

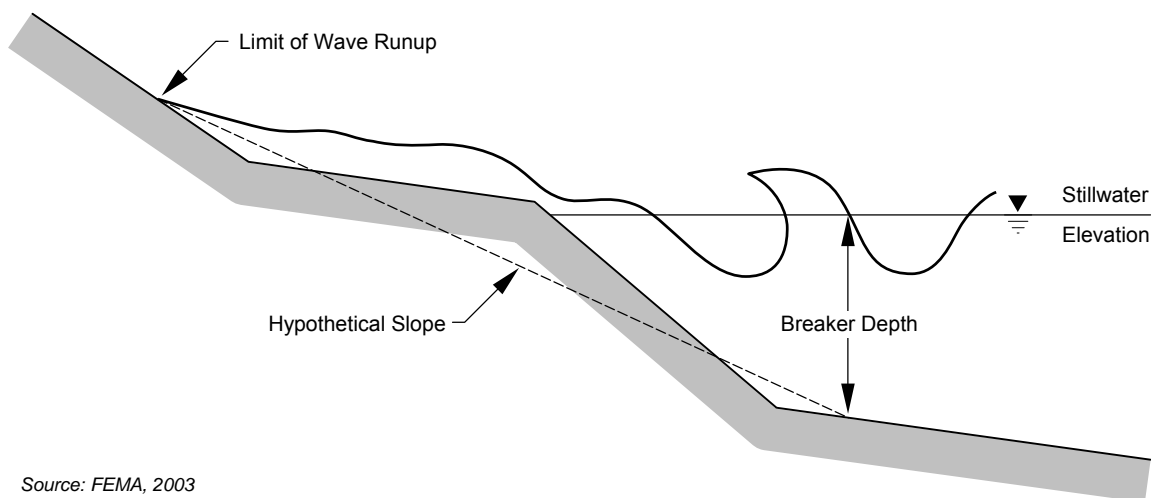
D.2.8 Wave Runup and Overtopping

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

D.2.8.1 Wave Runup

D.2.8.1.1 Overview

Wave runup is the uprush of water from wave action on a shore barrier intercepting stillwater level. The extent of runup can vary greatly from wave to wave in storm conditions, so that a wide distribution of wave runup elevations provides the precise description for a specific situation. The water wedge generally thins and slows during its excursion up the barrier, as residual forward momentum in wave motion near the shore is fully dissipated or reflected. The notable characteristic of this process for the present purposes is the wave runup elevation, the vertical height above the stillwater level⁶, ultimately attained by the extremity of the uprushing water. Wave runup at a shore barrier can provide flood hazards above and beyond those from stillwater inundation and incident wave geometry, as illustrated in Figure D.2.8-1.



Source: FEMA, 2003

Figure D.2.8-1. Wave Runup Sketch.

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

Current policy for the NFIP is to define the wave runup elevation as the value exceeded by 2 percent of the runup events⁷. The 2-percent value was chosen during the development of the Pacific Coast *Guidelines and Specifications for Flood Hazard Mapping Partners* (see Appendix D, Subsection D.4). This runup elevation is a short-term statistic associated with a group of waves or a particular storm. It is a standard definition of runup, commonly denoted as $R_{2\%}$. This 2 percent is different from the 1-percent-annual-chance condition that is associated with long-term extreme value statistics. The 1-percent condition has a 1-percent annual probability of occurrence, which corresponds approximately to the 100-year condition, while the runup statistic corresponds to a 2-percent exceedance occurrence in several hours of waves. To avoid confusion, the 2-percent runup is referred to as the “total runup” or just the “runup” and is denoted as $R_{2\%}$. Unless otherwise indicated, the runup referred to in all subsections of D.2 is the 2-percent runup.

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

$$\xi = \frac{m}{\sqrt{H/L}} \quad (\text{D.2.8-1})$$

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

$$\xi_o = \frac{m}{\sqrt{H_o/L_o}} \quad (\text{D.2.8-2})$$

where L_o is the deepwater wave length:

$$L_o = \frac{g}{2\pi} T^2 \quad (\text{D.2.8-3})$$

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

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

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

$$R_{inc} = 0.6 \frac{m}{\sqrt{H_o/L_o}} H_o \quad (\text{D.2.8-4})$$

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

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

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

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

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

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

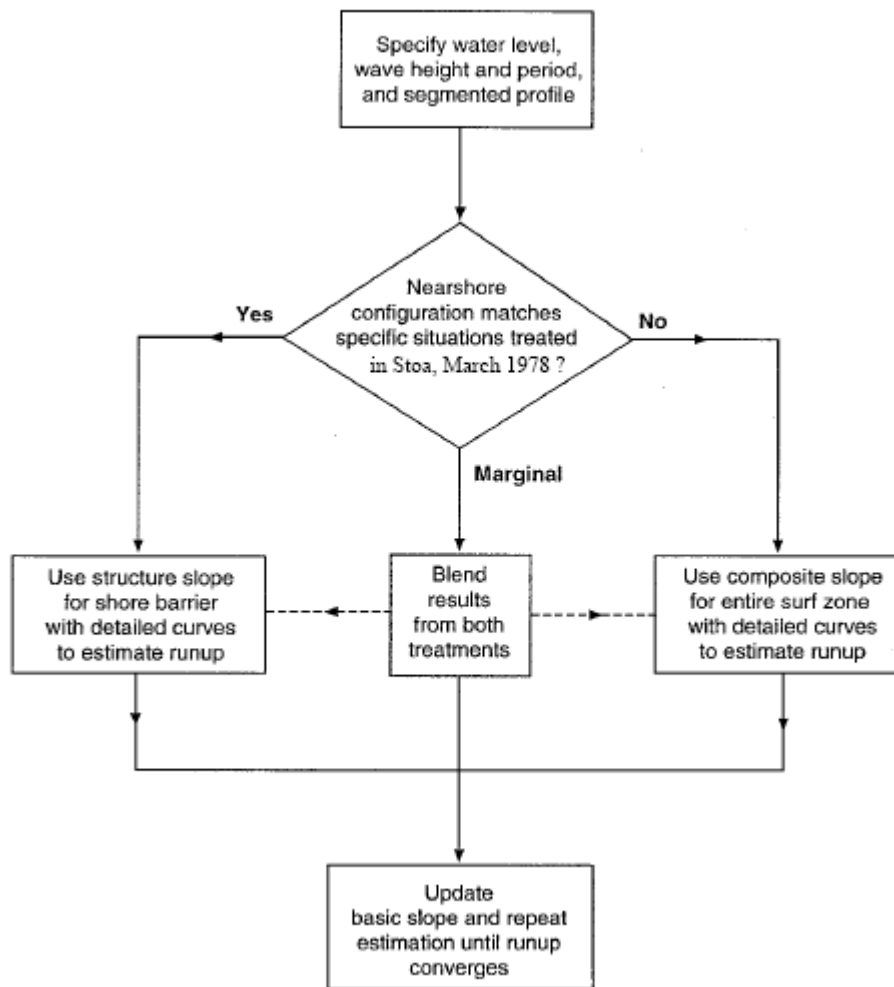


Figure D.2.8-2 Overview of Computation Procedure Implemented in FEMA Wave Runup Model (RUNUP 2.0)

D.2.8.1.2.1 RUNUP 2.0 Input Preparation

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

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

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

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

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

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

The mean wave condition to be specified for valid results with the Wave Runup Model may be derived from other common wave descriptions by simple relationships. Wave heights in

deepwater generally conform to a Rayleigh probability distribution, so that the mean wave height equals 0.626 times either the significant height based on the highest one-third of waves, or the zero-moment height derived from the wave energy spectrum. No exact correspondence between period measures exists; but the mean wave period can usually be approximated as 0.85 times the significant wave period or the period of peak energy in the wave spectrum.

Table D.2.8-2 lists a series of wave height and period combinations, of which one should be fairly suitable for runup computations at fully exposed coastal sites (depending on the local storm climate). These mean wave conditions have wave steepness values typical of U.S. hurricanes or within 30 percent of a fully arisen sea for extratropical storms. Commonly, the Mapping Partner may have some difficulty in specifying a precise wave condition as accompanying the base flood. In that case, the Mapping Partner shall also consider wave heights and periods 5 percent higher and 5 percent lower than those selected (or whatever percentages suit the level of uncertainty) and shall run the model with all nine combinations of those values. The average of computed runup values then provides a suitable estimate for mean runup elevation⁸. A wide range in computed runups signals the need for a more detailed analysis of expected wave conditions or for reconsideration of the transect representation.

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

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

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

D.2.8.1.2.2 RUNUP 2.0 Operation

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

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

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

Name Line

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

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

Job Description Line

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

Last Slope Line

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

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

Profile Lines

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

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

Water Level and Wave Parameter Lines

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

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

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

extreme events and also conform to the usual limits in detailed guidance on wave runup elevations.

- “DATA EXCEEDED TABLE” An entry into subroutine LOOK of the program is not within the parameter bounds of the data table from which a value is sought.
- “SOLUTION DOES NOT CONVERGE” After 10 iterations, the current and previous estimates of runup elevation continue to differ by more than 0.15 foot, and both values are provided in the output table. The calculation is usually oscillating between these two runup estimates when this occurs.
- “COMPOSITE SLOPE USED BUT WAVE MAY REFLECT, NOT BREAK” The output runup elevation relies to some extent on a composite-slope treatment, but the overall slope is steep enough that the specified wave may reflect from the nearshore barrier. Thus, the application of a calculated breaker depth in determining overall slope and runup elevation is questionable.
- “WARNING; COMPOSITE SLOPE USED, BUT INPUT PROFILE DOES NOT EXTEND TO BREAKER DEPTH” If the input profile does not extend seaward of the breaker depth, an incorrect breaker depth may be computed, and the associated runup elevation will also be incorrect. The input profile should include bathymetry to 30 or 40 feet in depth.

D.2.8.1.3 Wave Runup using ACES

FEMA also permits use of the Automated Coastal Engineering System (ACES, USACE, 1992) for runup and overtopping calculations against vertical and sloping structures. (Note that ACES v. 1.07 is on the FEMA list of accepted models for coastal wave effects, which can be found at http://www.fema.gov/plan/prevent/fhm/en_coast.shtm). It should also be noted that ACES uses more up-to-date methods than those contained in the *Shore Protection Manual* (USACE, 1984) or those used in RUNUP 2.0.

ACES v. 1.07 has three wave runup programs: *Irregular Wave Runup on Beaches*, *Irregular Wave Runup on Riprap*, and *Wave Runup and Overtopping on Impermeable Structures*. Wave setup contributions are included in each of the runup calculations.

The *Irregular Wave Runup on Beaches* module calculates several values of runup (R_{\max} , $R_{2\%}$, $R_{10\%}$, $R_{33\%}$, and \bar{R}) based on laboratory experiments of runup on smooth, impermeable slopes. The calculations are made given the deepwater significant wave height, peak wave period, and foreshore slope (which yield the surf similarity parameter, $\xi = \tan \theta / (H_o/L_o)^{1/2}$), and using the general relationship

$$\frac{R_{x\%}}{H_0} = a \xi^b \quad (\text{D.2.8-5})$$

where a and b are constants that depend on the statistic ($x\%$) desired, from Mase (1989).

The *Irregular Wave Runup on Riprap* calculation is part of the *Rubble-mound Revetment Design* module. This method calculates the expected maximum runup elevation and provides a conservative estimate of the maximum runup elevation, based on the small-scale laboratory tests of Ahrens and Heimbaugh (1988). The calculations are made given the deepwater significant wave height, peak wave period, and foreshore slope (which yield the surf similarity parameter), and using the general relationship

$$\frac{R_{\max}}{H_0} = a \xi / (1 + b \xi) \quad (\text{D.2.8-6})$$

where a and b are constants given by Ahrens and Heimbaugh (1989).

The *Wave Runup and Overtopping on Impermeable Structures* module calculates the runup elevation associated with incident uniform waves at the structure toe (described by $H_i = H_s$) acting on smooth or rough structures. Other inputs are the peak wave period, nearshore slope, structure slope, and roughness coefficients. The pertinent relationships are:

$$\frac{R}{H_i} = c \xi / (1 + d \xi) \quad (\text{D.2.8-7})$$

for rough slopes,

and

$$\frac{R}{H_i} = C \quad (\text{D.2.8-8})$$

for smooth slopes,

where c and d are the armor unit coefficients given by Ahrens and McCartney (1975), and coefficient C varies with the surf similarity parameter ξ , based on the work of Ahrens and Titus (1985).

The ACES runup modules represent improved guidance over that contained in the SPM (USACE, 1984). ACES guidance may be preferable to RUNUP 2.0 in some instances. The *Irregular Wave Runup on Beaches* calculation is maintained in the CEM. The *Irregular Wave Runup on Riprap* calculation is reported to be beneficial because it works well for both shallow water and deepwater at the toe of the revetment.

D.2.8.1.4 Runup on Vertical Structures

Basic empirical guidance incorporated within the RUNUP 2.0 computer model generally does not extend to vertical or nearly vertical flood barriers. For such configurations, RUNUP 2.0 will usually provide a runup elevation, but the result may be misleading because reliance on the composite-slope method can yield an underestimate of actual wave runup with the abrupt barrier. Where a vertical wall exists on a transect, the Mapping Partner shall develop a runup estimate

using the specific guidance in Figure D.2.8-3, taken from the *Shore Protection Manual* (USACE, 1984). As within RUNUP 2.0, these empirical results for uniform waves should be used by specifying the mean wave height and mean wave period for entry and taking the indicated runup as a mean value in storm wave action.

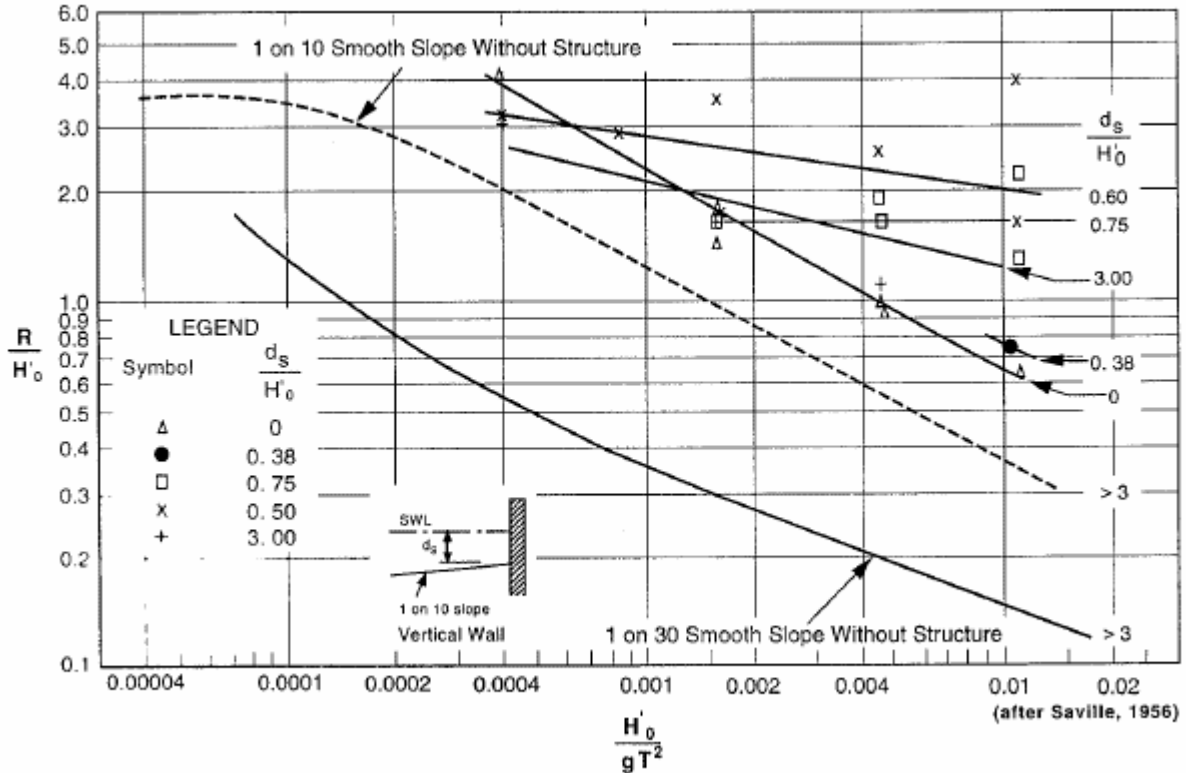


Figure D.2.8-3 Wave Runup Guidance from Vertical Wall, From Shore Protection Manual (USACE, 1984)

D.2.8.1.5 Methodology for Calculating Wave Runup on Barriers

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

The recommended approach to calculating wave runup on structures is based on the Iribarren number (ξ) and reduction factors developed by Battjes (1974), van der Meer (1988), de Waal and van der Meer (1992), and described in the CEM (USACE, 2003). This approach, referred to as the *Technical Advisory Committee for Water Retaining Structures* (TAW) method, is clearly

articulated in van der Meer (2002) and includes reduction factors for surface roughness, the influence of a berm, structure porosity, and oblique wave incidence. The TAW method is useful, as it covers a wide range of wave conditions for calculating wave runup on both smooth and rough slopes. In addition to being well documented, the TAW method agrees well with both small- and large-scale experiments.

It is important to note that other runup methods and equations for structures of similar form may provide more accurate results for a particular structure. The Mapping Partner shall carefully evaluate the applicability of any runup method to verify its appropriateness. Figure D.2.8-4 shows a general cross section of a coastal structure, a conceptual diagram of wave runup on a structure, and definitions of parameters.

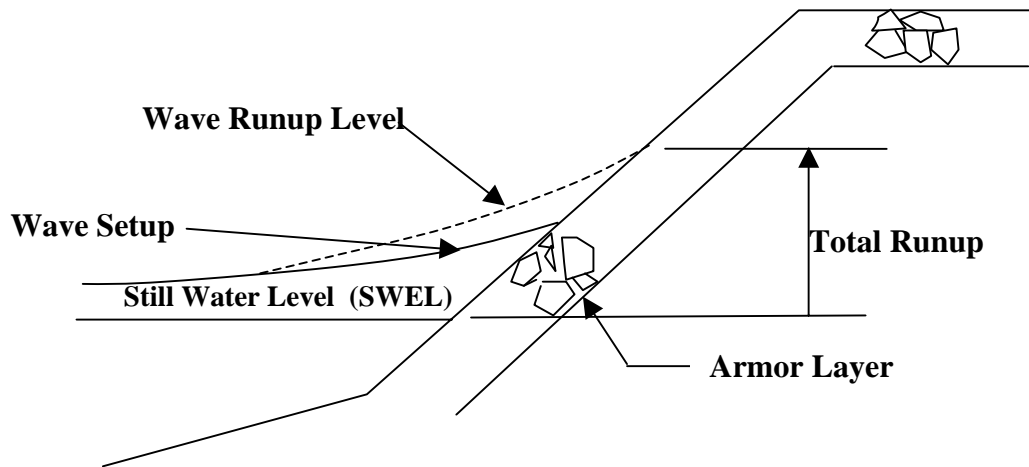


Figure D.2.8-4. Runup on Coastal Structures, Definition Sketch

Most of the wave runup research and literature shows a clear relationship between the vertical runup elevation and the Iribarren number. Figure D.2.8-5 shows the relative runup (R/H_{mo}) plotted against the Iribarren number for two different methods: van der Meer (2002) and Hedges and Mase (2004). In Figure D.2.8-5, both runup equations are derived from laboratory experimental data and are plotted within their respective domains of applicability for the Iribarren number. Each equation shows a consistent linear relationship between the relative runup and ξ_{om} for values of ξ_{om} below approximately 2. For values of ξ_{om} above approximately 2, only the van der Meer method is applicable. Moreover, due to its long period of availability and wide international acceptance, the van der Meer relationship (also referred to as the TAW runup methodology) is recommended here. The Mapping Partner shall characterize the wave conditions in terms of ξ_{om} and be aware of the runup predictions provided by the various methods available in the general literature.

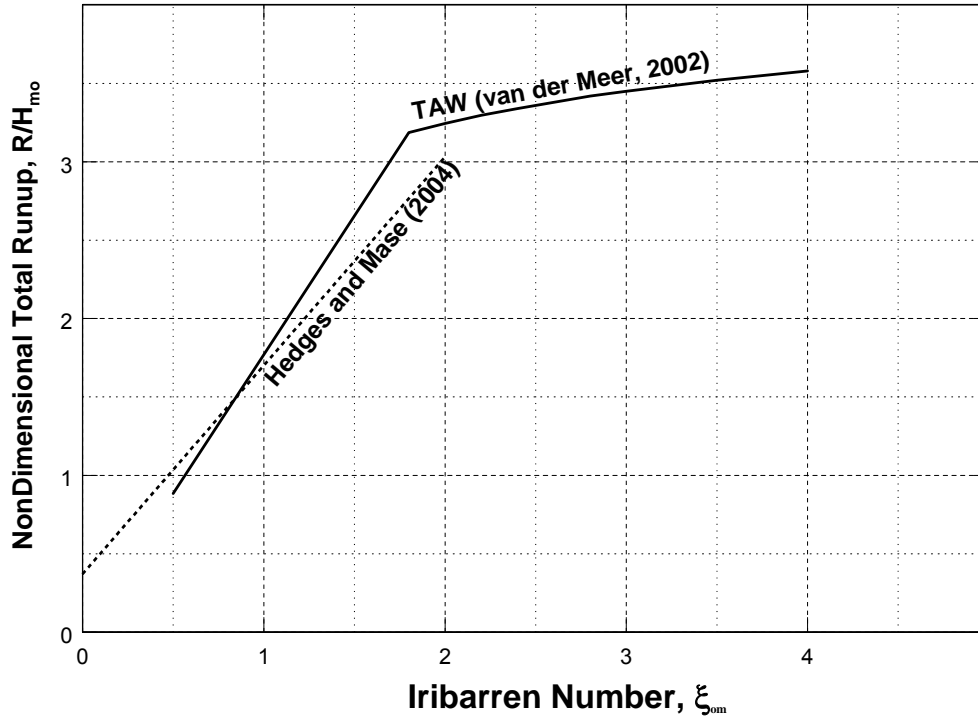


Figure D.2.8-5. Nondimensional Total Runup vs. Iribarren Number

The general form of the wave runup equation recommended for use (modified from van der Meer, 2002) is:

$$R = H_{mo} \left\{ \begin{array}{ll} 1.77 \gamma_r \gamma_b \gamma_\beta \gamma_P \xi_{om} & 0.5 \leq \gamma_b \xi_{om} < 1.8 \\ \gamma_r \gamma_b \gamma_\beta \gamma_P \left(4.3 - \frac{1.6}{\sqrt{\xi_{om}}} \right) & 1.8 \leq \gamma_b \xi_{om} \end{array} \right\} \quad (D.2.8-9)$$

where:

- R is the 2-percent runup = $2\sigma_2$
- H_{mo} = spectral significant wave height at the structure toe
- γ_r = reduction factor for influence of surface roughness
- γ_b = reduction factor for influence of berm
- γ_β = reduction factor for influence of angled wave attack
- γ_P = reduction factor for influence of structure permeability

Equations for quantifying the γ parameters are presented in Table D.2.8-5. The reference water level at the toe of the barrier for runup calculations is DWL2%. Additionally, because some

wave setup influence is present in the laboratory tests that led to Equation D.2.8-9, the following adjustments are made to the calculation procedure for cases of runup on barriers.

Table D.2.8-5. Summary of γ Runup Reduction Factors

Runup Reduction Factor	Characteristic/Condition	Value of γ for Runup
Roughness Reduction Factor, γ_r	Smooth Concrete, Asphalt, and Smooth Block Revetment	$\gamma_r = 1.0$
	1 Layer of Rock With Diameter, D. $H_s / D = 1$ to 3.	$\gamma_r = 0.55$ to 0.60
	2 or More Layers of Rock. $H_s / D = 1.5$ to 6.	$\gamma_r = 0.5$ to 0.55
	Quadratic Blocks	$\gamma_r = 0.70$ to 0.95. See Table V-5-3 in CEM for greater detail
Berm Section in Breakwater, γ_b , $B =$ Berm Width, $\left(\frac{\pi d_h}{x}\right)$ in radians	Berm Present in Structure Cross section. See Figure D.4.5-8 for Definitions of B , L_{berm} and Other Parameters	$\gamma_b = 1 - \frac{B}{2L_{berm}} \left[1 + \cos\left(\frac{\pi d_h}{x}\right) \right], 0.6 < \gamma_b < 1.0$ $x = \begin{cases} R \text{ if } \frac{-R}{H_{mo}} \leq \frac{d_h}{H_{mo}} \leq 0 \\ 2H_{mo} \text{ if } 0 \leq \frac{d_h}{H_{mo}} \leq 2 \end{cases}$ <p>(D.2.8-11)</p> <p>Minimum and maximum values of $\gamma_b = 0.6$ and 1.0, respectively</p>
Wave Direction Factor, γ_β , β is in degrees and $= 0^\circ$ for normally incident waves	Long-Crested Waves	$\gamma_\beta = \begin{cases} 1.0, 0 < \beta < 10^\circ \\ \cos(\beta - 10^\circ), 10^\circ < \beta < 63^\circ \\ 0.63, \beta > 63^\circ \end{cases}$ <p>(D.2.8-12)</p>
	Short-Crested Waves	$1 - 0.0022 \beta , \beta \leq 80^\circ$ $1 - 0.0022 80 , \beta \geq 80^\circ$ <p>(D.2.8-13)</p>
Porosity Factor, γ_P	Permeable Structure Core	$\gamma_P = 1.0, \xi_{om} < 3.3; \gamma_P = \frac{2.0}{1.17(\xi_{om})^{0.46}}, \xi_{om} > 3.3$ <p>and porosity = 0.5. for smaller porosities, proportion γ_P according to porosity . See Figure D.2.8-7 for definition of porosity</p> <p>(D.2.8-14)</p>

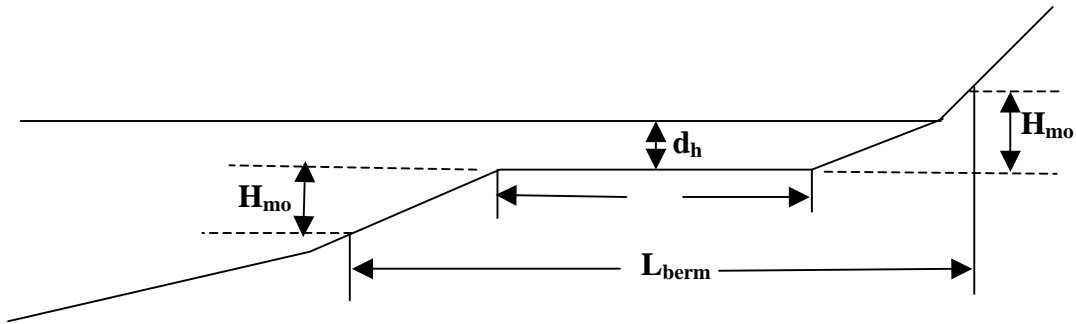


Figure D.2.8-6. Berm Parameters for Wave Runup Calculations

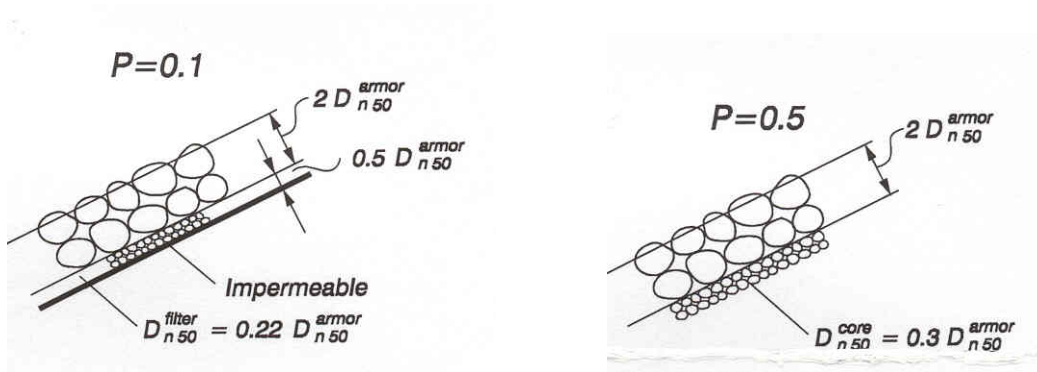


Figure D.2.8-7. Structure Porosity Definition

For a smooth, impermeable structure of uniform slope with normally incident waves, each of the γ runup reduction factors is 1.0.

In calculating the Iribarren number to apply in Equation D.2.8-9, the Mapping Partner shall use Equation D.2.8-2 and replace H_o with H_{mo} and replace T with $T_{m-1.0}$ (the spectral wave period). H_{mo} and $T_{m-1.0}$ are calculated as:

$$H_{mo} = 4.0\sqrt{m_o} \quad (D.2.8-15)$$

$$T_{m-1.0} = \frac{T_p}{1.1} \quad (D.2.8-16)$$

where H_{mo} is the spectral significant wave height at the toe of the structure and T_p is the peak wave period. In deepwater, H_{mo} is approximately the same as H_s , but in shallow water, H_{mo} is 10- to 15-percent smaller than the H_s obtained by zero up crossings (van der Meer, 2002). In many cases, waves are depth limited at the toe of the structure, and H_b can be substituted for H_{mo} , with

H_b calculated using a breaker index of 0.78 unless the Mapping Partner can justify a different value. The breaker index can be calculated based on the bottom slope and wave steepness by several methods, as discussed in the CEM (USACE, 2003). In terms of the Iribarren number, the TAW method is valid in the range of $0.5 < \xi_{om} < 8-10$, and in terms of structure slope, the TAW method is valid between values of 1:8 to 1:1. The Iribarren number as described above is denoted ξ_{om} , as indicated in Equation D.2.8-9.

Runup on structures is very dependent on the characteristics of the nearshore and structure geometries. Hence, better runup estimates may be possible with other runup equations for particular conditions. The Mapping Partner may use other runup methods, based on an assessment that the selected equations are derived from data that better represent the actual profile geometry or wave conditions. See the CEM (USACE, 2003) for a list of presently available methods and their ranges of applicability.

D.2.8.1.6 Runup from Smaller Waves

In some cases, neither of the previously described methods for computing runup on beaches or barriers is applicable. These special cases include steep slopes in the nearshore, with large Iribarren numbers or conditions otherwise outside the range of data used to develop the total runup for natural beach methods. Also, use of the TAW method is questionable where the toe of a structure, or a naturally steep profile such as a rocky bluff, is high relative to the water levels, limiting the local wave height and calculated runups to small values. In these cases, it is necessary to calculate runup with equations in the form of Equation D.2.8-9, to avoid double inclusion of the setup, and to carry out the calculations at several locations across the surf zone using the average slope in the Iribarren number. With this approach, it is possible that calculations with the largest waves in a given sea condition may not produce the highest runup, but that the highest runup will be the result of waves breaking at an intermediate location within the breaking zone.

The recommended procedure is to consider a range of (smaller) wave heights inside the surf zone in runup calculations. The concept of a range of calculated runup values is depicted schematically in Figure D.2.8-8, where an example transect and setup water-surface profile are shown. Figure D.2.8-8 also shows the corresponding range of depth-limited breaking wave heights calculated on the basis of a breaker index and plotted by breaker location on the shore transect. The Iribarren number was also calculated and plotted by breaker location in Figure D.2.8-8. The calculation of ξ at each location uses the deshoaled deepwater wave height corresponding to the breaker height, the deepwater wave length and the average slope calculated from the breaker point to the approximate runup limit. Note that this average slope, also called composite slope as defined in the CEM (USACE, 2003) and SPM (USACE, 1984), increases with smaller waves because the breaker location approaches the steeper part of the transect near the shoreline. This increases the numerator in the ξ equation. Also, the wave height decreases with shallower depths, reducing the wave steepness in the denominator of the ξ equation. Hence, as plotted in Figure D.2.8-8, ξ increases as smaller waves closer to shore are examined, increasing the relative runup (R/H). However, because the wave height decreases, the runup value (R) reaches a maximum and then decreases.

The following specific steps are used to determine the highest wave runup caused by a range of wave heights in the surf zone:

1. Calculate the runup using the methods described earlier for runup on a barrier. This requires iteration for this location to determine the average slope based on the differences between the runup elevation and the profile elevation at the location and the associated cross-shore locations. Iterate until the runup converges for this location.
2. Repeat the runup calculations at different cross-shore locations until a maximum runup is determined.

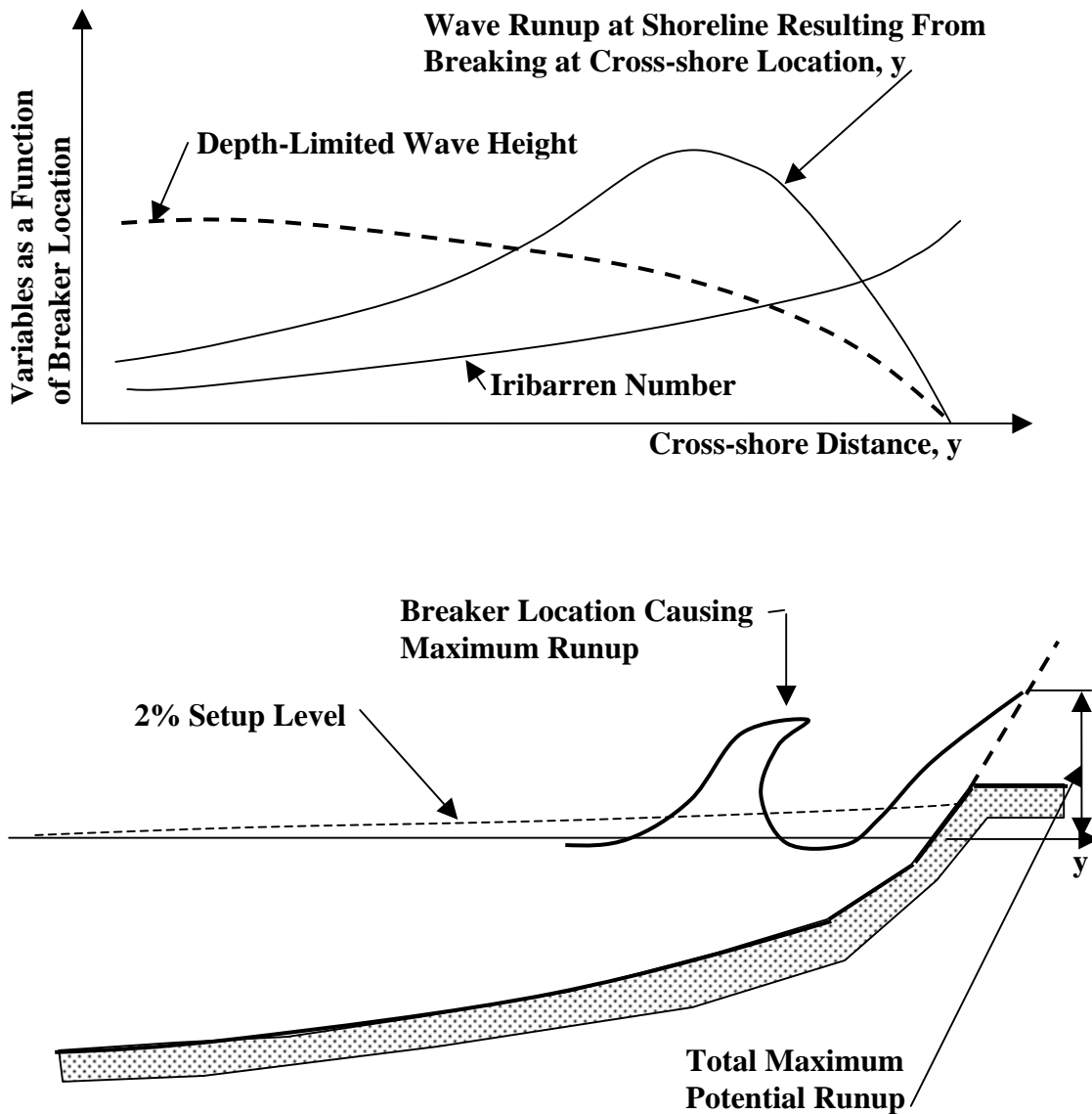


Figure D.2.8-8. Example Plot Showing the Variation of Surf Zone Parameters

D.2.8.1.7 Wave Runup in Special Situations

To interpret and apply the calculated results properly, the Mapping Partner shall examine the output of RUNUP 2.0 carefully for each situation. One important consideration is that a mean runup elevation below the crest of a given barrier does not necessarily imply that the barrier will not occasionally be overtopped by floodwaters (see Subsection D.2.8.2). Other cases may yield results of more immediate concern, in that RUNUP 2.0 may calculate a runup elevation exceeding the maximum barrier elevation; this outcome can occur because the program assumes the last positive slope to continue indefinitely. For bluffs or eroded dunes with negative landward slopes, a general rule has been used that limits the wave runup elevation to 3 feet above the maximum ground elevation, even when the potential runup along the imaginary slope extension exceeds 3 feet. When the runup overtops a barrier, such as a partially eroded bluff or a structure, the floodwater percolates into the bed and/or runs along the back slope until it reaches another flooding source or a ponding area. The runoff areas are usually designated as Zone AO, with a depth of 1, 2, or 3 feet. Ponding areas are designated as Zone AH (depth of 3 feet or less), with BFEs shown. Procedures for the treatment of sizable runoff and ponding are discussed in Section D.2.8.2.4. .

A fairly typical situation on the Atlantic and Gulf coasts is that wave runup exceeds the barrier top and flows to another flooding source, such as a bay, river, or backwater. It may not be necessary in this situation to compute overtopping rates and ponding elevations; only the flood hazard from the runoff must be determined. Simplified procedures have been used to determine an approximate depth of flooding in the runoff area (Williams, 1983). These procedures are illustrated in Figure D.2.8-9 and discussed below.

When the potential runup is at least 3.0 feet above the barrier crest, a VE Zone is delineated landward of the barrier, as shown in Figure D.2.8-9. The BFE for that VE Zone is capped at 3 feet above the crest of the barrier. When the runup depth in excess of the barrier crest is 0.1 to 1.5 feet, the VE Zone BFE is the runup elevation (rounded to the nearest whole foot), and an AO Zone with a depth of 1 foot should be mapped landward until another flooding source is encountered (Zone AE) or the floodplain limit is reached (Zone X). Similarly, for a runup depth of 1.5 to 2.9 feet above the barrier crest, the VE Zone BFE is the runup elevation (rounded to the nearest whole foot). In this case, however, an AO Zone with depth of 2 feet should be mapped, then transitioned landward into an AO Zone with a depth of 1 foot and then into subsequent flood insurance risk zones, if any.

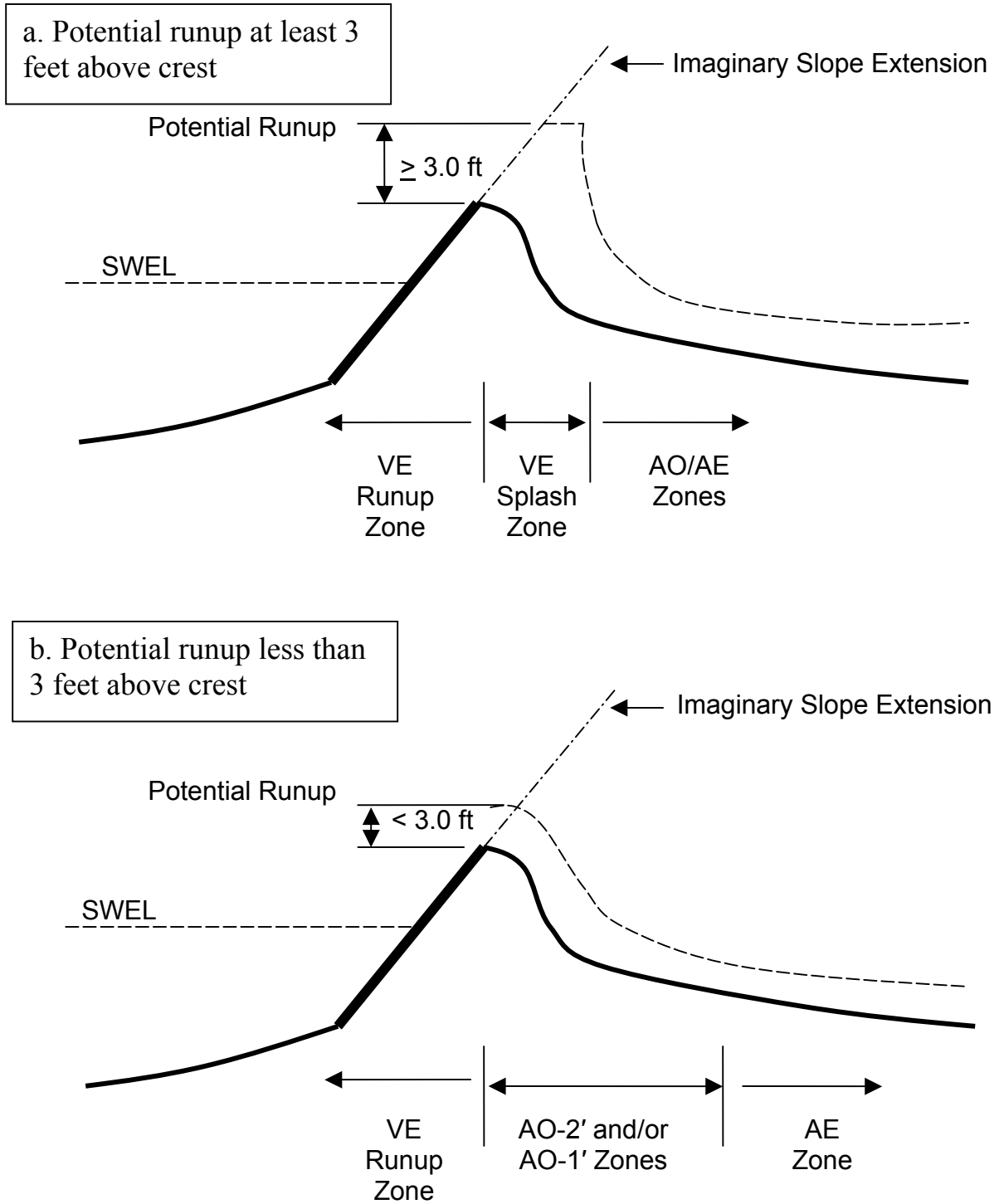


Figure D.2.8-9. Simplified Runoff Procedures (Zone AO)

A distinct type of overflow situation can occur at low bluffs or banks backed by a nearly level plateau, where calculated wave runup may appreciably exceed the top elevation of the steep barrier. A memorandum entitled *Special Computation Procedure Developed for Wave Runup Analysis for Casco Bay, FIS - Maine, 9700-153* provides a simple procedure to determine realistic runup elevations for such situations, as illustrated in Figure D.2.8-10 (French, 1982). An extension to the bluff face slope permits the computation of a hypothetical runup elevation for the barrier, with the imaginary portion given by the excess height $R' = (R - C)$ between the calculated runup and the bluff crest. Using that height (R') and the plateau slope (m), Figure D.2.8-11 defines the inland limit to a wave runup (X) corresponding to the runup above the bluff crest (mX) or an adjusted runup elevation of $R_a = (C + mX)$. This procedure is based on a Manning's "n" value of 0.04, with some simplifications in the energy grade line, and is meant for application only with positive slopes landward of the bluff crest. A different treatment of wave overflow onto a level plateau, for possible Flood Map Project use, is provided in *Overland Bore Propagation Due to an Overtopping Wave* (Cox and Machemehl, 1986).

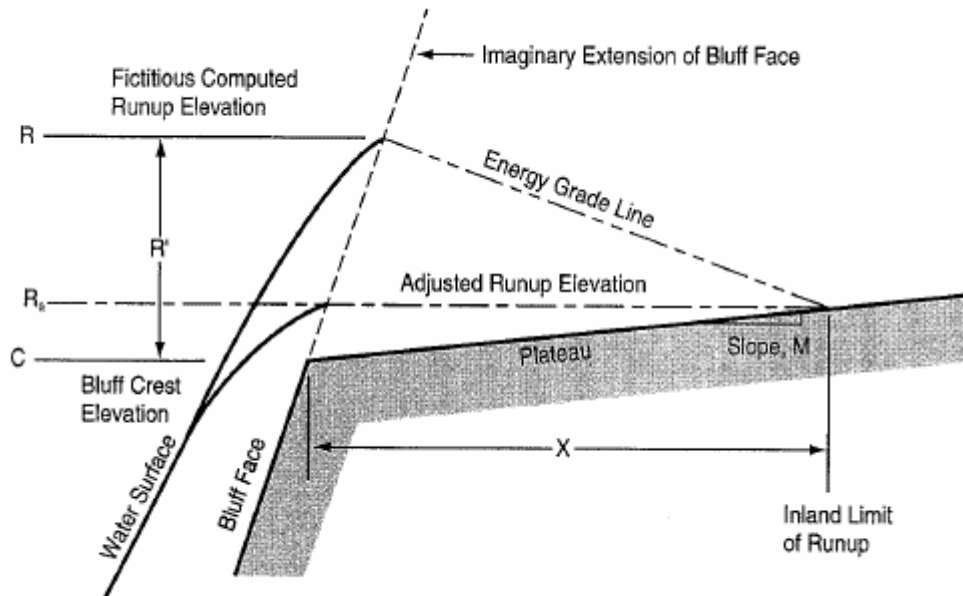


Figure D.2.8-10. Treatment of Runup onto Plateau above Low Bluff

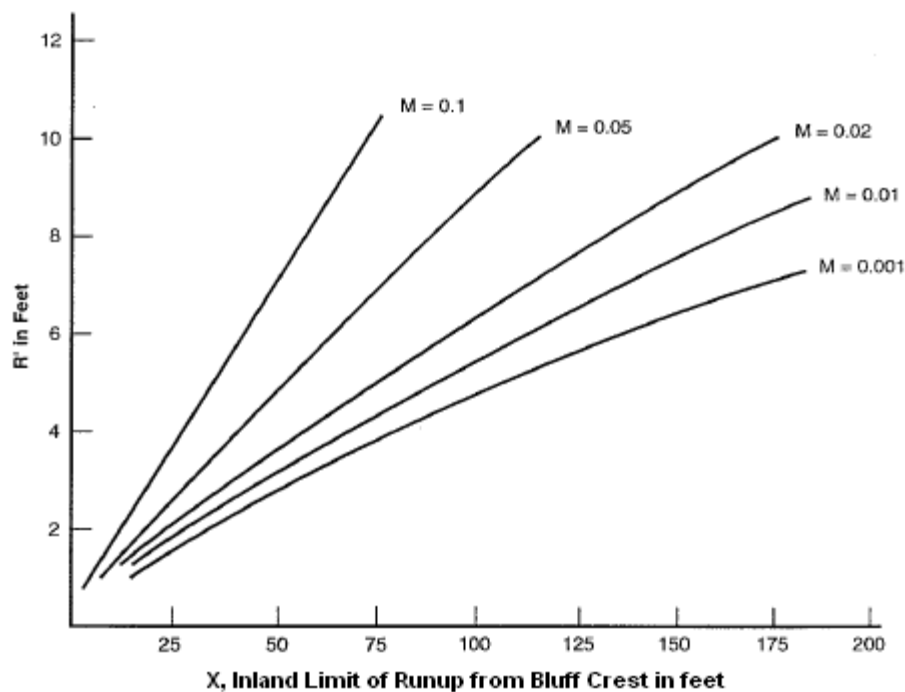


Figure D.2.8-11. Curves for Computation of Runup Inland of Low Bluffs

These runup assessment procedures are given for general guidance, but they may not be entirely applicable in certain situations. For example, runup elevations need to be fully consistent with the wave setup and wave overtopping assessments described in the 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. Subsection D.2.11 considers the integration of separately calculated wave effects into coherent hazard zones for the base flood. When a unique situation is encountered, the Mapping Partner shall prepare a Special Problem Report and discuss it with the FEMA Project Representative.

D.2.8.1.8 Advanced Wave Models

Wave models are becoming more sophisticated and able to account for the complexities of water waves. A rapidly developing class of these models is the Boussinesq group, which is both commercially and publicly available. The commercial models are generally more user friendly. In addition to wave setup, Boussinesq models can calculate wave runup. In conjunction with the development of these Guidelines and Specifications, 1-D Boussinesq models have been applied to calculate total wave runup, and the average and oscillating components were calculated separately. The comments below are based on an assessment of these Boussinesq results.

Compared to other methods, Boussinesq models yield generally realistic results. The main concern with Boussinesq modeling is the “learning curve” required to carry out these types of computations with confidence. Additionally, it was difficult to carry out calculations for deepwater waves with a small directional dependency. The reason for this difficulty lies in the associated substantial longshore wave lengths and the need for them to be represented by a 2-D

model. One possible FEMA application that would avoid the repeated learning curve requirement would be to carry out computations on a regional basis using Boussinesq models. The rate of the improvement/development of Boussinesq models is moderate at present; however, it is likely that this type of model will be much more capable in 10 to 20 years than at present. Thus, at this stage, a Mapping Partner may elect to apply Boussinesq models; however, for application on a regional basis, it is preferable to wait for further developments and improvements. If a Boussinesq model is applied, the Mapping Partner shall obtain FEMA's approval, and it is suggested that calculations also be carried out using the DIM methodology for comparison of results.

With these more advanced wave models, the wave setup component is combined with the storm-surge modeling, resulting in SWELs that include both storm surge and wave setup. Care must be taken not to double count wave setup when calculating wave runup with one of the methods presented here. The wave runup methods are based on scale laboratory tests that are thought to include wave setup, so that calculated runup values shall be added to the storm surge, excluding wave setup.

D.2.8.1.9 Documentation

The Mapping Partner shall document the procedures and values of parameters employed to establish the 1-percent-annual-chance total wave runup on the various transects on natural beaches and barriers, which could include steep dunes and structures. In particular, the basis for establishing the runup reduction factors and their values shall be documented. The documentation shall be especially detailed if the methodology deviates from that described herein and/or in the recommendations of the supporting documentation. Any measurements and/or observations shall be recorded, as well as documented or anecdotal information regarding previous major storm-induced runup. Any notable difficulties encountered and the approaches to addressing them shall be described clearly. Additional information on required documentation criteria can be found in Subsection D.2.12.

D.2.8.2 Overtopping (Open Coast and Sheltered Waters)

D.2.8.2.1 Overview.

Wave overtopping occurs when a barrier crest height is lower than the potential wave runup level, as shown in Figure D.2.8-12. Waves will flow or splash over the barrier crest, typically to an elevation less than the potential runup elevation (R'). The exact overtopping water surface and overtopping rate will depend on the incident water level and wave conditions and on the barrier geometry and roughness characteristics. Moreover, overtopping rates can vary over several orders of magnitude, with only subtle changes in hydraulic and barrier characteristics, and are difficult to predict precisely.

The assessment of potential wave overtopping for flood hazard mapping purposes must rely on readily available empirical guidance, historical effects, and engineering judgment. Except for very heavy overtopping, useful guidance is typically derived from laboratory tests with irregular waves, because the intermittently large overtopping discharges in storm situations are difficult to reproduce in the laboratory. Recent numerical modeling and field experiments are advancing the state of the art in overtopping predictions, but applying those methods in routine flood hazard mapping purposes is still problematic. Therefore, the Mapping Partner shall estimate only the

order of magnitude of mean overtopping rates, because there are clearly documented thresholds below which wave overtopping may be classified as negligible. While this approach does not account explicitly for highly variable peak overtopping rates and does not offer a complete specification of overtopping hazards, its use is recommended until overtopping rate calculation guidance is improved significantly.

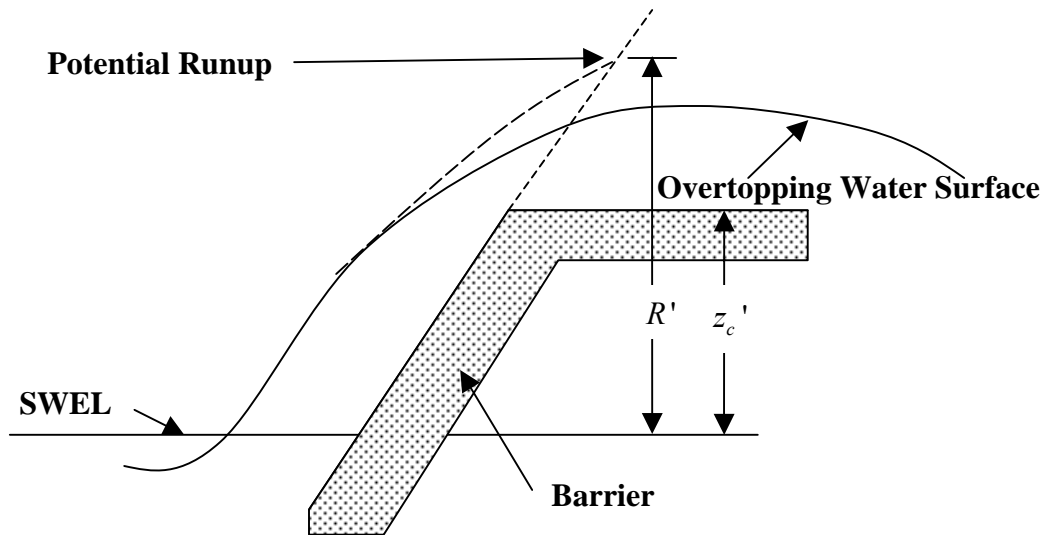


Figure D.2.8-12. Definition Sketch for Wave Overtopping

If a preliminary estimate indicates severe overtopping that threatens the stability of a given structure, that structure might be removed from the transect for analyses of the base flood, and further overtopping consideration may not be required. Two publications, *Design of Seawalls Allowing for Wave Overtopping* (Owen, 1980) and *Random Seas and Design of Maritime Structures* (Goda, 1985), appear to provide trustworthy and wide-ranging summaries of mean overtopping rates with storm waves. The former publication addresses smooth-plane or bermed slopes, and the latter publication considers vertical walls with or without a fronting rubble mound. Before surveying those primary sources of overtopping guidance, however, some introductory considerations can help to determine whether a detailed wave overtopping assessment is needed for base flood conditions at a specific shore barrier.

The initial consideration is an interpretation of the mean runup elevation already calculated (\bar{R}), in terms of likely extreme elevations according to the Rayleigh probability distribution usually appropriate for wave runups. To parallel the extreme wave height addressed in coastal studies (NAS, 1977), a controlling (base flood) runup magnitude may be defined as 1.6 times the significant runup, or 2.5 times the mean runup, according to the Rayleigh distribution.

The first overtopping calculation the Mapping Partner should make is a comparison of the potential mean runup (\bar{R}) to the freeboard (F) offered by the barrier. If the elevation of the barrier crest above the base flood total stillwater (MWL) elevation (freeboard) equals or exceeds

2.5 \bar{R} , then the landward area is considered not subject to wave overtopping discharges during the base flood. If $F \leq (2.0 \bar{R})$, then wave overtopping can be appreciable during the base flood, and the Mapping Partner shall assess overtopping rates and potential ponding behind the barrier. The extreme runups introduced here (2.0 \bar{R} and 2.5 \bar{R}) bracket the elevation exceeded by the extreme 2 percent of wave runups, which is a value commonly considered in structure design⁹.

D.2.8.2.2 Mean Overtopping Rates

Once the need for quantitative overtopping assessment is established, wave overtopping estimates for a specified situation must generally be based on measurements in a similar configuration. Before considering some implications of quantitative guidance for idealized cases, an overview of overtopping magnitudes gives a useful introduction (Goda, 1985; Gadd *et al.*, 1984).

Wave overtopping is often specified as a mean discharge: water volume per unit time and per unit alongshore length of the barrier, commonly in cubic feet per second per foot (cfs/ft). By interpreting or visualizing a given mean overtopping rate, the Mapping Partner may take into account actual discharges that are generally intermittent and isolated, being confined to some portion of occasional wave crests at scattered locations.

Distinct regimes of wave overtopping may be described as spray, splash, runup wedge, and waveform transmission, in order of increasing intensity. Flood discharges corresponding to those regimes naturally depend on the incident wave size, but certain overtopping rates have been associated with various characteristics (Goda, 1985). The right axis of Figure D.2.8-13 shows this association.

The mean overtopping rate of 0.01 cfs/ft seems to correspond to a value that generally should be considered appreciable, and a 1-cfs/ft mean overtopping rate appears to define an approximate threshold where the structural stability of even well-constructed shore barriers becomes threatened by severe overtopping. The 1-cfs/ft mean overtopping rate also appears to be well within the range where buildings exposed to overtopping are damaged.

Figure D.2.8-13 summarizes some empirical overtopping guidance for storm waves, in a schematic form meant to help Mapping Partners determine the likely significance of flooding behind a coastal structure. Variables describing the basic situation are cotangent of the front slope for a smooth structure with ideally simple geometry, and freeboard of the structure crest above total stillwater (mean water) level, as normalized by incident significant wave height (F/H_s). The mean overtopping rate (\bar{Q}) is provided in dimensionless form as

$$Q^* = \bar{Q}/(gH_s^3)^{0.5} \quad (\text{D.2.8-17})$$

with test results shown for structure slopes of 1:1, 1:2, and 1:4 (Owen, 1980), and for a smooth vertical wall (Goda, 1985). These results pertain to significant wave steepness of approximately $2\pi H_s/gT^2$
 $p = 0.035$, fairly appropriate for extreme extratropical storms or hurricanes; water depth near the

⁹ According to Walton (1992), the Rayleigh distribution would result in a 2-percent wave runup height that is approximately 2.2 times the mean runup and a maximum wave runup height (for levee analyses) that is approximately 2.9 times the mean wave runup.

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 interpolated curves is fixed as a maximum in overtopping rate for a structure slope of 1:2, corresponding to the gentlest incline producing (at this wave steepness) total reflection rather than breaking, and thus peak waveform elevations (Nagai and Takada, 1972).

These measured results for smooth and simple geometries clearly show severe or “green water” overtopping even at relatively high structures ($F \geq H_s$) for a wide range of common inclinations (cotangents between 0 and 4). Also, for freeboards considered here, a vertical wall (cotangent 0) permits less overtopping than common sloping structures with cotangent less than approximately 3.5. Gentler barriers are uncommon, because the construction volume increases with the cotangent squared, so steep coastal flood-protection structures usually face attenuated storm waves and/or have rough surfaces. The basic effects of those differences can be outlined for use in simplified overtopping assessments.

For sloping structures sited within the surf zone ($d_t < 2H_s$), *Design of Seawalls Allowing for Wave Overtopping* indicates that basic overtopping guidance in Figure D.2.8-13 can be used with attenuated rather than incoming wave height (Owen, 1980). A simple estimate basically consistent with other analyses of the base flood is that significant wave height is limited to

$H'_s = d_t/2$ at the structure toe. The value of $2F/d_t$ describes the effectively increased freeboard in entering Figure D.2.8-13, and the indicated Q^* value is then converted to \bar{Q} using H'_s .

The presumed wave attenuation ignores any wave setup as a small effect with the partial barrier, and d_t should always correspond to the scour condition expected in wave action accompanying the base flood.

Figure D.2.8-13 might also be made applicable to rough slopes, using a roughness coefficient (r) from Table D.2.8-1 to describe the effectively increased freeboard with greater wave dissipation on the structure. *Design of Seawalls Allowing for Wave Overtopping* proposed formulating effect of structure roughness as F/r , and *Beach and Dune Erosion during Storm Surges* confirmed a similar dependence of overtopping on roughness in measured results for irregular waves (Owen, 1980; Vellinga, 1986). The overtopping relation reported as reliable in *Wave Runup and Overtopping on Coastal Structures* is

$$Q^* = 8 \cdot 10^{-5} \exp[3.1(rR^* - F/H_s)] \quad (\text{D.2.8-18})$$

where $R^* = [1.5 m/(H_s/L_{op})^{0.5}]$, up to a maximum value of 3.0, is an estimated extreme runup normalized by H_s , for a barrier slope given as the tangent m (de Waal and van der Meer, 1992). Equation 3 is meant to pertain to very wide ranges of test situations with moderate overtopping, but it appears very approximate in comparison with specific results for $r = 1$, shown in Figure D.2.8-13. It may be advisable to evaluate Equation D.2.8.1-18 for both smooth and rough barriers, then to use the ratio to adapt a value from Figure D-19 for the case with roughness. *Design of Seawalls Allowing for Wave Overtopping* (Owen, 1980) and *Wave Runup*

and Overtopping on Coastal Structures (de Waal and van der Meer, 1992) provide further overtopping guidance on the effects of composite profiles, oblique waves, and shallow water with sloping structures.

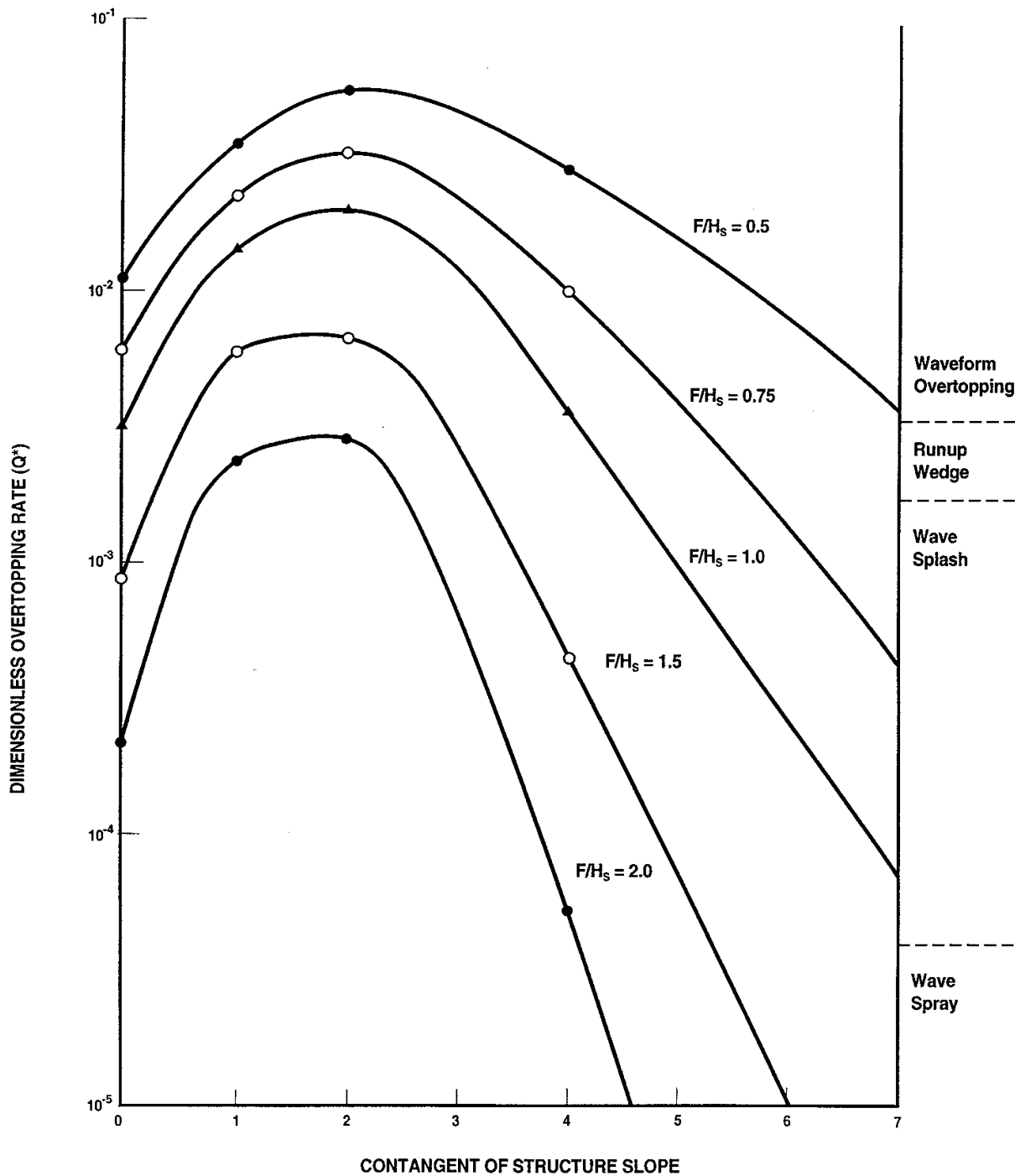


Figure D.2.8-13. Schematic Summary of Storm-Wave Overtopping at Structures of Various Slopes and Freeboards,

For overtopped vertical walls, the effects of wave attenuation appear relatively complex, but *Random Seas and Design of Maritime Structures* (Goda, 1985) provides extensive empirical guidance on various structure situations with incident waves specified for deepwater. Figure D.2.8-14 converts basic design diagrams for wave overtopping rate at a vertical wall, to display wall freeboard required for rates of 1 cfs/ft and 0.01 cfs/ft with various incident wave heights. Goda (1985) also provides a convenient summary on the effect of appreciable fronting roughness in storm waves: the required freeboard of a smooth vertical wall for a given overtopping rate is approximately 1.5 times that needed when a sizable mound having concrete block armor is installed against the wall. With this information, a specific vertical wall can be categorized as having only modest overtopping ($\bar{Q} < 0.01$ cfs/ft), intermediate overtopping, or severe overtopping ($\bar{Q} > 1$ cfs/ft) expected for the base flood. Likely runoff or ponding behind the wall must then be identified; severe overtopping requires a delineation of the landward area susceptible to wave action and velocity hazard.

Considering Figure D.2.8-14 with respect to common wall and wave heights, wave overtopping that endangers structural stability appears usual during the base flood.

An assessment of failure during the base flood for typical walls would be fully consistent with one recommendation of *Criteria for Evaluating Coastal Flood-Protection Structures*, which states that “FEMA not consider anchored bulkheads for flood-protection credit because of extensive failures” (Walton et al., 1989).

D.2.8.2.3 Overtopping Rate Considerations for Establishing Flood insurance risk zones

An interpretation of the estimated overtopping rate in terms of flood hazards is complicated by the projected duration of wave effects, the increased discharge possible under storm winds, the varying inland extent of water effects, and the specific topography and drainage landward of the barrier. However, Table D.2.8-6 provides guidance that is potentially applicable to typical coastal situations.

Table D.2.8-6. Suggestions for Interpretation of Mean Wave Overtopping Rates

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

*With estimated \bar{Q} much greater than 1 cfs/ft, removal of barrier from transect representation may be appropriate.

+Appropriate inland extent of velocity hazards should take into account barrier characteristics, incident wave conditions, overtopping flow depth and velocity, and other factors.

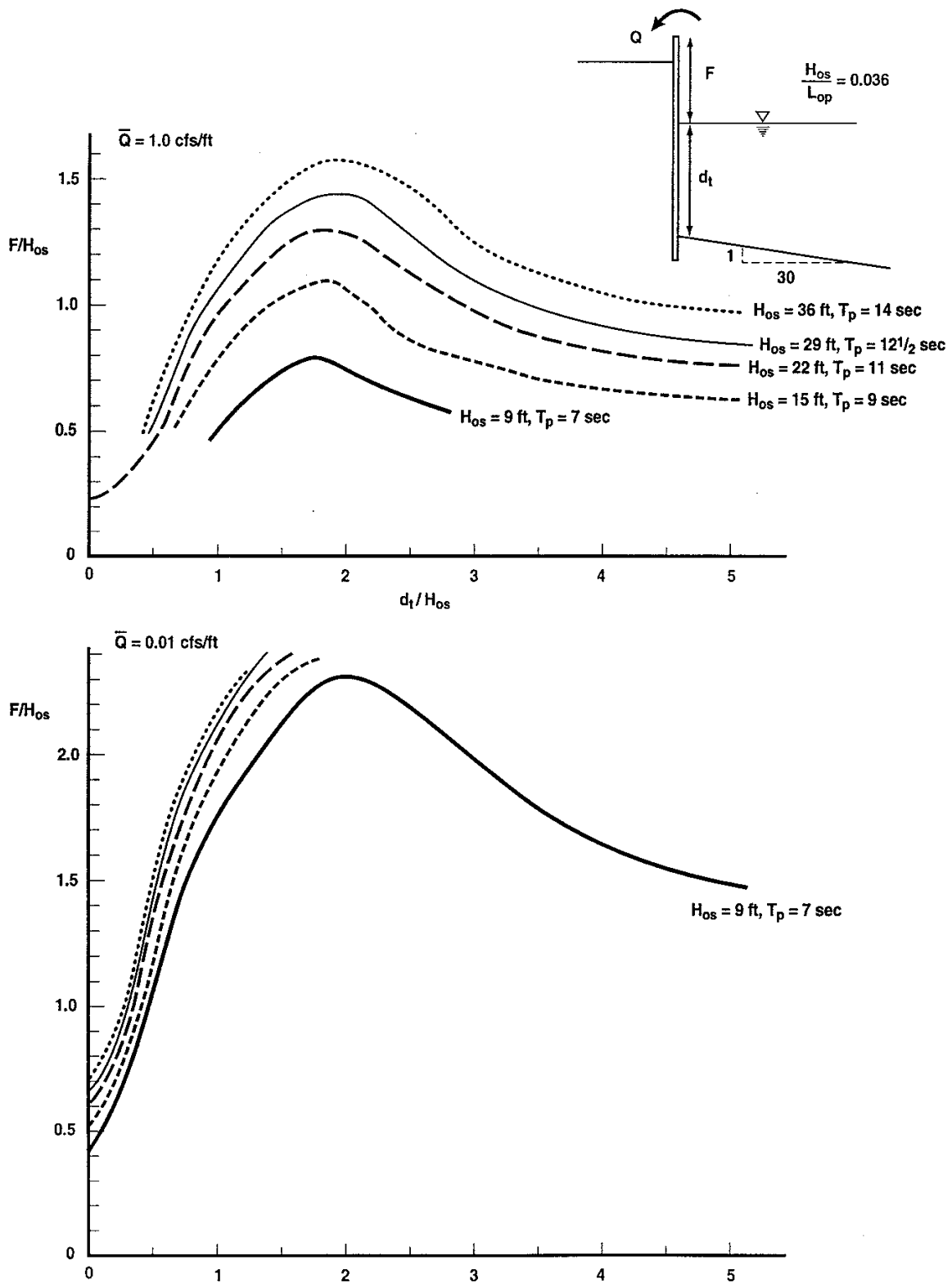


Figure D.2.8-14. Required Freeboard of Vertical Wall to Limit Mean Overtopping Rate to Certain Values, Based on Design Curves of Random Seas and Design of Maritime

D.2.8.2.4 Ponding Considerations

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

The duration of overtopping can vary widely, depending on the coastal flood cause, from a fast-moving hurricane to a nearly stationary extratropical storm. The final guidance is offered: a minimum assumption for the duration of flood-peak overtopping would generally be 2 hours. Durations of 10 hours or more could be appropriate for the cumulative effects of an extratropical storm causing flooding over multiple high tides.

D.2.8.2.5 Overtopping Depth and Velocity Considerations

In cases where the potential runup exceeds a barrier crest by 3.0 feet or more, the Mapping Partner will map a VE splash zone landward of the crest (see Subsection D.2.8.1.7). The Mapping Partner may consider the overtopping depth and velocity as one factor to determine the landward limit of the VE splash zone.¹⁰

¹⁰ This new mapping procedure was introduced in the *Pacific Mapping Guidelines*. More details are provided in Sections D.4.9.2.1 and D.4.5.2.5.