D.3.10.1 General Documentation

This portion of the reporting requirements includes background information compiled by the Mapping Partner related to changes in scope; special problem reports; minutes of meetings held with the FEMA, communities, and other Mapping Partners; and all correspondence for the study effort (e-mail and hard copy).

D.3.10.2 Engineering Analyses

Intermediate data submissions provide defined milestones in the coastal flood study process, for review of the study approach and results. The Mapping Partner must submit the data to FEMA in the sequence below.

Intermediate Submission No. 1 – Scoping and Data Review

Intermediate Submission No. 2 – Offshore Water Levels and Waves

Intermediate Submission No. 3 – Nearshore Hydraulics

Intermediate Submission No. 4 – Draft Flood Hazard Mapping

The Mapping Partner shall receive review comments within 30 days of receipt of each data submission. The Mapping Partner performing the study shall establish a work plan, so the interim review does not cause any delay in the submission of the draft FIS report and FIRM.

In each section of the engineering report, the Mapping Partner must provide a complete list of technical references, including computer program references, indicating how to obtain copies of the exact program and the input data sources used in the analysis. Any alterations to the computer code used should be noted.

D.3.10.2.1 Intermediate Submission No. 1 – Scoping and Data Review

In this report phase, the Mapping Partner shall provide the background information on the study setting and available data relevant to the study area. Any new data needed for the detailed coastal analyses in subsequent phases must be identified in this phase. Unless otherwise agreed upon with the FEMA Study Representative, the study must not proceed until all of the information is available and incorporated into the scoping document, which is then submitted for approval by FEMA.

Topographic and Bathymetric Data: If available at this stage, this submission must include survey control data, topographic data from aerial photography, LIDAR, and field and bathymetric surveys. If survey work is still in progress, the submission must include available data at the time of submission and a detailed description of the planned survey data collection. Information shall be submitted on the extent of topographic and bathymetric mapping, key mapping parameters (e.g., contour intervals and accuracy standards), horizontal and vertical datum, location and extent of transects, and other pertinent information describing the extent and quality of survey information to be used in the study. If existing community mapping data will be used to supplement survey efforts for the study, the Mapping Partner must submit information on the date, accuracy standards, datum, extent, and limitations of the mapping.

Water-Level, Wind, Wave, and Flooding Data: This submission must include a description of available water-level, wind, and wave data that relate to study analysis requirements. The submission shall include an evaluation of local and regional water-level records while recognizing that these records include storm surge, and possibly other influences (e.g., river flows and wave setup). The submission shall include the review and selection of wind stations in the vicinity of the study area that can provide reasonable length of record, hourly values, and peak gusts to help estimate extreme wind statistics; the evaluation of available wave or wave hindcast data; and the evaluation of available historical data (measured and anecdotal) on past coastal flood events. These data should be submitted as described in Appendix M: Data Capture Standards of these Guidelines.

- **Site Reconnaissance:** The results of the site reconnaissance must be documented to characterize exposure and coastal morphology, inventory existing coastal structures and levees (including buried coastal structures), identify shorelines where beach nourishment has occurred and could influence coastal flooding analyses and mapping, characterize coastal vegetation where it may influence coastal flooding analyses and mapping, locate analysis transects for subsequent field survey and ultimate use in wave calculations, and identify representative reaches with similar exposure, morphology, and features.
- **Technical Approach:** The submission must describe the technical approach for the analysis of coastal processes and the mapping of flood hazards in various settings and shoreline morphologies present in the study area.

D.3.10.2.2 Intermediate Submission No. 2 – Offshore Water Levels and Waves

Documentation of this phase must describe the primary analyses of water-level and wave conditions. Where applicable, the submission shall include:

- **Storm Climatology and Storm Windfield Methodology**: The Mapping Partner shall describe the basic climatological storm data used and the windfield methodology. The Mapping Partner shall also provide a discussion of any unique storm model treatments.
- **Wave Data and Hindcasts:** The submission must describe data and analyses used to select and define storm events for use in response-based analysis of nearshore processes and subsequent statistical analysis of 1-percent and 0.2-percent-annual-chance flood conditions. Documentation must include details of the sources of wave and wind data. It shall also include comparisons between alternate sources (where more than one is available and feasible for use in the FIS) and comparison with local measurements. Documentation of incident deepwater waves should include period, direction, and directional spreading parameters. The selection of coefficients for angular spreading and spectral peakedness parameters must be clearly stated and justified.
- **Hydrodynamic Storm-surge model**: This section of the engineering report should address the hydrodynamic storm-surge model employed in performing the coastal study. The Mapping Partner shall:
    - Report the unique model characteristics used for the study, including a discussion of the specific grid system and sub-grid systems employed, the grid used for bottom topography (bathymetry) and the shoreline, small-scale

features such as harbors and barrier islands, and the location and conditions applied for the open boundaries to the grid.

- Describe and document the method used to determine average ground elevations and water depths within the cells of the grid system. (This discussion is to be augmented by diagrams that show the grid systems as computer listings of the grid data used in the actual model calculations.)
- Describe the method used to relate windspeed to the surface drag coefficient.
- Discuss the Manning's "n" values used in the calculation of bottom and overland friction and provide values in tabular form, including a discussion of any sensitivity tests used to estimate these values in nearshore water. (Nearshore, bottom, and overland friction are important parts of the overall analysis and shall be described with care and in sufficient detail.)
- Provide a graphical depiction of the model cells and grid system as an overlay to the bathymetric charts and topographic maps covering the study area, annotated with the individual cell inputs for the grid system.
- Discuss the treatment of barriers, inlets, and rivers.
- Explain the procedures used to determine inland flooding, including parameterization of local features and selection of the friction factors used for the terrain.

- **Water level and Wave Model Calibration and Validation**: The Mapping Partner shall document the calibration and validation of the hydrodynamic surge and wave models. When observed (or model simulation) data are employed to calibrate (or compare) model results with other available studies, the Mapping Partner shall give a complete description of this calibration procedure (or model comparison). Calibration (and model comparison) is an important aspect of the model analysis; therefore, the Mapping Partner shall describe these activities with sufficient detail and care to allow an independent reviewer to understand the exact procedures and local historical records employed.
- **Estimation of the 1-Percent and 0.2-Percent-Annual-Chance Floods:** Documentation must be provided on the methods to be used to estimate the 1-percent and 0.2-percent-annual-chance coastal flooding conditions. Methods of extrapolation of hindcast and/or measured data to 1-percent and 0.2-percent-annual-chance values should be documented, including comparisons between alternate procedures if appropriate. Where extremal analyses of wave, wind, and water levels are used, the submission shall include documentation of the analyses to develop frequency relationships, including a description of the data sets and analysis assumptions.

- **Sheltered Waters – Hindcast Waves:** Documentation must be provided on fetch length determination and corresponding wind speeds, directions, and durations for use in hindcast analyses. This shall include documentation of wind speed adjustments and wind field hindcast methods.
- **Sheltered Waters – Water Levels:** The Mapping Partner must document the characteristics of water-level gages located within or near the study area that will potentially be used in study analyses or validation. Methods adopted to infer the variation of vertical datums between gages must be documented, as must procedures used to transpose data from one site to another. If a field effort is undertaken to determine the variation of vertical datums within ungaged regions, the Mapping Partner shall fully document that effort, including: locations of observations; observation methods and instrumentation; dates and times of all observations; meteorological and oceanographic conditions during and preceding the period of observation; and other factors that may have influenced water levels, or that may affect interpretation of the results. Inlet analyses shall be well documented, including all procedures, methodological assumptions, field surveys (dates, times, procedures, instrumentation, and findings), and all inlet data adopted from other sources.

Proposed Transect Location Map: The Mapping Partner should submit one or more maps as appropriate depicting the location and orientation of transects to be used in the subsequent wave elevation determination analyses. The transect location map(s) should be at a suitable scale and should show transects of sufficient length to account for modeling of all coastal flooding conditions.

D.3.10.2.3 Intermediate Submission No. 3 – Nearshore Hydraulics

The nearshore hydraulics phase must provide documentation of methods applied and detailed analyses conducted before the hazard zone mapping phase.

Wave Information: The Mapping Partner must document all assumptions used to define nearshore waves. In sheltered waters, the documentation must include a summary of fetch determination, winds (speed, direction, and duration), and bathymetry used in hindcasts. The documentation must include the approximations or assumptions used in the analysis. When observational data, such as wave buoy data, are available, the wave height, period, and spectral parameters should be compared to the predicted waves.

Wave Transformation: The Mapping Partner must document the assumptions, methods, and results of all analyses of wave transformations conducted for the study. This documentation must include the selection of offshore and nearshore points, the source of transformation coefficients, and any special assumptions regarding local transformation processes, such as sheltering and reflection. If a spectral wave model is applied for nearshore transformation, all modeling factors must be sufficiently documented so the modeling effort can be reproduced if necessary. If a field effort is undertaken to validate transformation models, the field work must be summarized in detail, including times and locations of all observations, general conditions at the time the work was performed, a full description of all equipment and procedures, and a summary of all data in archival form. A description of the bathymetric data used in the transformation calculations must also be provided.

Runup, Setup, and Overtopping Analyses: The Mapping Partner must document the runup, setup, and overtopping analysis assumptions, methods, input data, and results. This must include a determination of runup heights and still water elevations (SWELs) and determination of flood insurance risk zone parameters (1-percent and 0.2-percent-annual-chance flood depths, overtopping splash penetration and overtopping rate, and overland flow velocity) at each transect. This must include a description of profiles used, runup reduction factors, and the basis for splash zones to be used in hazard mapping. The documentation must include a description of any observations or measurements used to validate or adjust analysis results, any deviations from recommended procedures in Section D.3.5, any difficulties encountered in the analyses, and the technical decisions or approaches taken in their resolution. The Mapping Partner should include one or more transect location maps, as appropriate, and computer printout listings for the input and output data, keyed to the transect location map(s), as an appendix to the report.

Wave Dissipation and Overland Propagation: The Mapping Partner must describe the areas where wave attenuation was investigated, and document the analysis assumptions, methods, input data, and results. This must include documentation of any field observations or measurements, as well as available historical or anecdotal information regarding wave attenuation during flooding events. The Mapping Partner should include computer printout listings for the input and output data, keyed to the transect location map(s), as an appendix to the report.

Coastal Armoring Structures: The Mapping Partner must describe assumptions and investigations of the various coastal armoring structures (e.g., seawalls, revetments, bulkheads, levees, etc.) in the study area relevant to stability and capability to withstand 1-percent-annual-chance water-level and wave conditions. This documentation must include any assumptions or approximations used in the analyses. The same documentation is required in the event that coastal structures are apparently buried and not visible, but are indicated by information collected during the study. In cases where the Mapping Partner could not determine whether a given structure would survive the 1-percent-annual-chance flood intact, and where multiple analyses were conducted for the structure (i.e., intact condition, failed condition/removed from the analysis transect), the Mapping Partner must document each analysis and record the structure condition used to map flood insurance risk zones and BFEs. This information will be useful in the event a map revision is requested based on a structure condition different from that used as the basis for the FIRM.

Beach Stabilization Structures: The Mapping Partner must document the treatment of beach stabilization structures (e.g., groins, offshore breakwaters, sills, etc.) during the study. If the Mapping Partner proposes removal or modification of beach stabilization structures (or their shoreline effects) during the 1-percent-annual-chance flood, the Mapping Partner must document the existence, history of, and shoreline response to beach stabilization structures and consult with the FEMA Study Representative.

Miscellaneous Structures: If miscellaneous structures (e.g., piers, port and navigation structures, bridges, culverts, storm gates, etc.) are present in the study area and could exert a significant influence on nearshore waves, currents, sediment transport, or backshore ponding, the Mapping Partner must document the data, methods, and procedures used to evaluate the stability of these structures during the 1-percent-annual-chance flood and their effects on coastal flooding. This documentation must include assumptions or approximations used in the analyses. The Mapping Partner should document the treatment of all coastal structures as required in Appendix M: Data Capture Standards of these Guidelines.

Erosion Analyses: The Mapping Partner must document the erosion analysis assumptions, methods, input data, and results. The Mapping Partner must document any unusual conditions in the study area and the methods proposed to map hazard zones based on these conditions. These may include the effects of beach nourishment and/or flood borne debris; special hydrodynamic considerations at inlets and passages; the effects of riverine inflows, unusual erosion or other sedimentation characteristics; unusual structure effects and/or the effects of multiple levees, and any other factors that the Mapping Partner considers relevant to mapping flood hazards accurately.

D.3.10.2.4 Intermediate Submission No. 4 – Draft Flood Hazard Mapping

The draft flood hazard mapping phase must provide documentation of the methods used to convert the results of the detailed hydraulic analyses into flood insurance risk zones.

Flood Insurance Risk Zone Limit Identification: The Mapping Partner must document the analysis results used in the determination of hazard zone limits and BFEs. In addition, the summary must include a description of the basis for erosion and coastal structure conditions (e.g., overtopping cases, method of profile determination, failed and buried coastal structure cases, etc.) used in the determination of the hazard zones.

Flood Insurance Risk Zone Map Boundary Delineation: The Mapping Partner must provide draft work maps for the study area showing all flood insurance risk zone limits identified along the transects resulting from the detailed analyses and transferred to the topographic work maps. This submission must describe the engineering judgment used to interpolate and delineate hazard zones between transects, including land features that might affect flood hazards, changes in contours, and the lateral extent of coastal structures. It must also provide detailed documentation and technical justification of adjustments in the hazard zone mapping due to observed historical flood data and/or damages in the study area.

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D.3.12 Notation

Symbol	Description	Units	Typical Units English	Typical Units SI
B	Berm height	L	ft	m
C	Wave phase velocity or celerity	L/T	ft/s	m/s
D	Diameter	L	ft	m
	Quarrystone diameter	L	ft	m
	Dune height			
d_h	Depth over berm	L	ft	m
d_s	Local still water depth	L	ft	m
E_B	Computed erosion estimate	L^2	ft^2	m^2
E_{WH}	Estimated eroded area for the recurrence interval of the wave height	L^2	ft^2	m^2
E_{WL}	Estimated eroded area for the recurrence interval of the water level	L^2	ft^2	m^2
F	Freeboard	L	ft	m
G	Gravitational constant	L/T^2	ft/s^2	m/s^2
H	Wave height	L	ft	m
$\overline{H}$	Mean, average over all waves			
H'_o	Unrefracted deep water wave height	L	ft	m
H_b	Breaking wave height	L	ft	m
H_c	Controlling wave height	L	ft	m
H_{mo}	Zero moment wave height	L	ft	m
H_o	Significant deep water wave height	L	ft	m
H_s	Significant wave height	L	Ft	m
h	Water depth height of the bluff crest above the event water level	L	ft	m
L	Wave Length	L	ft	m
L_{berm}	Berm width	L	ft	m
L_0	Deep water wave length, $gT^2/2\pi$	L	ft	m
m	Beach slope (rise/run)	L/L	--	--
P	Average porosity of rubble structure cover layer	--	--	--
Q	Dimensionless overtopping	--	--	--
$\overline{Q}$	Mean overtopping rate	L^3/T	ft^3/s	m^3/s
R'	Excess height (runup)	L	ft	m
	Potential Runup Elevation			
R	Mean Runup	L	ft	m
R_a	Adjusted runup elevation	L	ft	m

Symbol	Description	Typical Units		
		Units	**English**	**SI**
R_{inc}	2-percent incident wave runup on natural beaches	L	ft	m
$R_{2\%}$	Runup exceeded by 2% of the runup events	L	ft	m
r	Linear correlation coefficient			
T	Wave period	T	s	s
$\overline{T}$	Mean, average over all waves	T	s	s
$T_{m-1.0}$	Spectral wave period	T	s	s
T_p	Spectral peak period, $1/f_p$	T	s	s
T_s	Significant wave period	T	S	S
v	Horizontal (y) component of local fluid velocity (water particle velocity)	L/T	ft/s	m/s
		L/T	ft/s	m/s
x,y,z	Right-handed Cartesian coordinates	L	ft	m
z_c	Structure crest elevation	L	ft	m
β	Storm profile response coefficient	--	--	--
	Wave angle at structure	deg	deg	deg
γ	Runup reduction coefficients			
γ_r	Roughness reduction factor	--	--	--
γ_b	Berm section in breakwater			
γ_β	Wave direction factor			
γ_P	Porosity factor			
ΔR	Potential excess runup	L	ft	m
$\overline{\eta}$	Mean or static wave setup	L	ft	m
$\overline{\eta}_{max}$	Maximum static wave setup	L	ft	m
$\overline{\eta}_o$	Static setup at the shoreline	L	ft	m
Θ_d	Deep water wave direction			
κ	ratio of breaking wave height to breaking water depth			
ξ	Iribarren number	--	--	--
ξ_{om}	Spectral deep water ξ	--	--	--
π	Constant = 3.14159	--	--	--
B	Berm height	L	ft	m
C	Wave phase velocity or celerity	L/T	ft/s	m/s
D	Diameter	L	ft	m
	Quarrystone diameter	L	ft	m
	Dune height			
d_h	Depth over berm	L	ft	m
d_s	Local still water depth			

Symbol	Description	Units	Typical Units English	Typical Units SI
E_B	computed erosion estimate (d.3.7-5)			
E_{WH}	estimated eroded area for the recurrence interval of the wave height, in square feet (d.3.7-5)			
E_{WL}	estimated eroded area for the recurrence interval of the water level, in square feet (d.3.7-5)			
F	Freeboard	L	ft	m
F_c				
g	Gravitational constant	L/T^2	ft/s^2	m/s^2
H	Wave height	L	ft	m
$\overline{H}$	Mean, average over all waves			
H'_o	Unrefracted deep water wave height	L	ft	m
H_b	Breaking wave height	L	ft	m
H_c	Controlling wave height			
H_{mo}	Zeroth moment wave height			
H_o	Significant deep water wave height	L	ft	m
H_s	Significant wave height	L	Ft	m
h	Water depth height of the bluff crest above the event water level (d.3.7-8)	L	ft	m
L	Wave Length	--	--	--
L_{berm}	Berm width	L	ft	m
L_0	Deep water wave length, $gT^2/2\pi$	L	ft	m
m	Beach slope (rise/run)	L/L	--	--
P	Average porosity of rubble structure cover layer	--	--	--
Q	Dimensionless overtopping	--	--	--
$\overline{Q}$	mean overtopping rate (pg d.3.5-19)			
R'	Excess height (runup) Potential Runup Elevation			
R	Mean Runup			
R_a	Adjusted runup elevation			
R_{inc}	2-percent incident wave runup on natural beaches			
$R_{2\%}$	Runup exceeded by 2% of the runup crest	L	ft	m
r	Linear correlation coefficient			
T	Wave period	T	s	s
$\overline{T}$	Mean, average over all waves			
$T_{m-1.0}$	spectral wave period			
T_p	Spectral peak period, $1/f_p$	T	s	s

Symbol	Description	Typical Units		
		Units	English	SI
T_s	Significant wave period	T	S	S
	Time scale for beach profile response	--	--	--
v	Horizontal (y) component of local fluid velocity (water particle velocity)	L/T	ft/s	m/s
		L/T	ft/s	m/s
x,y,z	Right-handed Cartesian coordinates	L	ft	m
z_c	Structure crest elevation	L	ft	m
β	Storm profile response coefficient	--	--	--
	Wave angle at structure	deg	deg	deg
γ	Runup reduction coefficients			
γ_r	Roughness reduction factor	--	--	--
γ_b	Berm section in breakwater			
γ_β	Wave direction factor			
γ_P	Porosity factor			
ΔR	Potential excess runup	L	ft	m
$\bar{\eta}$	Mean or static wave setup	L	ft	m
$\bar{\eta}_{max}$	Maximum static wave setup	L	ft	m
$\bar{\eta}_o$	Static setup at the shoreline	L	ft	m
Θ_d	deep water wave direction			
κ	ratio of breaking wave height to breaking water depth (pg d.3.5-3)			
ξ	Iribarren number	--	--	--
ξ_{om}	Spectral deep water ξ	--	--	--
π	Constant = 3.14159	--	--	--
B	Berm height	L	ft	m
C	Wave phase velocity or celerity	L/T	ft/s	m/s
D	Diameter	L	ft	m
	Quarrystone diameter	L	ft	m
	Dune height			
d_h	Depth over berm	L	ft	m
d_s	Local still water depth			
E_B	computed erosion estimate (d.3.7-5)			
E_{WH}	estimated eroded area for the recurrence interval of the wave height, in square feet (d.3.7-5)			
E_{WL}	estimated eroded area for the recurrence interval of the water level, in square feet (d.3.7-5)			
F	Freeboard	L	ft	m
F_c				
g	Gravitational constant	L/T^2	ft/s^2	m/s^2
H	Wave height	L	ft	m

Symbol	Description		Typical Units	
		Units	English	SI
$\overline{H}$	Mean, average over all waves			
H_o'	Unrefracted deep water wave height	L	ft	m
H_b	Breaking wave height	L	ft	m
H_c	controlling wave height (table d.3.1-2)			
H_{mo}	Zero moment (tbl d.3.1-2)			
H_o	Significant deep water wave height	L	ft	m
H_s	Significant wave height	L	Ft	m
h	Water depth	L	ft	m
	height of the bluff crest above the event water level (d.3.7-8)			
L	Wave Length	--	--	--
L_{berm}	Berm width	L	ft	M
$L_0 L_{op}$	Deep water wave length, $gT^2/2\pi$?	L	ft	m
L_0	Deep water wave length, $gT^2/2\pi$	L	ft	m

D.3.13 Acronyms

The Federal Emergency Management Agency (FEMA) has an extensive list of acronyms posted on the FEMA website at <http://www.fema.gov/fhm/dl_cgs.shtm>, *Acronyms and Abbreviations*. The acronyms below are specific to this document and include some of the acronyms given in the FEMA list.

2-D	Two-Dimensional
ACES	USACE ACES program
BATHYS	BATHYS—computer program
BFE	Base Flood Elevation
BST	Bathystrophic Storm Tide
CEM	Coastal Engineering Manual
CERC	Coastal Engineering Research Center
CFR	Code of Federal Regulations
CHAMP	Coastal Hazard Analysis Modeling Program
CZM	Coastal Zone Management
DCS	Data Capture Standards
DFIRM	Digital Flood Insurance Rate Map
DHL	Delft Hydraulics Laboratory of the Netherlands
DIM	Direct Integration Method
ERDC	Engineer Research and Development Center
EST	Empirical Simulation Technique
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
G&S	FEMA *Guidelines and Specifications*
GEV	Generalized extreme value
GIS	Geographic Information Systems
GLERL	Great Lakes Environmental Research Laboratory
IF	Inland Fetch
IGLD85	International Great Lakes Datum of 1985
JPM	Joint Probability Method
LIDAR	Light Detection and Ranging (System)
LWD	Low Water Datum
MIP	Mapping Information Platform
MAS	Mapping Activity Statement
MSL	Mean Sea Level
MWL	Mean water level
NAVD88	North American Vertical Datum of 1988
NDBC	National Data Buoy Center
NFIP	National Flood Insurance Program
NOAA	National Oceanic and Atmospheric Administration
NWLON	National Water Level Observation Network
OF	Overwater Fetch
PFD	Primary Frontal Dune

SPM	Shore Protection Manual
SWEL	Still water elevation
SWL	Still water level
TAW	Technical Advisory Committee for Water Retaining Structures
TWG	Technical Working Group
TWL	Total water level
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey
WHAFIS	Wave Height Analysis for Flood Insurance Studies
WIS	Wave Information Study