Guidance for Flood Risk Analysis and Mapping

Coastal Wave Setup

November 2015

FEMA

Requirements for the Federal Emergency Management Agency (FEMA) Risk Mapping, Assessment, and Planning (Risk MAP) Program are specified separately by statute, regulation, or FEMA policy (primarily the Standards for Flood Risk Analysis and Mapping). This document provides guidance to support the requirements and recommends approaches for effective and efficient implementation. Alternate approaches that comply with all requirements are acceptable.

For more information, please visit the FEMA Guidelines and Standards for Flood Risk Analysis and Mapping webpage (www.fema.gov/guidelines-and-standards-flood-risk-analysis-and-mapping). Copies of the Standards for Flood Risk Analysis and Mapping policy, related guidance, technical references, and other information about the guidelines and standards development process are all available here. You can also search directly by document title at www.fema.gov/library.

Document History

Affected Section or Subsection	Date	Description
First Publication	November 2015	Initial version of new transformed guidance. The content was derived from the Guidelines and Specifications for Flood Hazard Mapping Partners, Procedure Memoranda, and/or Operating Guidance documents. It has been reorganized and is being published separately from the standards.

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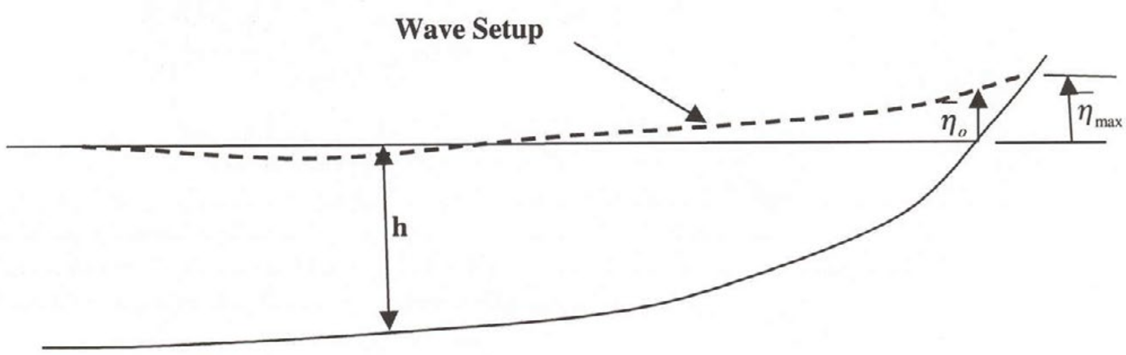
1.0 Coastal Wave Setup Overview

This topic provides guidance for the determination of wave setup—an increase in the total stillwater elevation against a barrier (dunes, bluffs, or structures) caused by breaking waves.

Wave setup can be a significant contributor to the total water level and should be included in the determination of coastal Base Flood Elevations (BFEs). The manner in which it is included, however, is critical to the accuracy of the BFEs. In addition to wind, waves can also affect the mean nearshore water levels during hurricanes and severe storms. This occurs as a result of the transfer of momentum from waves to the water column (see Figure 1-1). Wave setup increases as the water depth near a barrier decreases and wave dissipation increases.

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



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

Oscillations in the wave setup also occur, and this oscillation is known as "dynamic" wave setup. These oscillations typically occur with periods of 10 to 20 times the mean wave period. The dynamic wave setup increases with narrow frequency spectra and narrow directional spectra, both uncharacteristic of hurricane and nor'easter conditions.

2.0 Atlantic & Gulf

For the Atlantic and Gulf Coast regions, there are two ways of estimating stillwater levels for use in a Flood Risk Project. One involves separate calculations of storm surge and wave setup, and one computes storm surge and wave setup concurrently. Recall that the stillwater level comprised of the combination of these two components is the *mean water level* (MWL).

In the first case, wave setup must be added to the storm surge stillwater level for overland wave transformation calculations, wave runup calculations, and for dune erosion removal/retreat calculations.

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

2.1 Estimating Static Wave Setup

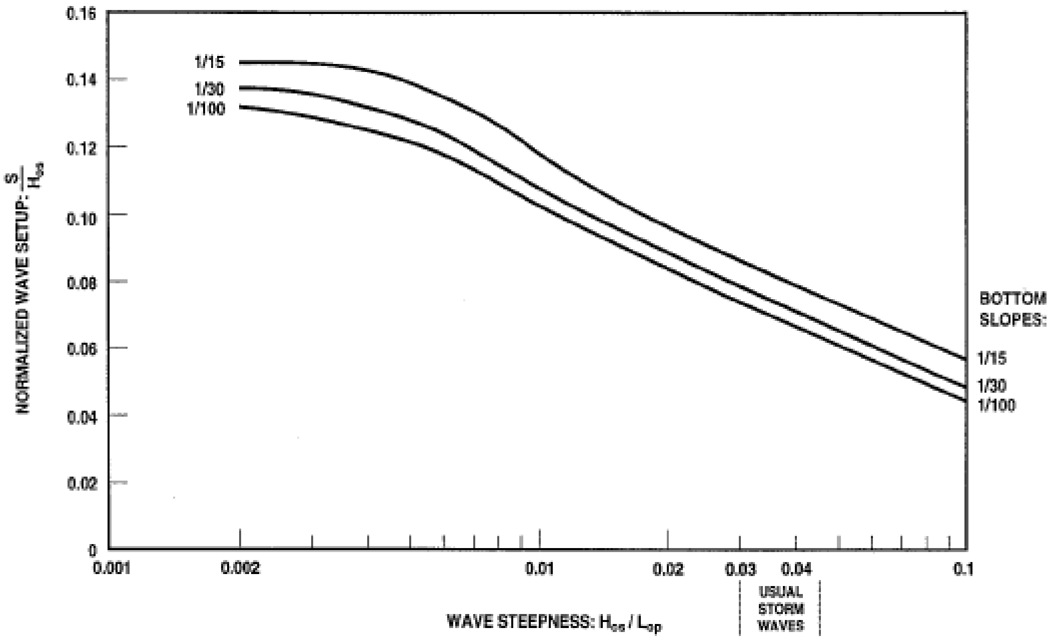
There are several methods for establishing static wave setup. One method uses the results described in the USACE Shore Protection Manual (SPM), which present normalized wave setup as a function of bottom slope and the deepwater wave steepness (H_o/L_o), as shown in Figure 2-1. (The symbol *S* for static wave setup in Figure 2-1 will be replaced by $\bar{\eta}$ here). Other methods include those developed by Goda (2000) and the Direct Integration Method (DIM), an integration of the governing equations. DIM was developed in conjunction with the development of the Pacific Coast Guidelines (FEMA 2004). The first two methods (SPM and Goda) yield a computation of wave setup at the landward limit of flooding, while the DIM method yields wave setup estimates at any point along a shore-normal transect.

A comparison analysis of these three methods was conducted by the Pacific Coast Guidelines technical working group (TWG). The TWG found that the DIM methodology yielded static wave setup values ranging from 60 to 100 percent larger than those from the SPM methodology. However, the DIM methodology values were less than 16 percent greater than those predicted by Goda. It was concluded by the TWG that the DIM methodology provides a better estimate of wave setup than the SPM methodology (FEMA 2004).

The Mapping Partner can use the DIM methodology to determine static wave setup. A reduction of up to 16 percent (based on the comparison with the Goda methodology) may be applied to the DIM results if evidence[1] suggests a reduction is appropriate.

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

Figure 2-1. Methodology for Calculating Wave Setup (from USACE SPM).



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

$$\bar{\eta} / H_o^{'} = 0.160 \frac{m^{0.2}}{(H_o^{'} / L_o)^{0.2}} \qquad \text{(Eq. 2-1)}$$

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

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

which simplifies to

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

where κ is the ratio of breaking wave height to breaking water depth. For the case of significant wave height and non-vegetated slopes, typical values of κ range from 0.4 to 0.6.[2] These values result in

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

(Eq. 2-4)

Procedures for calculating wave setup on an open coast will be presented, followed by cases of setup on levees, which entail modifications to the open coast method. As seen in Equation 2-1, wave setup calculations require a reference wave height. In this case, the effective deepwater significant wave height is H'_o.

2.2 Wave Setup on an Open Coast

2.2.1 Determining a Reference Deepwater Significant Wave Height

Estimation of the static wave setup requires an estimate of the deepwater significant wave height, which can be calculated or determined from hindcast data (such as that provided by the USACE Coastal and Hydraulics Laboratory Wave Information Studies (WIS) or other sources). WIS modeling stations are located continuously along the Atlantic and Gulf coasts. Because there are two primary statistical approaches for estimating storm surge elevations (Joint Probability Method (JPM) and Empirical Simulation Technique (EST)), two approaches are recommended to determine a reference deepwater wave height. The JPM methodology requires the development of synthetic storms in accordance with the historical database. For tropical storms, this involves calculating storm surges and waves based on a large number of synthetic storms. For nor'easters, the database may be better suited to the EST method or the use of a wave hindcast method based on the windfields used to generate the storm surge.

2.2.1.1 JPM—Wave Setup Due to Tropical Storms

The SPM provides recommendations for calculating the deepwater wave characteristics associated with a tropical storm. The SPM method includes two equations, one for the maximum significant wave height and one for the associated wave period. In addition, a graph is provided that represents the non-dimensional distribution of significant deepwater wave heights in a hurricane. The equations and graph are discussed below.

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[2] The values of κ cited here assume wave setup is due to wave breaking only (i.e., no reduction in wave setup due to vegetation – see Section 2.2.3.1) and waves are passing over a sloping surface without significant changes in slope. If the ground surface along the transect changes slope suddenly (e.g., a bluff or levee landward of a marsh) then the Mapping Partner may consider breaking the wave setup analysis into segments and calculating a different κ for each segment.

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

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

where the Coriolis parameter, f, is given by

$$f = 0.524 \sin\phi \quad \text{(Eq. 2-5)}$$

and ϕ is the latitude at the location of interest.

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

$$H'_{o,\max} = 16.5 e^{\frac{R\Delta p}{100}} \left[1 + \frac{0.208 \alpha V_F}{\sqrt{U_R}} \right] \quad \text{(Eq. 2-6)}$$

and

$$T_s = 8.6 e^{\frac{R\Delta p}{200}} \left[1 + \frac{0.104 \alpha V_F}{\sqrt{U_R}} \right] \quad \text{(Eq. 2-7)}$$

where

$$U_{\max} = 0.868 \left(73\sqrt{\Delta p} - 0.575 Rf \right) \quad \text{(Eq. 2-8)}$$

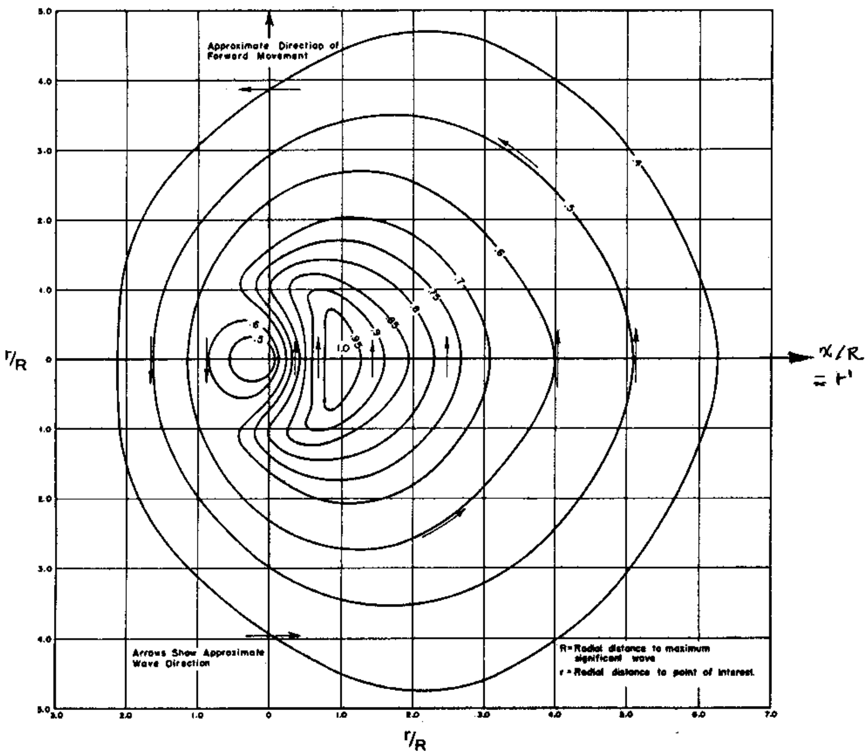
The parameter U_R, is expressed in terms of U_{max} as:

$$U_R = 0.865 U_{\max} + 0.5 V_F \,. \quad \text{(Eq. 2-9)}$$

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

Figure 2-2 presents the relationship for non-dimensional significant wave height as a function of non-dimensional distances relative to the tropical storm center. The distances are made non-dimensional by the tropical storm radius to maximum winds (*R*).

Figure 2-2. SPM Relationship for Wave Heights Relative to Their Maximum in a Tropical Storm (USACE).



As shown in Figure 2-2, the SPM model predicts waves that propagate in approximately the same direction as the local winds. For these purposes, wave height distributions are presented for two distances offshore, and it is recommended that the applied distribution be prorated by the actual distance of the tropical storm center from the shoreline. The two distributions are presented in Figure 2-3 along with the SPM distribution. The deviations from the SPM model are based on the recognition that waves diffract and disperse in advance of a hurricane. The two distributions are associated with the following positions: (1) distances of more than 4 radii from the shoreline, and (2) at the shoreline. Specifically, the recommended relevant deepwater wave heights at the shoreline are:

2.2.1.1.1 Tropical Storm Center More Than 4 Radii (R) From the Shoreline

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

$$H_o / H_{o,\max} = 0.40$$, $r' < -10$, $r' > 14$ (Eq. 2-10)

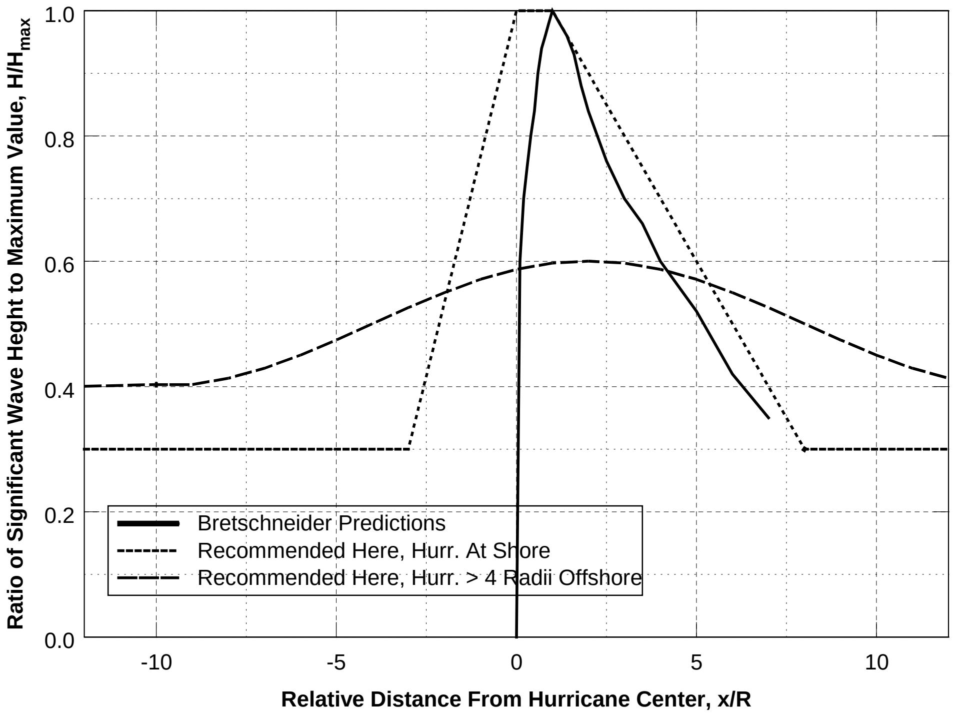
2.2.1.1.2 Tropical Storm Center At the Shoreline

$$
\begin{aligned}
&H_o / H_{o,\max} = 0.3, \quad r' < -3.0 \\
&H_o / H_{o,\max} = 0.3 + 0.233(r' + 3), \quad -3.0 < r' < 0 \\
&H_o / H_{o,\max} = 1.0, \quad 0 < r' < 1.0 \\
&H_o / H_{o,\max} = 1.0 - 0.10(r' - 1), \quad 1 < r' < 8 \\
&H_o / H_{o,\max} = 0.3, \quad r' > 6
\end{aligned}
$$

(Eq. 2-11)

where $r' = x / R$.

Figure 2-3. Recommended Relative Wave Height Along a Line Perpendicular to Tropical Storm Translation Direction



With the maximum significant wave height and associated period known along a line perpendicular to the tropical storm translation direction, the effective wave height at any location can be determined from the approximate graphical relationship in Figure 2-3 or Equations 2-10 and 2-11, which present local significant deepwater wave height relative to the global maximum deepwater significant wave height. The recommended effective wave period at all locations is given by Equation 2-7.

The effective profile slope (m) can be based on the average slope out to the breaking depth, which may be approximated by H'_o, and the static wave setup calculated by Equation 2-1.

2.2.1.2 EST - Wave Setup Due to Nor'easters

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

The method for determining a deepwater wave height in cases where the EST method is used to calculate wind surges differs only slightly from that of the JPM method. The difference is that historical storms, rather than synthetic storms, are used in the EST methodology. The general approach is to estimate the necessary parameters Δp, R, V_F, *etc.,* for each of the historical storms and then to apply the procedures presented for the JPM method to calculate static wave setup. The forward velocity (V_F) is determined from the path characteristics used in the simulation, so only the central pressure deficit (Δp) and the radius to maximum winds (R) need to be determined. The subsections below describe one approach to determine these variables. The Mapping Partner may evaluate other approaches.

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

The central pressure deficit (Δp) can be related approximately to the maximum wind (U_{max}) in the windfield used in Equation 2-8, which is provided below in a different form:

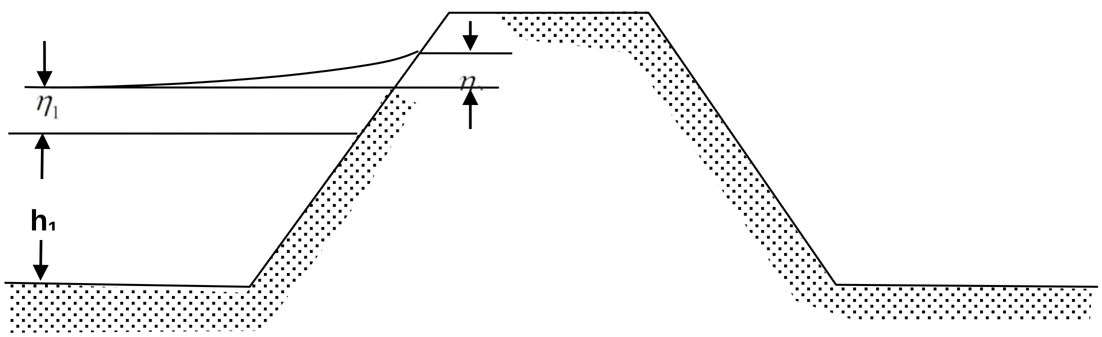
$$\Delta p = 1.88x10^{-4}\left(\frac{U_{\max}}{0.868} + 0.575Rf\right)^2$$ (Eq. 2-12)

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

2.2.2 Wave Setup On a Coastal Structure

The following subsections address the case of wave setup on a coastal structure. Figure 2-4 presents the case of a non-overtopped structure.

Figure 2-4. Definition Sketch for Non-Overtopped Structure

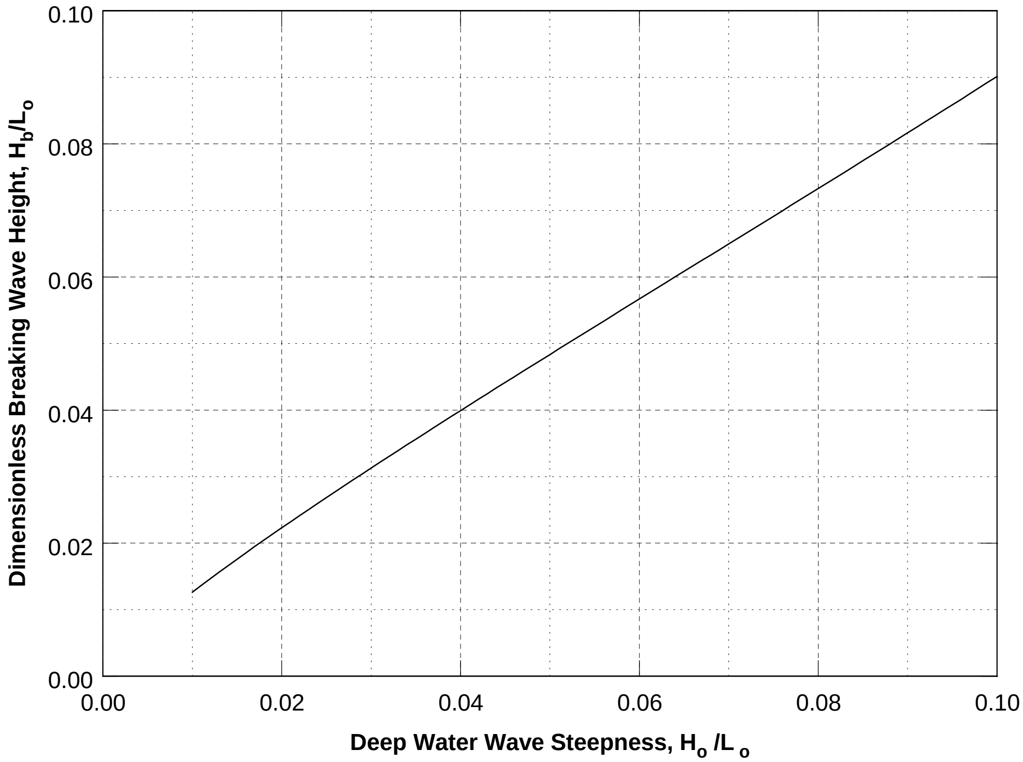


Because of the steep slopes associated with some coastal structures like breakwaters and seawalls, wave setup may be greater over this portion of the profile and should be treated separately. Referring to Figure 4, the setup must be considered in two components. The first setup component (η_1) is the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

2.2.3 Determining the Breaking Wave Height and Water Depth

It can be shown that the non-dimensional breaking wave height (H_o/L_o) is a function of the deepwater wave steepness (H'_o/L_o), as shown in Figure 2-5.

Figure 2-5. Dimensionless Breaking Wave Height vs. Deepwater Wave Steepness



The non-dimensional breaking wave height and water depth associated with the maximum local waves are based on the deepwater wave steepness (H'_o/L_o), where L_o=5.127^2 in the English system of units being used here. The breaking wave height differs from the deepwater wave height by ±10 percent at most, over the range plotted in Figure 2-5. Figure 2-6 presents the dimensionless breaking water depth (h_o/L_o), which will be useful later.

2.2.4 Non-Overtopped Structure

The wave setup at depth h_1 is determined by referring to Figure 2-7, which presents the proportion of wave setup that would occur in any depth proportional to the breaking depth (the latter determined from Figure 2-6). The value of η_2 is determined as

$$\eta_2 = 0.15(h_1 + \eta_1) \tag{Eq. 2-13}$$

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

Later examples will illustrate the application of these methods.

Figure 2-6. Dimensionless Breaking Water Depth vs. Deepwater Wave Steepness.

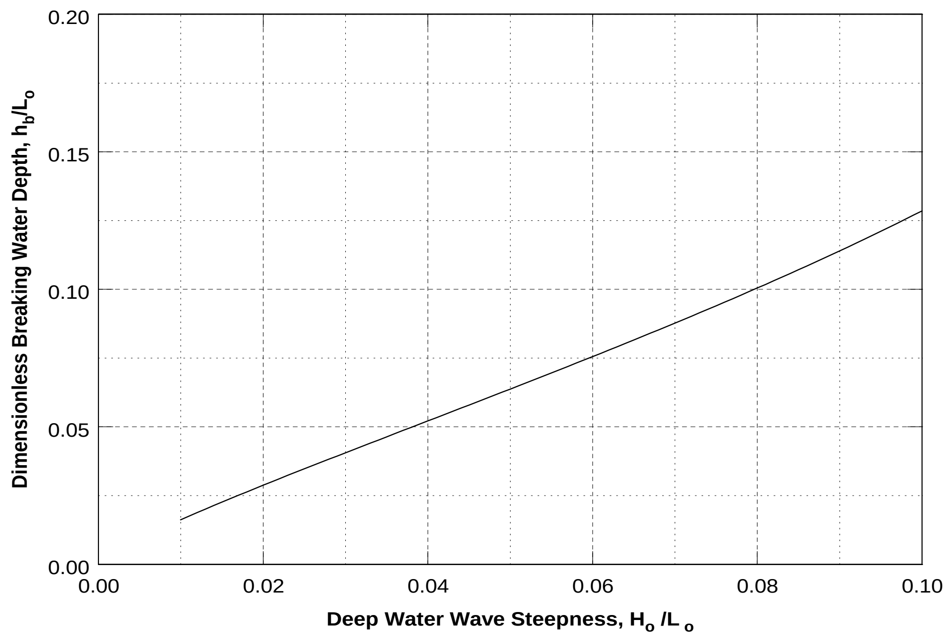
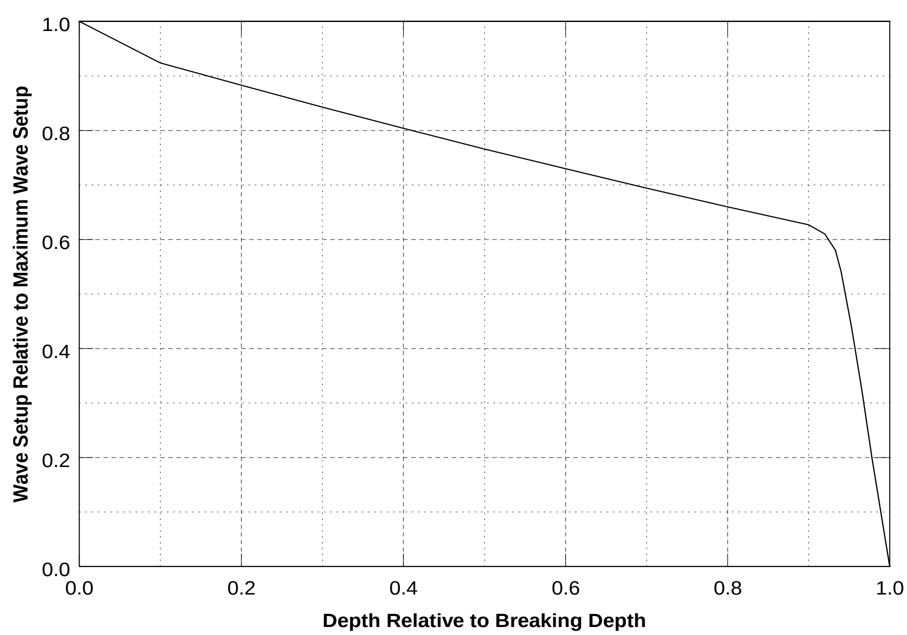


Figure 2-7. Proportion of Maximum Wave Setup that Has Occurred vs. a Proportion of the Breaking Depth.



2.2.5 Overtopped Structure

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

$$\eta_2 = 0.15(h_1 + \eta_1)\left[1 - \left(\frac{h_2}{h_1}\right)^2\right]$$

(Eq. 2-14)

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

2.2.6 Wave Setup—Special Cases

2.2.6.1 Vegetation and Bottom Friction Effects

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

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

2.2.6.2 Wave Setup across Barriers Islands and Large Bays

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

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

If, in the first case, storm surge inundates the entire barrier island or a large portion of the island, waves will pass over the island, possibly regenerate across the bay, and propagate onto the mainland. Wave setup in this case will increase as the overtopped barrier is approached, then will remain roughly constant across the bay, and will increase again as the waves break on the mainland (FEMA, 2004). The wave setup on the mainland may be higher than it would have been on a non-overtopped portion of the barrier, due to wave regeneration across the bay.

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

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

3.0 Pacific Coast

Wave, meteorological, and bathymetric characteristics are quite different from those on the Atlantic and Gulf coasts. The wave differences are due to the longer period waves and generally distant generation locations resulting in narrower spectra for the Pacific Coast whereas the meteorological differences are fewer hurricanes and thus lower winds. The major bathymetric differences are due to the relatively narrow Pacific Coast continental shelf widths. There are two major consequences of these differences for the 1% annual chance Pacific Coast hazards: (1) the wind surge component is relatively small due to the lower wind velocities coupled with the narrow shelf widths, and (2) the narrow spectra result in a substantial oscillating component of the wave setup with periods of tens to hundreds of seconds. Thus, the oscillating wave setup is a significant component of the total wave runup and a major contributor to coastal hazards on the Pacific Coast.

3.1 Pacific Coast Methods

Wave setup is the additional elevation of the water level due to the effects of transferring wave-related momentum to the surf zone. Momentum is transferred from winds to waves in the wave-generating area (usually in deep water for the Pacific Coast) and then is conveyed to shore by the waves similar to the manner that waves transport energy from the generating area to shore; see Figure 3-1. A main difference between energy and momentum is that energy is dissipated in the surf zone whereas momentum is transferred to the water column. This transfer is equivalent to a shoreward-directed “push” on the water column that causes a *tilt* of the water surface; see Figure 3-2. The wave setup is small and negative seaward of the surf zone (*setdown*) and begins to increase in the surf zone due to the transfer of momentum; see Figure 3-3. If only one wave of a constant height and period were present, the wave setup would be steady.

Wave setup and runup contribute significantly to the damage potential of severe waves along the Pacific Coast. The total runup, R, includes three components: (1) static wave setup, $\overline{\eta}$, (2) dynamic wave setup, $\hat{\eta}$, and (3) incident wave runup, R_{inc}, i.e., conceptually:

$$R = \overline{\eta} + \hat{\eta} + R_{inc} \quad \text{(Eq. 3-1)}$$

in which $\overline{\eta}$ and $\hat{\eta}$ are the magnitudes of the mean and oscillating wave setup components and R_{inc} is the runup component due to the incident waves. Guidance for calculating wave runup can be found in the Coastal Wave Runup Guidance document. In application, the two oscillating components ($\hat{\eta}$ *and* R_{inc}) are combined statistically to determine exceedance levels. Unless stated differently in this document, *R* refers to 2% runup conditions. The oscillating component of wave setup is a type of infragravity wave and is referred to here as *dynamic wave setup*. Each of the three components of total runup is defined and discussed below.

Figure 3-1. Schematic of Energy and Momentum Transfer from Winds to Waves within the Wave-generating Area, and to the Surf Zone and Related Processes.

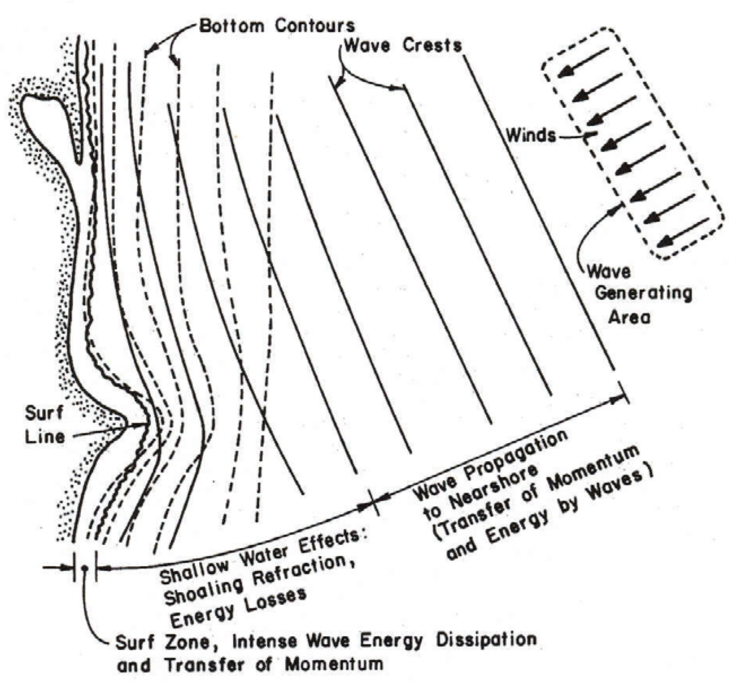


Figure 3-2. Wave Setup Due to Transfer of Momentum

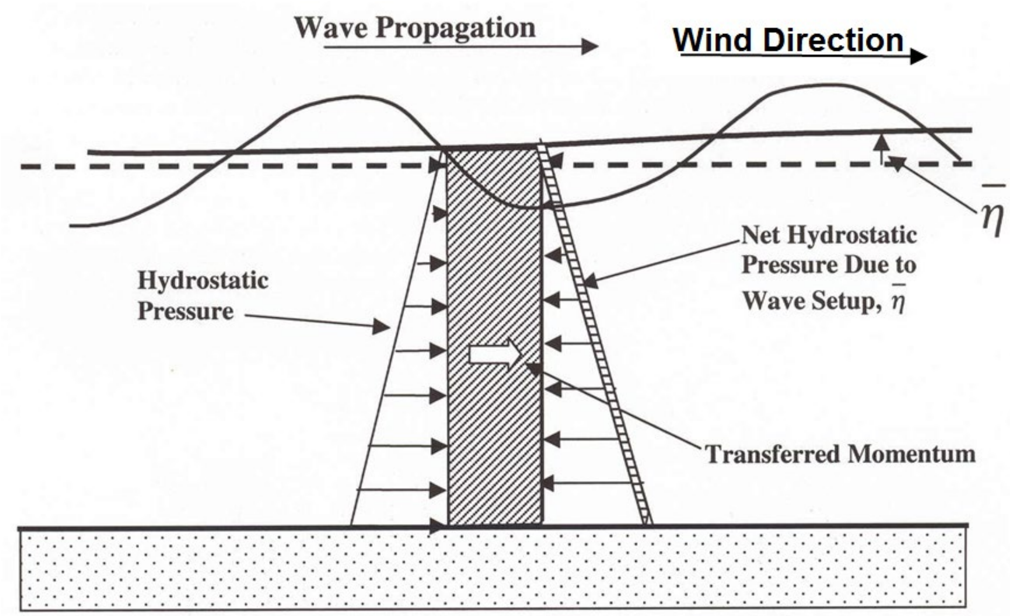
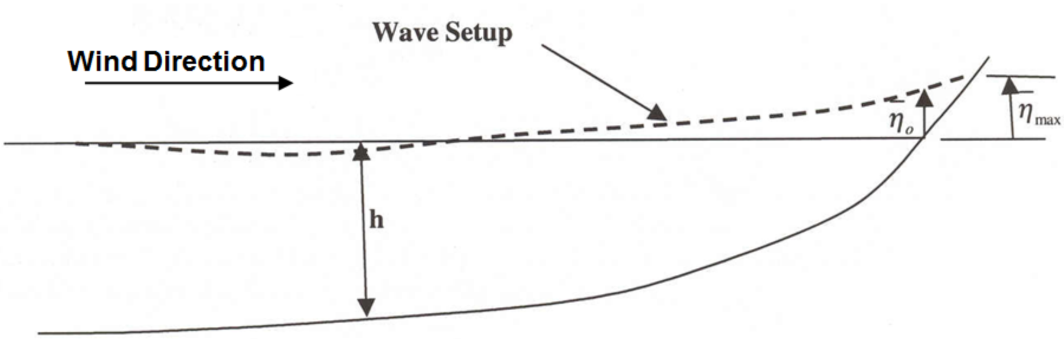


Figure 3-3. Static Wave Setup Definitions at Still Water Level, $\bar{\eta}_o$, and Maximum Setup, $\bar{\eta}_{\max}$



For a single wave component, the static setup, $\bar{\eta}(h)$, at any water depth, h, can be expressed as:

$$\bar{\eta}(h) = \left(-\frac{\kappa}{16} + \frac{(3/8)\kappa}{1+(3/8)\kappa^2}\right)H_b - \frac{(3/8)\kappa^2}{1+(3/8)\kappa^2}h \qquad \text{(Eq. 3-2)}$$

where κ is the ratio (assumed a constant) of the breaking wave height to water depth within the surf zone and h is the still water depth, i.e., the depth in the absence of waves or wave effects. The wave setup at the still water line, $\bar{\eta}_o$, and the maximum wave setup, $\bar{\eta}_{\max}$, can be expressed from Equation 3-2 in terms of the breaking wave height, H_b:

$$\overline{\eta_o} = (-\frac{\kappa}{16} + \frac{(3/8)\kappa}{1+(3/8)\kappa^2})H_b \qquad \text{(Eq. 3-3)}$$

The equivalent expression for the maximum wave setup, $\overline{\eta}_{\max}$, is:

$$\overline{\eta}_{\max} = \left\{ \frac{(-\frac{\kappa}{16} + \frac{(3/8)\kappa}{1+(3/8)\kappa^2})}{(1 - \frac{(3/8)\kappa^2}{1+(3/8)\kappa^2})} \right\} H_b \qquad \text{(Eq. 3-4)}$$

For the usual value of κ = 0.78, the following relations result:

$$\overline{\eta}(h) = 0.189H_b - 0.186h \qquad \text{(Eq. 3-5)}$$

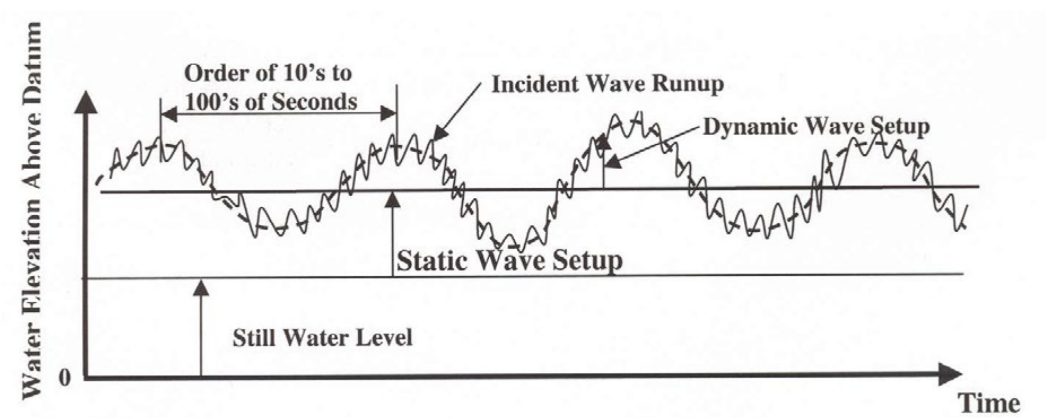
$$\overline{\eta_o} = 0.189H_b \qquad \text{(Eq. 3-6)}$$

$$\overline{\eta}_{\max} = 0.232H_b \qquad \text{(Eq. 3-7)}$$

More realistic wave-breaking models that account for the actual profile will usually reduce the wave setup for the relatively mild profile slopes of the Pacific Coast. For a wave system consisting of more than one wave component (i.e., a wave spectrum), the breaking wave height in the above expressions is replaced by the root mean square breaking wave height, $(H_b)_{rms}$.

Of significance on the Pacific Coast is that for wave systems consisting of more than one wave component, the setup is oscillating consisting of a steady and a so-called *dynamic component*; see Figure 3-4. The dynamic wave setup component is larger for narrower wave spectra and is substantial on the Pacific Coast during extreme storms and thus will require quantification for

Figure 3-4. Definitions of Static and Dynamic Wave Setup and Incident Wave Runup



flood mapping purposes. In addition to contributing to the total wave runup and thus the shoreward reach of the waves, dynamic wave setup can carry floating debris such as logs at high velocities and thus increase the damage potential in coastal areas. Figure 3-4 illustrates the three components that define the upper limit of wave effects.

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

$$\xi = \frac{m}{\sqrt{H/L}} \qquad \text{(Eq. 3-8)}$$

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

$$\xi_o = \frac{m}{\sqrt{H_o/L_o}} \qquad \text{(Eq. 3-9)}$$

where L_o is the deep water wave length:

$$L_o = \frac{g}{2\pi}T^2 \qquad \text{(Eq. 3-10)}$$

and *g* is the gravitational constant. The beach profile slope is the average slope out to the breaking depth associated with the significant wave height. Other definitions of the Iribarren number are defined later in this section as needed.

The term *still water level* (SWL) has an accepted definition in coastal engineering as the water level in the absence of wind waves and their effects and thus would include the astronomical tide, El Niño, and surge due to wind effects, but would not include either of the wave setup components. However, the wave setup components are included in the base water level for calculating wave runup and overtopping. Thus, the term *static water level* (STWL) is defined here as the sum of the SWL and the static wave setup, $\overline{\eta}$. Terminology is also useful to describe the sum of the static water level and a X% dynamic wave setup component. For purposes here, this will be defined as the *dynamic water level X%* (DWLX%). For example, the elevation corresponding to a 2% Dynamic Water Level would be the sum of the SWL (including astronomical tide, El Niño, and wind surge if present), the static wave setup, and the 2% dynamic wave setup. The term *reference water level* (RWL) is used as general terminology to refer to the water level that is appropriate for the particular application being discussed. The total water level (TWL) is the sum of the SWL, the wave setup, and wave runup.

3.2 General Input Requirements

The wave transformation element of these guidelines produces a nearshore shallow water wave spectrum outside the breaking zone and an equivalent deep water wave spectrum. The approaches detailed in the following subsections base the total wave runup on the equivalent deep water wave spectrum for the case of natural beaches or, for the case of runup on a barrier, the significant wave height at the toe of the barrier. To apply some of these methods, a parameterized (Joint North Sea Wave Project [JONSWAP]) spectrum is developed. The following wave characteristics are quantified: (1) equivalent deep water significant wave height, (2) peak wave period, and (3) spectral width (here spelled out as *Gamma* to avoid confusion with the Greek letter γ used to denote other parameters in this subsection). Large values of *Gamma* are associated with narrow spectra. Additionally, in some of the methods, an approximate uniform nearshore slope of the profile, m, must be established.

The deep water significant wave height and the peak period can be determined using the information provided from the wave transformation output. The recommended basis for determination of the spectral peakedness parameter (*Gamma*) is described below.

A parameter defined by Longuet-Higgins to quantify the spectrum narrowness (or peakedness) is based on the moments of the frequency spectrum, m_i, defined previously and refined below as Equation 3-11:

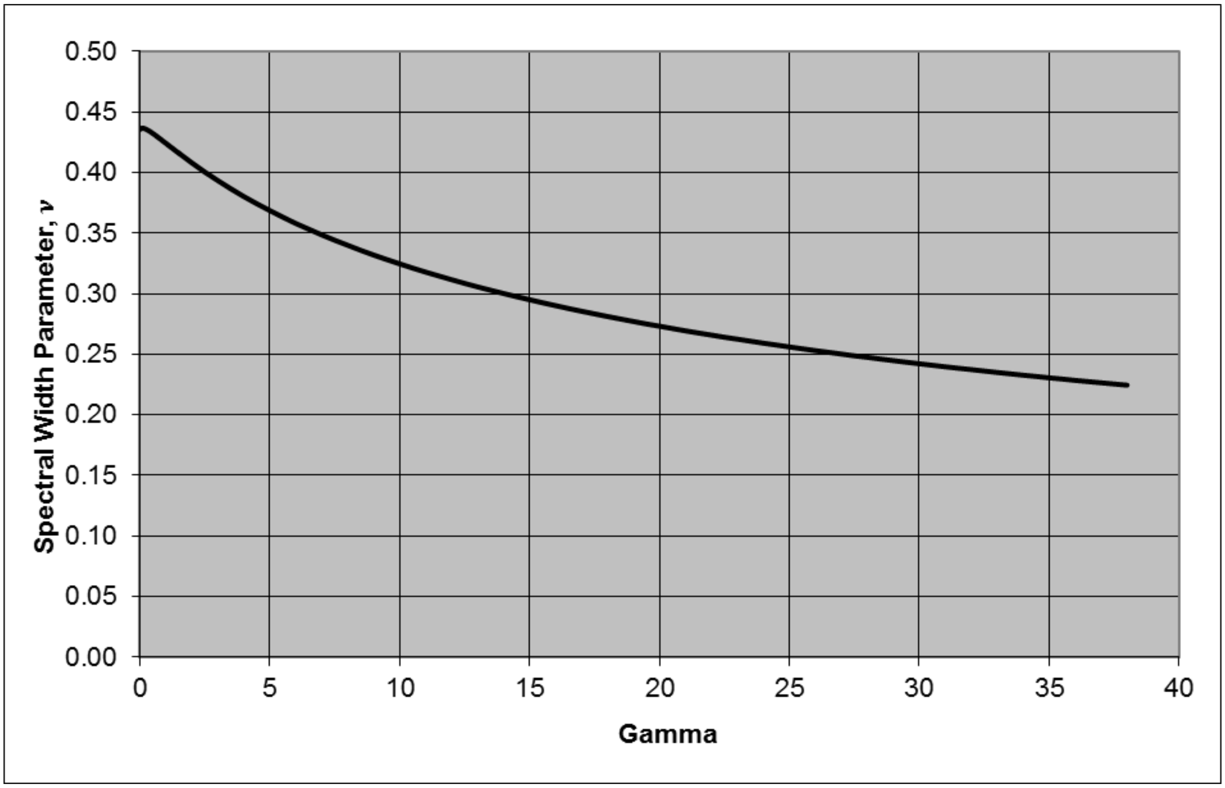
$$m_i = \sum_{n=1}^{N} f_n^i S(f_n) \qquad \text{(Eq. 3-11)}$$

where $S(f_n)$ is the wave energy at the discrete frequency, f_n . The Longuet-Higgins definition of the spectral narrowness, υ, is expressed in terms of the spectral moments:

$$\upsilon = \left[\frac{m_o m_2}{m_1^2} - 1 \right]^{1/2} \qquad \text{(Eq. 3-12)}$$

such that for an infinitely narrow spectrum, υ = 0. For purposes here, the two spectral peakedness parameters, υ and *Gamma*, have been plotted for JONSWAP spectra and the results are presented in Figure 3-5. The spectral moments, m_0, m_1, and m_2, for the actual equivalent deep water spectrum are provided from the wave transformation analysis effort, and υ is determined from Equation 3-12 and then *Gamma* determined from Figure 3-5 as input into the total wave runup methodology for the case of natural beaches.

Figure 3-5. Spectral Width Parameter Versus Gamma for JONSWAP Spectra



3.2.1 Setup and Runup on Beaches: Descriptions and Recommendations

A basic difficulty exists in applying the usual total runup equations to Pacific Coast conditions. The total runup shall include wave setup; however, when these equations are applied to approximate 1% annual chance Pacific Coast wave conditions, the total wave runup can be less than predicted for static and dynamic wave setup alone. This apparent paradox stems from the fact that most laboratory experiments on which these equations are based were conducted under conditions much different than those of concern on the Pacific Coast and the equations governing wave setup and incident wave runup have different dependencies on the variables (beach slope and wave characteristics) and thus the methods based on available experimental data cannot be extended outside the range of variables for which the experiments were conducted. Thus, it is necessary to account for this limitation of the usual equations for total wave runup in developing recommendations for the Pacific Coast.

The Direct Integration Method (DIM) was developed for calculating static and dynamic (infragravity) components of wave setup accounting for as much of the physics as possible. This one-dimensional method accounts for the spectral shape and the detailed bathymetry, and is based on integration of the governing equations from deep to shallow water. DIM can be applied by a simple set of empirical equations and by full implementation of the numerical model.

Three general approaches to address the wave setup components of the total wave runup on natural beaches are available: (1) empirical methods, (2) DIM developed in conjunction with this effort, and (3) advanced wave models, primarily the Boussinesq type. Because the dynamic wave setup is considered to be very significant on Pacific Coast shorelines and depends on the spectral width and DIM is the only method (other than the Boussinesq models) that can account for variable spectral width, DIM is the preferred method for application. As new models become available, the wave setup terms that contribute to total wave runup will be more explicitly described and may improve upon or replace DIM application.

3.2.2 Direct Integration Method

Because the DIM approach does not include the effects of incident wave runup, it is recommended that the 2% incident runup be incorporated and added statistically as discussed in more detail later. The recommended formulation is:

$$R_{inc} = F_R \xi_o H_o \qquad \text{(Eq. 3-13)}$$

The coefficient F_R in the above equation will differ for sandy beaches and barriers as discussed in the following subsections. The DIM approach allows the wave and bathymetric characteristics to be taken into consideration. Specifically, the spectral shape and actual bathymetry can be represented. A detailed discussion of the DIM program is presented in a User's Manual in the supporting documentation to this guidance document. Two applications of DIM are available to the Mapping Partner: the computer program and a set of equations. The equations are based on parameterized spectra (the JONSWAP spectrum that allows various spectral widths to be considered) and uniform profile slopes. The DIM program calculates the total wave setup and provides as output the static (average) wave setup, $\overline{\eta}$, and the root mean square (rms), η_{rms}, of the fluctuating wave setup around the average. Static and dynamic wave setup increase with wave period and the rms of the fluctuating setup component has been found to increase with the narrower spectra. The static setup component, $\overline{\eta}$, and rms of the dynamic setup component, η_{rms}, can be determined using the DIM program or the following equations:

$$\overline{\eta} = 4.0 F_H F_T F_{Gamma} F_{Slope} \qquad \text{(Eq. 3-14)}$$

and

$$\eta_{rms} = 2.7 G_H G_T G_{Gamma} G_{Slope} \qquad \text{(Eq. 3-15)}$$

where the units of $\overline{\eta}$ and η_{rms} are in feet and the factors are for wave height (F_H and G_H), wave period , (F_T and G_T), JONSWAP spectrum narrowness factor (F_{Gamma} and G_{Gamma}), and nearshore slope (F_{Slope} and G_{Slope}). These factors are defined in Table 3-1. With the exception of the spectral narrowness factors, the *F* and *G* factors are the same. The nearshore slope is the average slope between the runup limit and twice the break point of the significant wave height with the depth, h_b, at this point defined as $h_b = H_b / \kappa$. For purposes here, κ can be taken as 0.78. Because the wave setup components vary with the 0.2 power of this effective slope, these values are not overly sensitive to the value of effective slope.

Table 3-1. Summary of Factors to Be Applied with DIM

Variable	Wave Height	Wave Period	Spectral Narrowness	Nearshore Profile Slope
$\bar{\eta}$	$(H_o/26.2)^{0.8}$	$(T/20.0)^{0.4}$	1.0	$(m/0.01)^{0.2}$
η_{rms}	$(H_o/26.2)^{0.8}$	$(T/20.0)^{0.4}$	$(Gamma)^{0.16}$	$(m/0.01)^{0.2}$

In applying the DIM method (whether from the DIM program or from the equations and Table 3-1), it is necessary to develop the statistics of the oscillating wave setup and incident wave runup. This combination is based on the rms values (or standard deviations, σ) of each component. The standard deviation of setup fluctuations, $\sigma_1(\equiv \eta_{rms})$, is determined from the program or from the guidance provided in Table 3-1. The recommended standard deviation for the incident wave oscillations, σ_2, on natural beaches is given by:

$$\sigma_2 = 0.3\xi_o H_o \qquad \text{(Eq. 3-16)}$$

and the standard deviation associated with the relatively steep barriers is addressed later. With the two standard deviations (σ_1 and σ_2) available, the total oscillating contribution to the 2% total wave runup, $\hat{\eta}_T$, is determined as the combination of the two standard deviations of the fluctuating components, σ_1 *and* σ_2:

$$\hat{\eta}_T = 2.0\sqrt{\sigma_1^2 + \sigma_2^2} \qquad \text{(Eq. 3-17)}$$

The results of the computations using DIM suggest that the fluctuating component of the wave setup is normally distributed and that the maxima of the fluctuating component of wave setup are Rayleigh-distributed, similar to the general behavior found by Hedges and Mase (2004) in laboratory experiments of wave setup and wave runup.

3.3 Runup on Barriers

3.3.1 Special Considerations Due to Dynamic Wave Setup

Previous discussions have emphasized that a large wave runup event on the Pacific Coast is anticipated to have a more substantial dynamic wave setup than is present in the database on which available runup methods are based. Thus, special consideration is required in the calculation of wave runup and wave overtopping, which is described in the Coastal Wave Runup Guidance. The issue is to include the dynamic wave setup appropriately without double inclusion of the static and dynamic wave setup components that are inherent in the empirical database from which the runup and overtopping methodology were based. Table 3-2 describes the recommended methodology for both open coast and sheltered water settings. This methodology is illustrated through example calculations and separate supporting documentation.

Table 3-2. Recommended Procedure to Avoid Double Inclusion of Wave Setup Components

Case	Procedure
Open Coast, Sandy Beach	Apply DIM for wave setup with statistically combined incident runup, Equations 3-16 and 3-17
Open Coast, Coastal Barrier Present	Apply DIM for wave setup and reduce dynamic wave setup by amount considered to be most likely present in laboratory tests on which runup equations are based
Sheltered Waters, Sandy Beach	Same as open coast, sandy beach
Sheltered Waters, Coastal Barrier Present	Same as open coast, coastal barrier present

4.0 Great Lakes

For the Great Lakes region, wave setup can be a significant contributor to the total still water level (TSWL) (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.

4.1 Great Lakes Region Methods

4.1.1 Wave Setup Using a 1-D Surf Zone Model

Use of a one-dimensional surf zone dynamics model for transects, such as CSHORE, 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 the 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 cross-shore resolution that is on the order of meters.

4.1.2 Wave Setup Using Coupled 2-D Wave and Surge Model

The application of a basin-wide coupled 2-D wave and surge model results in a TSWL and it may not be necessary to compute wave setup outside the basin-wide modeling effort. However, this is only the case if sufficient resolution is adopted in the surf zone to compute wave setup accurately for all storms.

4.1.3 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 implicitly treat wave setup. DIM yields wave setup estimates at any point along a shore-normal transect.

The Atlantic and Gulf Coast guidance (Section 2.0) provides 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.