Guidance for Flood Risk Analysis and Mapping

Coastal Wave Runup and Overtopping

November 2023

FEMA

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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 (https://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 https://www.fema.gov/resource-document-library.

Table of Revisions

Affected Section or Subsection	Date	Description
Appendix A	November 2023	Changed previous Section 7 to separate Appendix.
Appendix A – Section 1.2	November 2023	Added supporting information for CSHORE input parameters.
Appendix A – Section 2	November 2023	Added runup and overtopping equations including defined variables and reference information. Provided the SPM vertical runup figure, which was referenced in superseded guidance. Provided clarification on wave setup application to runup methods.
Throughout document	November 2023	Incorporated Eurotop as an approved runup methodology. Provided runup conversions. Incorporated “Runup for Small Waves” text from superseded guidance. Restructured some document content.

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1. Overview and Purpose

Wave runup and overtopping are wave-induced flood hazards that occur along coastal areas where waves encounter the shoreline and break, resulting in an uprush of water. Figure 1 shows a typical cross-shore profile divided into four coastal zones that are based on different physical processes. Wave runup and overtopping are complex physical processes occurring in the surf and backshore zones. They depend on the local water level, incident wave conditions, and the nature of the beach or structure encountered. In narrow, developed coastal floodplains, the process of high-velocity wave runup and overtopping puts residential and non-residential structures located above the storm surge levels along the shorelines at an increased flood risk. In these floodplains, wave runup elevations and overtopping rates determine 1% annual-chance Base Flood Elevations (BFEs) as well as flood zone designations and spatial extents of the Special Flood Hazard Area (SFHA).

This guidance document presents the physical processes and theory behind the analysis of wave runup and overtopping. This document also provides a framework for evaluating different methods of analysis and the considerations and best practices for each method when performing Federal Emergency Management Agency (FEMA) coastal flood hazard studies in different physical settings.

This document is not intended to be prescriptive or procedural, as there is sufficient guidance within the coastal engineering profession addressing wave runup and overtopping calculation protocols. This document is therefore intended to provide an overarching framework for assessing and mapping coastal flood hazards due to wave runup and overtopping for the production of FEMA Flood Insurance Rate Maps (FIRMs) and Flood Insurance Studies (FISs). It is recommended that the information presented herein be utilized by professionals with a high degree of engineering expertise and understanding of coastal processes. It should also be understood that this document is intended to be a companion to Guidance Document No. 88: Guidance for Flood Risk Analysis and Mapping: Determination of Wave Characteristics, which addresses the physical processes in offshore zones and shoaling zones. These two documents, as well as other FEMA guidance and technical literature, should be used to guide wave runup and overtopping analyses and mapping for FEMA flood hazard studies. For any terms that are not defined in this document, the user should reference Guidance Document No. 66: Guidance for Flood Risk Analysis and Mapping: Coastal Notations, Acronyms and Glossary of Terms.

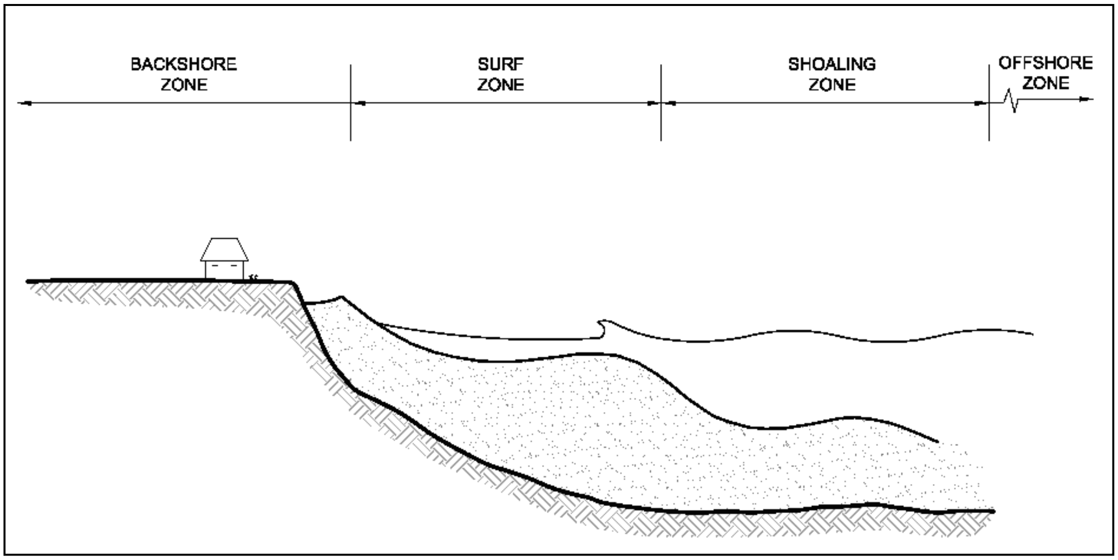


Figure 1: Coastal Zones

2. Wave Runup and Overtopping Theory and Analytical Methods

2.1. Wave Runup

Wave runup is the uprush of water above the stillwater level caused by wave action on a beach or shore barrier. In this document, a shore barrier is defined as a feature (i.e., dune, bluff, revetment, seawall, etc.) along the shoreline upon which waves interact. Runup at a beach or shore barrier can produce flood hazards beyond those from stillwater inundation and incident waves (Figure 2). The runup water wedge or bore generally thins and slows during its excursion up the beach or shore barrier as residual forward momentum in the wave motion near the shore is fully dissipated. When waves break against a steep shore barrier, runup occurs as a jet or spray of water in a near-vertical direction rather than a wedge or bore as previously described. The runup height (R) is defined as the vertical height above the stillwater elevation (SWEL) attained by the extremity of the uprushing water.

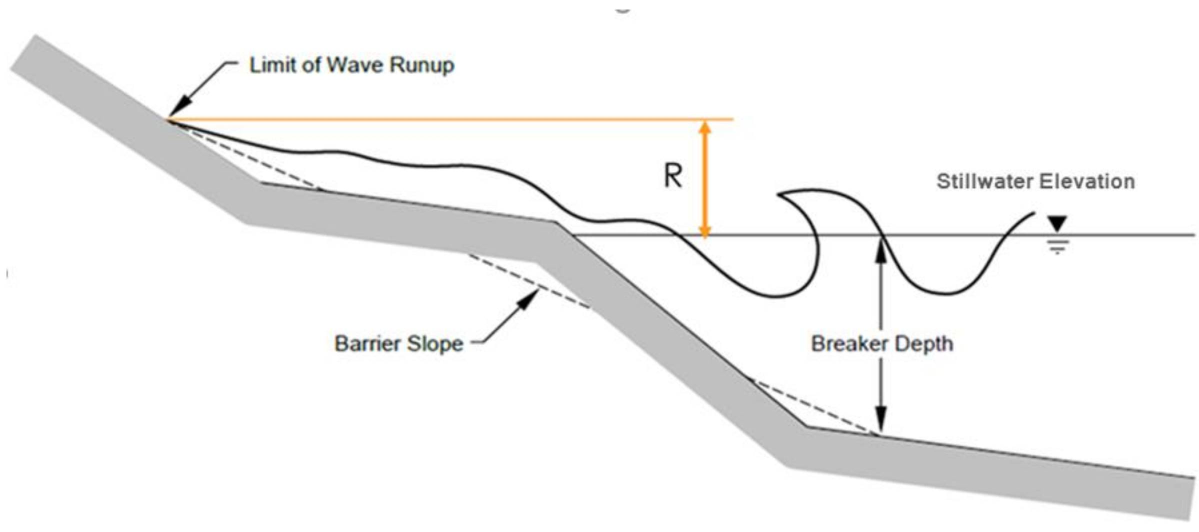


Figure 2: Wave Runup Schematic

Runup heights are dependent on incoming wave characteristics, specifically, wave height, period, and direction, as well as the physical properties of the surf zone and the shore barrier upon which these waves act. If the shore barrier is an engineered structure, runup is influenced by bathymetry seaward of the structure, structure geometry, porosity/roughness, toe elevations, and core permeability. If the shore barrier is a natural feature such as a dune or bluff, runup is similarly influenced by the seaward bathymetry, geometry, surface roughness, and the toe elevation of the feature. Runup can be influenced by erosion or accretion across the nearshore bathymetry, erosion of natural features (dunes and erodible bluffs), and structure failure. Since runup is sensitive to many physical characteristics as well as variations in shore-approaching wave characteristics and erosion, it can vary considerably along the shore. Further, runup can vary locally along a short distance of coastline in response to variations in shore barrier type or characteristics along these distances.

More generally, wave runup elevations are determined by the kinematics of wave breaking and how much wave energy is available. Waves that curl and plunge provide less shoreward momentum to drive wave runup than waves that gently break or surge up the shore unbroken. The non-dimensional parameter describing these kinematics is the Iribarren number (ξ), defined as the ratio of the characteristic profile slope to the wave slope:

Equation 1:

$$\xi = \frac{m}{\sqrt{H/L}}$$

in which m is a representative profile slope and is defined as the beach or shore barrier slope, and H and L are the characteristic wave height and length, respectively. The definitions of the characteristic wave height and length depend on the runup methodology applied. The wave energy available to drive wave runup is proportional to the square of the wave height. Recognizing these general characteristics, Hunt (1959) proposed the following empirical formula to estimate the wave runup height (R):

Equation 2:

$$\frac{R}{H} = \xi$$

Using Hunt's general relationship, researchers have fit runup observations from laboratory and field settings with exponential best-fit factors to describe runup on various shorelines. The most general form of empirically derived formula relating breaking kinematics and incoming wave energy has the following form (Holman, 1986):

Equation 3:

$$\frac{R}{H} = \xi^a + b$$

where a and b are derived by fitting this formula to observations. Note that the definition of the slope and characteristic wave parameters (height and period) used may vary among the empirical methods. Empirical runup methods usually determine estimates for irregular wave runup. The most frequently modeled runup heights are R_{mean}, the mean runup elevation, and $R_{2\%}$, the runup exceeded by 2% of the runup values attained by a group of irregular waves. The current National Flood Insurance Program (NFIP) policy, as of 2017, defines the wave runup height as $R_{2\%}$, which is used to determine BFEs. For levees, the maximum wave runup (where R_{max} = R_{mean}*2.87 or R_{max} = $R_{2\%}$*1.29) is required and can be referenced in the FEMA Guidance Document No. 95: Guidance for Flood Risk Analysis and Mapping: Levees. Unless otherwise indicated, the runup referred to hereafter is $R_{2\%}$.

The Mapping Partner should ensure that the mapped wave runup elevations are the $R_{2\%}$ values. If the modeled wave runup elevations are not the $R_{2\%}$ values, conversions to this value can be made assuming wave runup elevations follow a Rayleigh distribution (Walton, 1992). Most often, wave runup methods and models return either the $R_{2\%}$ or R_{mean}. Following the Rayleigh distribution, $R_{2\%}$ is equal to 2.2*R_{mean}. Additionally, some models will provide Significant Runup, such as the Automated Coastal Engineering System (ACES) Wave Runup and Overtopping on Impermeable Structures module. Assuming a Rayleigh distribution, R_{mean} = 0.626*Significant Runup, and therefore, Significant Runup can be converted using $R_{2\%}$ = Significant Runup*1.38.

Summaries of different methods for calculating wave runup have been compiled in various peer reviewed publications (e.g., Kobayashi, 1999; USACE, 2011 [*Coastal Engineering Manual* (CEM)]; and EurOtop, 2018). In addition, Melby (2012) provides a review of runup methods that is focused on flood hazard studies. As noted by Kobayashi (1999), wave runup on coastal structures has been studied mostly by engineers using hydraulic physical models, whereas wave runup on beaches has been studied mostly by oceanographers using field measurements. There are several computer programs available to compute runup. The most commonly applied programs used in FISs for runup calculations are ACES (USACE, 1992), Runup 2.0 (FEMA, 1981; 1991), and CSHORE (Kobayashi et al., 2013 and Johnson et al., 2013). ACES and Runup 2.0, are based on empirical runup methods, while CSHORE is a phase-averaged cross-shore numerical model that utilizes a combination of empirical derivations; Section 1.2 of Appendix A contains recommendations on the configuration and application of CSHORE for FEMA projects. Table 1 provides a general guide to the selection of wave runup methods and models for environments of interest. These methods and models have been employed in recent FISs. Other methods and models for computing wave runup exist. The state-of-the-art for computing wave runup is expected to advance as more studies are published and more computer models are developed.

As shown in Section 3, many environmental parameters are considered in runup method selection. It is the responsibility of the Mapping Partner to ensure that a defensible method is employed for computing runup at each site. Further, the Mapping Partner should consider models and methods that are generally accepted in coastal modeling practice at the time of study in addition to those listed in Table 1.

Table 1: General Guide for Runup Method Selection

		Criteria		
	Runup Methods	**Slope**	**Iribarren Number**	**Shoreline Type**
Empirical Equations	**TAW**[1]	1:8 to 1:1	0.5 – 8-10	Rock-Armored Structures with Narrow Surf Zones
	Stockdon	Up to 1:1	—	Sandy Beaches without Dune
	Van Gent	Up to 1:1	1 – 10	Impermeable Structures Located in the Surf Zone
	SPM[2]	∞	N/A	Vertical Walls (Sea Walls and Bulkheads
Computer Models	**ACES**	Up to 1:1	Up to 2 for Beaches; 0.5 – 10 for Shore barriers	Beaches, Riprap, and Impermeable Structures
	Runup 2.0	Up to 1:8	—	Multiple Types
	CSHORE	N/A – steep, near-vertical slopes not applicable	Greater than 0.3	Multiple Types

[1] The Technical Advisory Committee for Water Retaining Structures method.

[2] Shore Protection Manual (USACE, 1984).

Equations and defined variables corresponding to the empirical runup equations listed in Table 1 are included for reference in Section 2.1 of Appendix A. The sourced references associated with each equation should be evaluated to fully understand the underlying assumptions for each runup methodology. The Mapping Partner should judiciously investigate the definition of wave height (significant wave height versus mean wave height or other) and wave period (peak period versus mean period or other) required for empirical runup methods or models. Supplemental information for the Shore Protection Manual (SPM), CSHORE, and TAW are also provided in Appendix A. The SPM figure used to determine runup wave heights on impermeable vertical structures is provided in Section 2.4 of Appendix A. The SPM figure in this guidance should be used, as there may be conflicting figures in superseded guidance. Additionally, previous guidance was developed for incident wave height and slope for the use of the Technical Advisory Committee for Water Retaining Structures method (TAW) to determine the runup wave height. The additional TAW guidance is included in Section 2 of Appendix A.

2.2. Wave Overtopping

Wave overtopping occurs when the shore barrier's crest elevation is lower than the wave runup elevation (Figure 3). Overtopping is quantified as the volumetric flow rate of water over shore barrier crests per unit length along the shore barrier. There are three physical forms of overtopping; green water, splash, and spray:

- Green water overtopping occurs when waves break onto or over the shore barrier and the overtopping volume is relatively continuous.
- Splash overtopping occurs when waves break seaward of the shore barrier face, or where the shore barrier is high in relation to the wave height, and overtopping is a concentrated stream of water droplets. Splash overtopping can be carried over the shore barrier under its own momentum or may be driven by onshore-directed wind.
- Spray overtopping is generated by the action of wind on the wave crests immediately offshore of the shore barrier. Spray overtopping may cause damage to salt-sensitive vegetation, crops, or building facades. Without the influence of a strong onshore wind, the volume of overtopping spray contributes is negligible relative to the other two types.

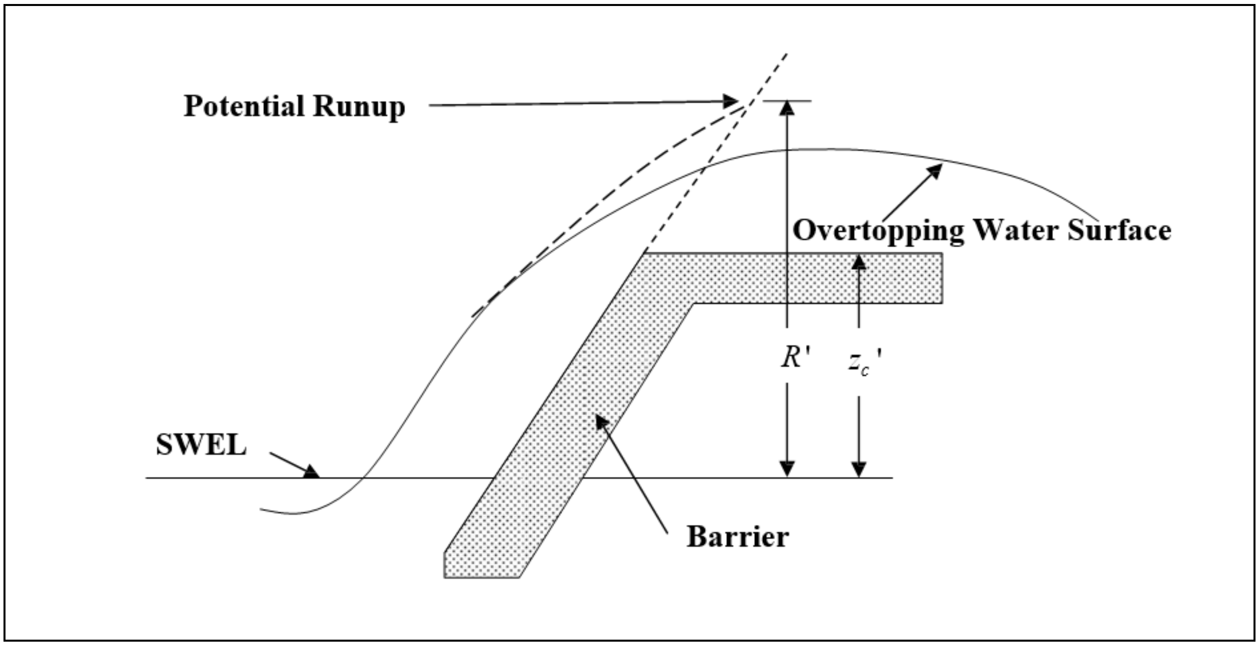


Figure 3: Definition Sketch for Wave Overtopping

Wave-induced overtopping is often a driving mechanism that creates coastal floodplains shoreward of steep shore barriers. Mapping flood hazard zones due to green water and splash overtopping requires an estimate of the water velocity and overtopping rate propelled over the crest as well as the envelope of the water surface, which is defined by the water depth landward of the crest.

The overtopping rate and water surface envelope will depend on the incident water level and wave characteristics, as well as the physical properties of the shore barrier. Other considerations are the type of wave breaking observed and the waves' interaction with the shore barrier. Overtopping rates (q) are computed using empirical equations. The principal formula used for overtopping is determined by an exponential function with the dimensionless overtopping discharge and the relative crest freeboard as follows:

Equation 4:

$$\frac{q}{\sqrt{gH^3}} = aexp(-bZ_c/H)$$

where q is mean overtopping rate, g is the gravity acceleration, H is a characteristic wave height, Z_c is the freeboard measured as the height from the SWEL to the shore barrier crest elevation, and empirically derived coefficients *a* and *b* depend on different coastal structures (i.e., dikes, rubble mounds, revetments, seawalls, etc.) with distinct features (i.e., parapets, toe protection, berms, etc.). A complete description of methods for overtopping calculations can be found in the EurOtop Manual (EurOtop, 2018). The manual also provides methods for computing runup in special scenarios, such as shallow and very shallow foreshores and very steep to vertical walls. The EurOtop Manual equations include contributions from 'green water' and 'splash' overtopping but neglect spray overtopping. Equations and defined variables corresponding to overtopping equations listed are included for reference in Section 2.2 of Appendix A. The sourced references associated with each equation should be evaluated to fully understand the underlying assumptions for each wave overtopping methodology.

3. Runup and Overtopping Modeling

This section discusses various factors to be considered when determining runup and overtopping for different physical properties and wave characteristics. The discussion covers the most commonly used methods in effective FISs to date (Table 1). However, more advanced modeling approaches may exist that eliminate the need to conduct one-dimensional (1-D) wave analyses. If alternative models are applied, the Mapping Partner must provide documentation that the modeling approach meets or exceeds FEMA's current standards for flood risk analysis and mapping, which are documented in the NFIP's regulations 44 CFR 65.6(a)(6). This document provides guidance on modeling overtopping and runup using 1-D transect analysis because this is the approach taken in all FISs to date and is most accessible to Mapping Partners.

3.1. Statistical Consideration

As stated in Guidance Document No. 2: Guidance for Flood Risk Analysis and Mapping: Coastal General Study Considerations, the primary goal of an FIS is to determine the frequency of recurrence of flood elevations throughout the study area, and to establish the 10%-, 4%, 2%, 1%, and 0.2% annual-chance flood elevations in a study area. Prior to performing a runup and overtopping analysis, the Mapping Partner should determine whether an event selection method or response-based approach will be used, each of which require specific statistical analyses to determine the flood elevations.

As described in Coastal General Study Considerations Guidance, the event selection method involves analysis of a single event (or a small number of events), which requires a statistical analysis of the forcing conditions for all storm events prior to runup analysis. Essentially, the event selection method establishes which forcing combinations (wave height, wave period, SWELs) and, possibly, eroded state of the nearshore profile, will produce the 1% annual-chance runup. This forcing combination is used to compute a single runup height equal to the 1% annual-chance runup. On the other hand, the response-based method requires statistical analysis directly on the computed runup for the duration of modeled storm events. Statistical methods used on the forcing conditions for the event selection method or directly on the runup elevations for the response-based method are described in Guidance Document No. 76: Guidance for Flood Risk Analysis and Mapping: Coastal Flood Frequency and Extreme Value Analysis.

As described in Section 2, incident wave characteristics (height and period) and barrier characteristics (e.g., slope, porosity, etc.) determine wave runup and overtopping at a specific location. Section 2 also notes that barrier characteristics may be influenced by erosion or accretion, or by structure failure. A response-based approach can dynamically account for the interconnected physical characteristics of the system during wave runup and overtopping analysis. A typical response-based method utilizes time series of wave heights, wave periods, wave directions, and water levels for entire historical (hindcasts) or probabilistic storm events. Within this method, empirical methods or runup and overtopping models may be used to produce a corresponding time series of runup and overtopping for each storm event simulated.

In certain cases, storm results must be removed from the statistical analysis through an optimization process. For historic storm events sampled using the Peak Over Threshold (POT) approach, storm events are typically selected from different locations throughout the study area. Because the study area often covers very large regions, storm events selected at one location might not meet the selection criteria at other locations. Therefore, resulting runup elevations from all storm events at a particular location might not follow the extreme distribution well. To remove the bias introduced by results from storms that do not meet the selection criteria, one recommended approach is the quantile-quantile (Q-Q) optimization as described in Melby et al. (2012a, b). In this approach, the 25% and 75% quantile-quantile line from the samples and fitted distribution with reduced samples are compared by continuously ignoring the lowest values. The one aligning the best with the 1:1 line determines the best fit.

3.2. Transect Layout

As mentioned in previous sections, wave runup and overtopping are sensitive to physical properties of beaches or shore barriers and incoming wave characteristics. Cross-shore transects are generally placed along a reach to capture specific shoreline characteristics. In order to accurately capture variations in runup and overtopping along the coastline, transect placement should be refined enough to capture notable variations in the following factors:

- Wave characteristics [wave height (H), wave period (T), wave direction (θ)]
- Shoreline settings
- Beach or shore barrier slope
- Beach or shore barrier surface roughness
- Shore barrier height and width

The Mapping Partner is encouraged to document the physical parameters and shoreline changes during the transect placement process in Intermediate Data Submittal (IDS) 3 because this information can aid the runup mapping process (Section 4).

3.2.1. WAVE CHARACTERISTICS

The wave characteristics influence wave kinematics and the computed runup height (Section 2). During transect placement, the Mapping Partner should capture regions where the wave height, period, and direction are anticipated to change as a result of changes in the shoreline orientation as well as the presence of coastal structures or natural offshore obstructions.

3.2.2. SHORELINE SETTINGS

The shoreline settings refer to the localized geomorphic characteristics of the surf and backshore zones, including the site-specific geology, profile shape, material composition, and profile erodibility. Transects should be placed to capture distinct runup elevations and overtopping rates owing to changes in the shoreline settings. The following shoreline settings may require distinct methods for computing wave runup heights and/or result in different runup heights and overtopping rates:

- Sandy beach, with or without dune
- Sandy beach backed by coastal structures
- Cobble, gravel, shingle beach or mixed grain size beach
- Coastal bluffs and cliffs
- Vertical coastal structures (bulkheads, sea walls, etc.)
- Sloped coastal structures (revetments, dikes, levees, etc.)

3.2.3. EXAMPLES OF TYPICAL COASTAL SETTINGS

The following offers examples of typical coastal settings where runup occurs (Figure 4). The Mapping Partner should explore the shoreline types and consider the location of runup as well as the physical processes that the shoreline reach is subject to prior to assigning the runup method. Common features that can impact the runup profile, such as berms, bars, and dunes, will need to be addressed during the erosion analyses. Additional considerations concerning winter vs. summer profiles, identification of coastal structures, and transect placement should be addressed during the initial phase of the study. Identifying factors that will influence the runup and overtopping results, like profile slope and barrier height/width, is important to ensure a representative analysis is conducted and will result in mapping that can be interpolated laterally along the representative shoreline section.

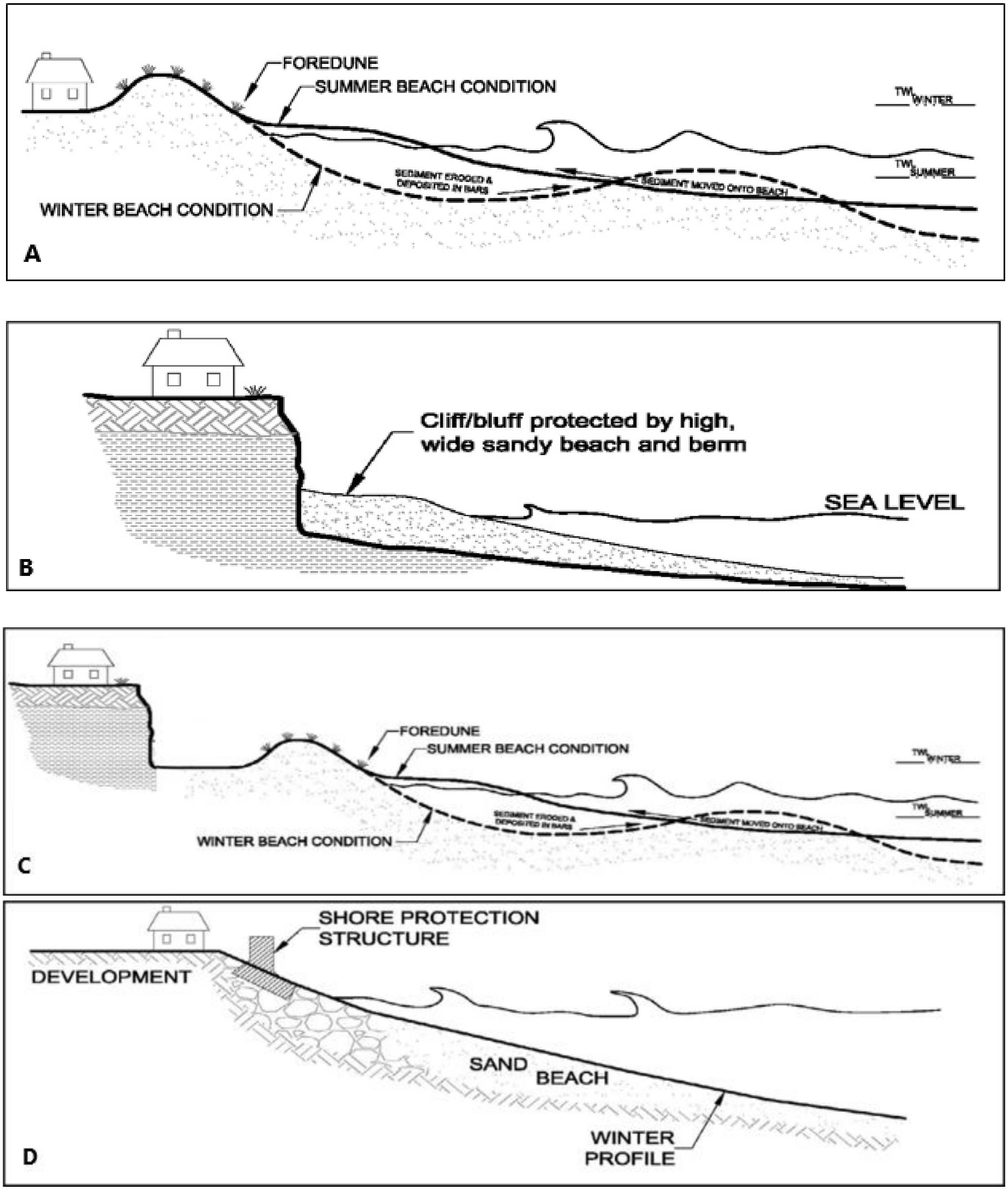


Figure 4 A-D: Typical Coastal Setting Profiles

Sandy Beaches with No Shore Protection Structures

Profiles for sandy beaches backed by a low berm or high dune (Figure 4A) are directly related to the wave energy present. Runup in this environment can occur on either the beach or eroded dunes, depending on the eroded profile and water level elevations relative to grade. Shorelines with narrow to nonexistent beaches backed by high, steep, erodible coastal bluffs and cliffs (Figure 4B) are common, and runup behavior will change dramatically depending on whether the water levels and runup extents are limited to the sandy beach or if water levels inundate the beach and allow waves to interact with the steep bluff. If the SWEL plus wave setup does not reach the bluff, waves will runup along the sandy beach as swash. If the SWEL plus wave setup intersects the bluff, however, waves may energetically break and induce runup against the bluff face and pose potential overtopping hazards inland of the bluff's crest.

A more complex shoreline type is a sandy beach backed by an erodible dune (foredune) followed by a bluff or a series of dunes (Figure 4C). For these complex shorelines, modeling the erosion of the beach (berm) and foredune should be performed in accordance with Guidance Document No. 40: Guidance for Flood Risk Analysis and Mapping: Coastal Erosion. Erosion analysis should determine where storm-induced water levels (SWEL plus wave setup) may reach. If the foredune remains intact after erosion is considered, runup will occur along the foredune. If the foredune is removed, exposing the bluff to wave attack, runup will occur on the face of the bluff, posing potential overtopping hazards near the crest of the bluff.

Sandy Beaches with Shore Protection Structures

Figure 4D illustrates a shore protection structure fronted by a sandy beach. If the SWEL plus wave setup does not reach the structure after erosion of the sandy berm is considered, waves will runup along the sandy beach as swash and may cause runup on the structure. If the SWEL plus wave setup intersects the structure, waves and energetic wave bores may break against the structure, inducing wave runup along the structure face. If the wave runup exceeds the structure crest, wave overtopping will pose potential flood risks to development that lies landside of the structure crest.

Wave energy may induce scour at the toe of and/or hydrodynamic loads on coastal structures, which may cause complete or partial structure failure. Guidance Document No. 42: Guidance for Flood Risk Analysis and Mapping: Coastal Structures provides methods for deriving a partially failed profile of coastal structures based on scour. The overtopping hazards in a partially failed state may produce more hazardous flooding inland of the structure than a scenario in which the structure is fully intact or completely removed. If it is determined that the coastal structure may fail or partially fail due to toe scour, runup, and overtopping, the structure should be modeled on both scenarios to determine the most hazardous conditions.

3.3. Forcing Conditions for 1-D Analysis

Conducting 1-D analysis of surf zone and backshore hydrodynamics depends on the SWEL and wave characteristics. These forcing conditions can be obtained from 1-D or two-dimensional (2-D) models.

For guidance regarding the generation and propagation of waves from offshore water to the shoreline using 2-D models and guidance on obtaining SWEL and wave conditions if 2-D models are unavailable, please reference the companion document, Guidance Document 88: Guidance for Flood Risk Analysis and Mapping: Determination of Wave Characteristics.

The underlying assumption for the 1-D transect analysis is longshore uniformity, so there is a reliance on the 2-D model to capture the complex coastal wave processes, such as refraction and shoaling, which are not uniform alongshore. Generally, it is expected that 2-D SWEL and wave models adequately resolve waves from offshore through the shoaling zone (Figure 1) but may not resolve wave characteristics adequately within the surf zone. In such a case, the wave characteristics and SWEL should be extracted from 2-D models for 1-D analyses outside of the surf zone, seaward of where wave breaking and other shallow water wave dissipative processes occur.

The most straightforward approach to establish the surf zone limit for extracting wave and water level parameters from 2-D models is to define this location based on a representative depth contour or another fixed location based on model results and/or engineering judgment. Other approaches use the 2-D model results to determine the location of the surf zone limit based on the ratio between wave height and water depth or the fraction of breaking waves.

3.4. Runup Model Selection and Parameterization

When selecting runup models or methods, the Mapping Partner should consider the limitations and applicability of the selected models. The applicability of commonly used runup methods based on slope, Iribarren number, and shoreline type is also included in Table 1 (Section 2). Figure 5 below provides additional guidance for determining a runup method for an individual transect. In order to apply the flow chart in Figure 5, the information in subsequent sections should be considered. The Mapping Partner should ensure that the selected runup method or model produces results that are defensible.

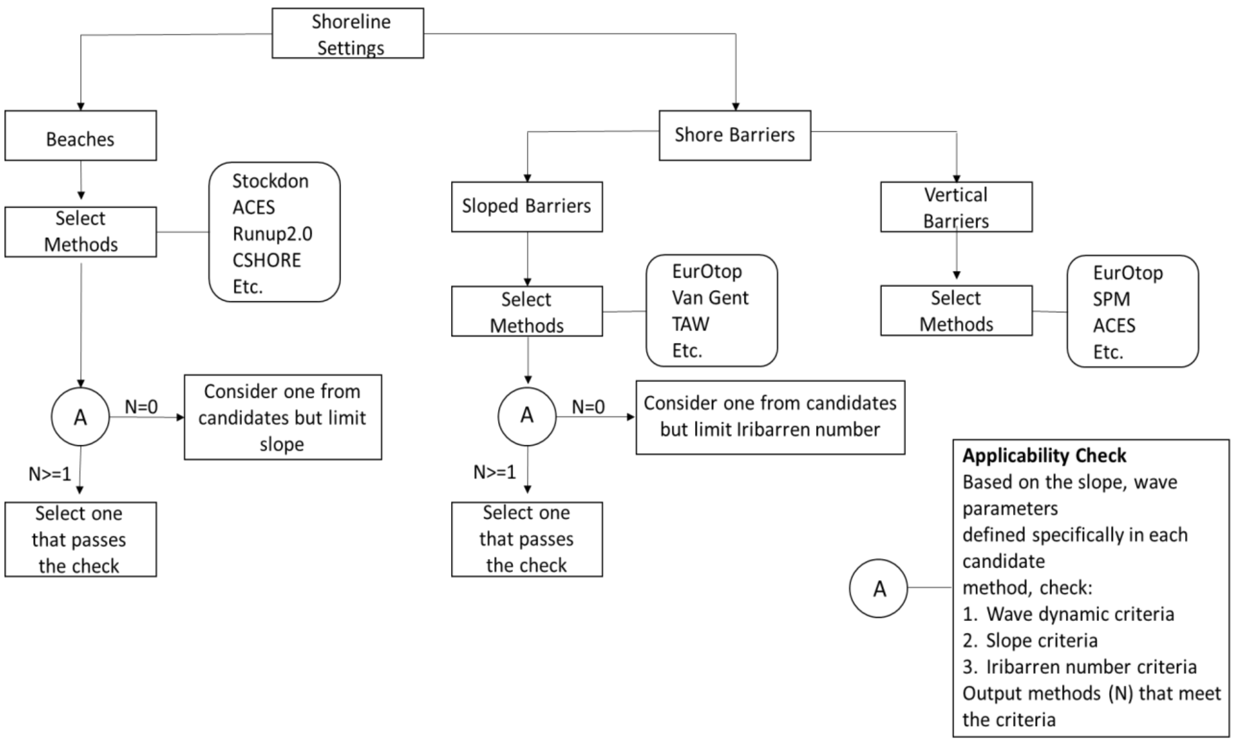


Figure 5: Decision Logic for Runup Method Selection

3.5. Runup Behavior

The focus of this section is the physical processes and empirical formulas applicable when runup occurs on beaches or shore barriers. The Mapping Partner should consult the original documentation associated with the model or equation selected for the runup calculation to ensure its applicability and proper use in calculating runup. The guidelines herein are intended to help the Mapping Partner apply common runup methods.

Identifying the runup location along the transect profile is crucial for correct method selection and model parameterization. Wave runup occurs at approximately the intersection of the SWEL and shore barrier profile. If the water levels resulting from the SWEL plus wave setup are not intercepted by a beach or steep barrier and, instead, inundate portions of the backshore zone, wave runup is unlikely to occur. If a response-based method is applied, this intersection will vary during a storm and among different storms. Therefore, it may require multiple runup methods as the Iribarren number varies or the runup interacts with different features along the profile. When applying the event selection method, the intersection with the shore barrier is determined by the defined SWEL, and the runup method should correspond to the profile properties near that intersection.

A transect profile may change significantly due to erosion or structure failure. Adjustments to the transect profile slope may change the feature that governs runup physics. The empirical runup method selected should correspond to the runup feature identified from the adjusted shoreline profile. Mechanisms for determining a shoreline's eroded profile can be determined using Coastal

Erosion Guidance. Details for structure failure analysis and the inclusion of the failed profile in runup analyses are included in Coastal Structures Guidance.

3.5.1. Runup INPUT PARAMETERS

Runup Slope

Runup calculations are sensitive to the characteristic profile slope, with runup elevations generally increasing along steeper shorelines, since larger slopes increase the Iribarren number (Equation 1). Profiles often do not present a single, uniform slope. The Mapping Partner should follow the definition of the characteristic profile slope provided in the original reference for the selected empirical equation. For example, the beach slope in the Stockdon method is defined as the average slope over a region $\pm 2\sigma$ around η, where σ is the standard deviation of the continuous water level record, $\eta(t)$. Some researchers suggest using the surf zone slope, defined as the slope between the shoreline (the cross-shore position of η) and the cross-shore location of wave breaking. If an empirical method is employed and the slope is defined differently from how it was defined during the derivation of the empirical formula, justification for this alternative determination of slope must be provided in the study documentation.

Erosion-Induced Profile Changes

Episodic erosion may require consideration prior to calculating the runup slope. During storm events, partially buried structures may be exposed, and beach and dune erosion may occur. Modifications to the transect profile slope due to erosion can change runup behavior, and consequently it is important to account for and incorporate the expected erosion when performing wave runup analysis. Changes in erodible shore barriers may also impact the inland extent of the coastal floodplain. Mapping Partners should consider toe scour and its influence on water depth at the toe as described in Coastal Structures Guidance. Examples of coastal settings that can be impacted by erosion are presented in Section 3.2.3. Several methods and models exist to calculate the expected shoreline erosion, and additional details on incorporating erosion are included Coastal Erosion Guidance.

Wave Height, Wave Period, and Wave Transformation

In nature, waves are irregular such that the individual waves that approach a coastline are seldom of constant characteristics. These irregular waves are typically parameterized by a single characteristic height, period, and direction. The values of these representative characteristics vary from offshore and across the shoaling zone.

Wave transformation may be required to determine the appropriate wave characteristics for runup analyses. Empirical methods that require the wave conditions at the breaking location or in deep, offshore water may require transformation of wave heights from the wave conditions at the surf zone limit (Section 3.2). For example, wave conditions in the shoaling zone or near the surf zone limit may need to be deshoaled and unrefracted to yield equivalent deep water wave heights for some runup methods. For steep or hardened shorelines, some empirical runup methods are governed by the wave energy at or near the toe of the shore barrier. These waves can be readily transformed to the toe location using linear wave theory or a transect-based wave transformation model. Process-based computer models such as CSHORE can also be used to transform waves across the surf zone to the shore barrier toe. Often, the wave heights at the toe of the shore barrier are limited by the depth at the toe. The Mapping Partner should judiciously investigate the definition of wave height (significant wave height versus mean wave height or other) and wave period (peak period versus mean period or other) required for empirical runup methods or models and ensure that the proper form of the wave parameters are applied and that the location of input wave conditions are appropriate based on the empirical method or model. Example references on how to perform the wave transformation calculations are the CEM (USACE, 2011) and *Water Wave Mechanics for Engineers and Scientists* (Dean and Dalrymple, 1991).

Wave Setup

The total water level (TWL) includes storm surge, wave setup (both static and dynamic where applicable), and wave runup. To avoid underestimating (or overestimating) the TWL by missing (or double counting) the wave setup component, the Mapping Partner should verify whether wave setup is included in the runup methodology. Details on estimating wave setup are included in Guidance Document No. 44: Guidance for Flood Risk Analysis and Mapping: Coastal Wave Setup. Guidance for inclusion of wave setup in wave runup methods based on both technical documentation and common practice is included in Section 2.3 of Appendix A. In some flood studies where the SWEL surface implicitly includes wave setup (such as 2-D ADCIRC+SWAN-based storm surge studies), the recommendations in Section 2.3 of Appendix A cannot be applied. Wave setup cannot be efficiently removed from the SWEL + Setup 1% annual-chance surface, and consequently, common practice applies all runup methods to this surface assuming a negligible impact to the runup elevation. This practice is essentially assuming that wave setup is not implicit in the runup methods (i.e., wave setup is not included in the calculated wave runup value).

Empirical Equation Applicability

Due to large variations in the physical and dynamic characteristics (in terms of the Iribarren number) of shore barriers, it is difficult to find one empirical runup equation that applies to all shorelines. For example, a beach runup method would not likely be applicable for a steep, hardened shoreline. Coefficients in the empirical runup equations are typically calibrated for a specific physical setting and wave environment. For example, the TAW method is valid in the range of $0.5 < \xi_{om} < 8\text{-}10$ in terms of the Iribarren number, and valid for structure slopes in the range of 1:8 to 1:1. Some general criteria for approved runup methods are referenced in Table 1, and Figure 5 identifies applicable runup methods based on shoreline settings. The Mapping Partner should choose empirical equations developed for barriers with physical properties and wave characteristics similar to the area of interest. It is recommended that the Mapping Partner consult source references for each method to ensure that appropriate parameter ranges are utilized. Additional resources are provided in Appendix A.

Runup Reduction Factors

Most empirical equations for runup on shore barriers require reduction factors to account for roughness, wave directionality, and the presence of a berm. For details, refer to the discussion in Section 2 and the source reference for each method chosen for the study. Additional resources are provided in Appendix A. Mapping Partners should consider applying reduction factors as defined in the original documentation for each empirical method. If runup models are employed Mapping Partners must refer to the user's manuals for those models to check for reduction factors.

Runup from Smaller Waves

Some runup methods are questionable where the toe of a structure, or a naturally steep profile such as a rocky bluff, is at an elevation close to the input water levels. If a runup calculation recognizes a shallow-depth toe location and the runup method relies on an input depth-limited wave condition, a subjective selection of the toe location could limit the local wave height and underestimate the calculated runup result.. In these cases, the runup profile may be subject to larger waves breaking in proximity to the shoreline, and a small depth-limited wave may not accurately represent the level of risk that could come from the larger nearby wave conditions. To resolve this issue, it may be necessary to calculate wave runup at several locations across the surf zone. 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 following 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 appropriate methods described in Table 1 for runup on a barrier. Oftentimes this requires iteration for this location to determine the average slope based on the runup elevation and the profile elevation selected as the toe location. Iterate until the runup converges for this location.

2. Repeat the runup calculations at different cross-shore locations until a maximum runup is determined.

This recommended procedure considers a range of (smaller) wave heights inside the surf zone in runup calculations and prevents the influence of subjective selection of toe elevations used in runup calculations. A sensitivity test using alternative toe locations to begin the runup computation helps to ensure that the more conservative potential runup risk is accounted for in modeling. For example, in some countywide studies in steep shoreline environments, multiple TAW runup calculations are modeled for one modeling transect, assuming a toe elevation at 1-foot depth increments along the profile until the maximum runup condition is determined.

3.6. Overtopping Analysis

Overtopping analysis should be performed on any shore barrier with a crest elevation lower than the runup elevation. As discussed in Section 2.2, the EurOtop Manual (EurOtop, 2018) provides some of the best guidance for overtopping calculations. In superseded guidance (FEMA, 2007), an additional empirically based overtopping equation from De Waal and Van Der Meer is available. This equation is provided in Appendix A, Section 2.2, as a reasonable method for calculating overtopping. The Mapping Partner should refer to the latest version of the EurOtop Manual for more detailed overtopping analyses with variations to the equations and inputs based on site conditions. This guidance offers key considerations influencing overtopping.

3.6.1. OVERTOPPING BEHAVIOR

Most generally, wave overtopping can be in the form of 'green water,' in which complete sheets of water flow off the shore barrier crest, or 'white water,' which is a spray of water entrained with air that is carried over the crest by its own momentum and onshore winds. 'Green water' conditions occur on a sloping shore barrier (rubble mounds, dikes, levees, revetments, etc.) where the wave runup acts as a wedge or bore of water propagating up the face of the shore barrier. The remaining energy in the wedge or bore as it reaches the structure crest provides energy for that wedge or bore to continue inland of the shore barrier. 'Green water' conditions also occur on vertical or near-vertical shore barriers when the water depths at the toe and foreshore portion of the profile are large enough that waves are unbroken as they interact with the barrier and the unbroken wave crest elevation exceeds the shore barrier crest. If waves plunge near the toe of a vertical or near-vertical shore barrier, the collision of the plunging wave against the face of the shore barrier will drive a near-vertical jet of white water up and over the crest of the shore barrier.

3.6.2. DETERMINISTIC VS. PROBABILISTIC OVERTOPPING RATES

The EurOtop Manual offers not only the empirical models for the mean overtopping rate estimation, but the associated model uncertainties as well. The model uncertainty is considered to be the accuracy with which a model can describe a physical process or a limited state function. In the EurOtop Manual, model uncertainties are measured by the standard deviation, which is derived from the comparison of the measured data and model predictions. As such, two sets of parameters for all empirical models in the EurOtop Manual are provided corresponding to probabilistic design value and deterministic value. The probabilistic design value describes the mean approach for all underlying data points, while the deterministic design value is given as the mean value plus one standard deviation. Deterministic equations provide conservative overtopping values by taking into account model uncertainty for wave overtopping and should be employed by Mapping Partners when calculating overtopping rates for flood hazard mapping purposes.

3.6.3. OVERTOPPING REDUCTION FACTORS

Most empirical equations for overtopping on shore barriers require reduction factors to account for roughness, wave directionality, and the presence of a berm along shore barriers. For details, refer to the discussion in Section 2 and the source references for the empirical equations chosen for the study. Mapping Partners should consider applying reduction factors as defined in the original documentation for each empirical method. If runup models are employed, Mapping Partners must refer to the user's manuals for those models to check for reduction factors.

3.6.4. OVERTOPPING FLOWS

When overtopping is in the form of 'green water,' bores or sheets of water can flow over terrain inland of the shore barrier crest. Generally speaking, overtopping flows are driven inland by the momentum contained in the overtopping bore and gravity forces. In some FISs to date, the method proposed in Cox and Machemehl (1986), to calculate the inland limit of the overtopping bore was adapted to compute the bore height and velocity profile overland to account for the slope of the inland terrain. Experimental results of overtopping bore depth and velocities on landward slopes of sea dikes are explained in Chapter 5.5.5 of the EurOtop Manual. This chapter also provides an analytical function of the overtopping flow velocities and sheet flow depth on landward slopes.

3.6.5. OVERTOPPING VOLUMES

If a response-based modeling method is employed as described in Section 3.1, time series of mean overtopping rates can be computed over the duration of all modeled storms. These time series can be integrated over time and multiplied by the longshore length of the shore barrier to which the overtopping rates are applied to yield overtopping volumes. When calculating overtopping volumes, the Mapping Partner should also consider the potential volume of water added by concurrent rainfall and/or the volume of water drained through any drainage infrastructure in the local area.

4. Floodplain Mapping

Floodplain mapping is the translation of runup results from the 1-D transect analysis to a 2-D planimetric map or dataset to provide an understanding of flood hazards along the shoreline. Shorelines dominated by runup are generally characterized by high flood elevations and velocities. Therefore, high hazard and high-velocity (VE) zone classifications are typically applied to shorelines susceptible to structural damage, while regions with runup elevations less than 3 feet above the ground are designated with a less hazardous (AE) flood zone. Inland of feature crests, sheet flow (Zone AO) and ponding (Zone AH) overtopping may occur (Section 4.2.2). Additional information on flood zone classifications is available in Guidance Document No. 39: Guidance for Flood Risk Analysis and Mapping: Coastal Floodplain Mapping.

There are many variables along the shoreline contributing to coastal flooding that influence mapping methodologies. Evaluating both the modeled physical characteristics and runup results during the mapping process provides a stronger understanding of the physics occurring along a transect and aids in mapping decisions. The following sections provide guidance on how to evaluate the coastal setting and calculated BFEs for runup and overtopping mapping applications. Coastal Floodplain Mapping Guidance provides further information on mapping techniques for runup processes and flood zone designations.

4.1. Lateral Zone Extent

Except for coastlines where overland wave propagation dominates coastal flooding, runup behavior may abruptly change in response to modifications in the physical setting. Transects are generally placed along a reach to capture specific shoreline characteristics (as described in Section 3.2), and the Mapping Partner should ensure mapped transect results apply to shoreline reaches with comparable characteristics. Variability in the calculated runup may result in the appearance of 'jumps' in BFEs along the shoreline on a FIRM, where adjacent VE zones have BFE differences greater than 1 foot. SFHA boundaries on the FIRM dividing Zone VE or AE with different BFEs are placed between transects at shoreline locations where there are anticipated transitions in runup behavior due to variations in wave characteristics, shoreline settings, slope, and surface roughness. Each of these considerations as well as shore barrier heights and widths are discussed below.

4.1.1. WAVE CHARACTERISTICS

Based on aerial imagery and an understanding of wave mechanics, the Mapping Partner should inspect the coastal environment to develop an understanding of the wave behavior within the study area. This includes evaluating features such as coastal structures (i.e., breakwaters and jetties) that influence wave processes (i.e., wave propagation, refraction, and diffraction).

4.1.2. SHORELINE SETTINGS

The shoreline settings are often the most notable distinction between modeled transect locations. Inspection of overhead aerials and oblique shoreline imagery provides key indications of changes in shoreline type, such as whether a shoreline is characterized as a sandy beach, bluff, wetland, or structure. A flood zone boundary (lateral flood zone break) should be placed at locations where the shoreline type changes and results in modifications to the BFE. For long stretches of shoreline with the same shoreline type, the Mapping Partner should evaluate whether notable variations in other factors, such as shoreline slope, erosion, or surface roughness, affect runup behavior, as described below.

4.1.3. RUNUP SLOPE

Runup elevations are significantly influenced by slope; therefore, changes in the terrain slope are associated with transitions in the calculated runup. Variations in the slope of the terrain are often apparent from inspection of the topographic data used during the runup modeling as well as overhead aerial and oblique imagery. Generating contour lines from the topographic dataset is also useful for establishing slope changes. Changes in the elevation contour spacing provide visual cues for slope changes. When the BFE changes in response to runup slope changes, a flood elevation boundary should be placed to define the lateral extent of calculated runup results.

To properly reflect the flood hazard, the Mapping Partner should include the influence of erosion on the runup and inland extent of the mapped floodplain when assessing slope changes. As large waves crash onto erodible shorelines, sediments are moved offshore, and the runup face (i.e., slope) changes. When translating modeling results to produce a flood hazard map, the Mapping Partner should question how erosion influences runup and whether the topographic information used for mapping reflects the eroded transect profile. The Mapping Partner should consider factors such as the inland extent of erosion to determine whether shorelines steepen (i.e., partial dune erosion) or flatten (i.e., dune removal). A flood zone boundary should be placed at locations where the shoreline erosion characteristic and associated BFE are anticipated to change.

4.1.4. SLOPE SURFACE ROUGHNESS

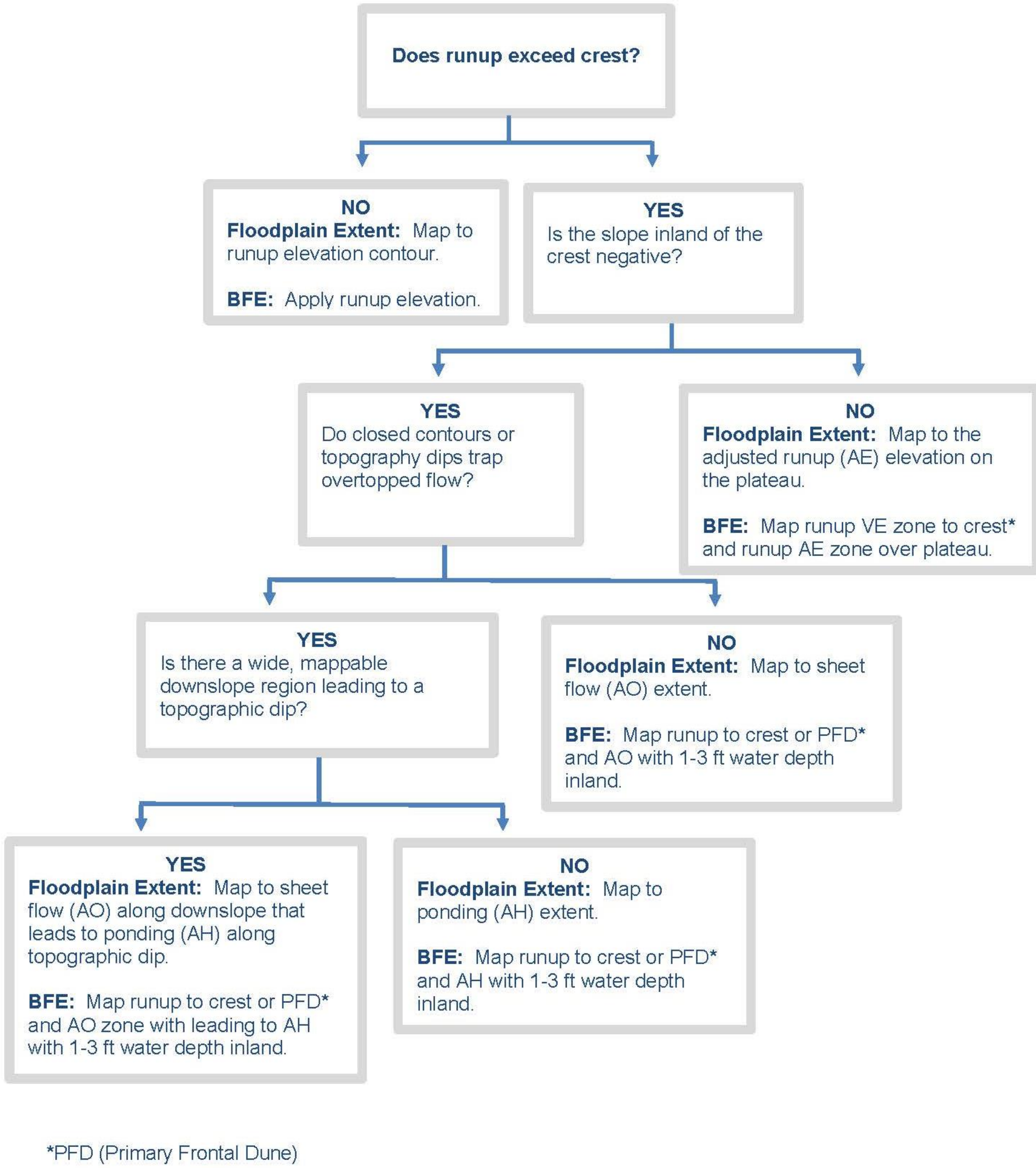
In addition to the steepness of the slope, the surface roughness will also impact the magnitude of the runup elevation. While reviewing runup results, the Mapping Partner should apply modeled transect results along shoreline reaches with similar roughness properties. Abrupt changes in the runup slope roughness associated with changes in the BFE should be captured in the mapping by placing a flood zone boundary at the location of the change.

4.1.5. SHORE BARRIER HEIGHT AND WIDTH

A final consideration while applying mapping results is the variation in barrier height and width along the shoreline. Modifications in the barrier height influence the amount of flow overtopping the feature crest and may influence inland mapping extents. The Mapping Partner should also investigate whether barrier features, such as dunes, taper to narrow widths along the shoreline. A narrow barrier may be removed, while a wide barrier may only be partially eroded. A flood zone boundary should be placed at locations where the runup behavior and BFE are anticipated to change in response to modifications in the barrier geometry.

4.2. Inland Mapping Extent

Larger inland floodplain extents are observed in areas of wave runup when overtopping occurs. Guidance on mapping the inland extent of runup and overtopping is provided in the sections below and demonstrated in Figure 6. Sections 4.2.1 and 4.2.2 include additional information on the types of inland flooding.



*If runup >= 3 feet above the crest, the Zone VE extent is determined by buffering the crest 30 feet inland to account for a splash zone.

Figure 6: FEMA Guidance for the Application of Mapping Flood Hazards Resulting from Wave Runup and Overtopping

4.2.1. RUNUP-DOMINATED INLAND EXTENT

In cases where the runup elevation is less than the crest of the barrier feature, the runup elevation dictates the inland extent and BFE at the shoreline. Along non-erodible shorelines, the contour associated with the runup elevation can be applied to map the inland flooding extent. If erosion is applied to the wave runup profile, care must be taken to estimate the correct inland extent of the runup elevation because the topographic information and contours will not be representative of the erosion. Coastal Floodplain Mapping Guidance provides additional instruction on mapping inland runup floodplain extents given special shoreline protection features, such as mapping to the inland extent of the primary frontal dune. In these instances, the floodplain should not inundate broad floodplain areas. If the runup elevation contour extends inland of the runup feature crest, it is an indication that overtopping should be mapped, and additional analysis may be needed.

A special consideration for runup-dominated inland floodplain extents occurs when runup acts on topographic plateaus, which are features with a flat or mildly positive slope inland of the runup feature crest. The runup elevation along a plateau feature is adjusted to capture runup propagation inland of the barrier crest. French (1982) provides guidance for calculating adjusted runup for plateau overtopping, and this guidance has been applied in many coastal flood hazard studies (Figure 7). The inland limit, X, of the adjusted runup is determined using Figure 8.

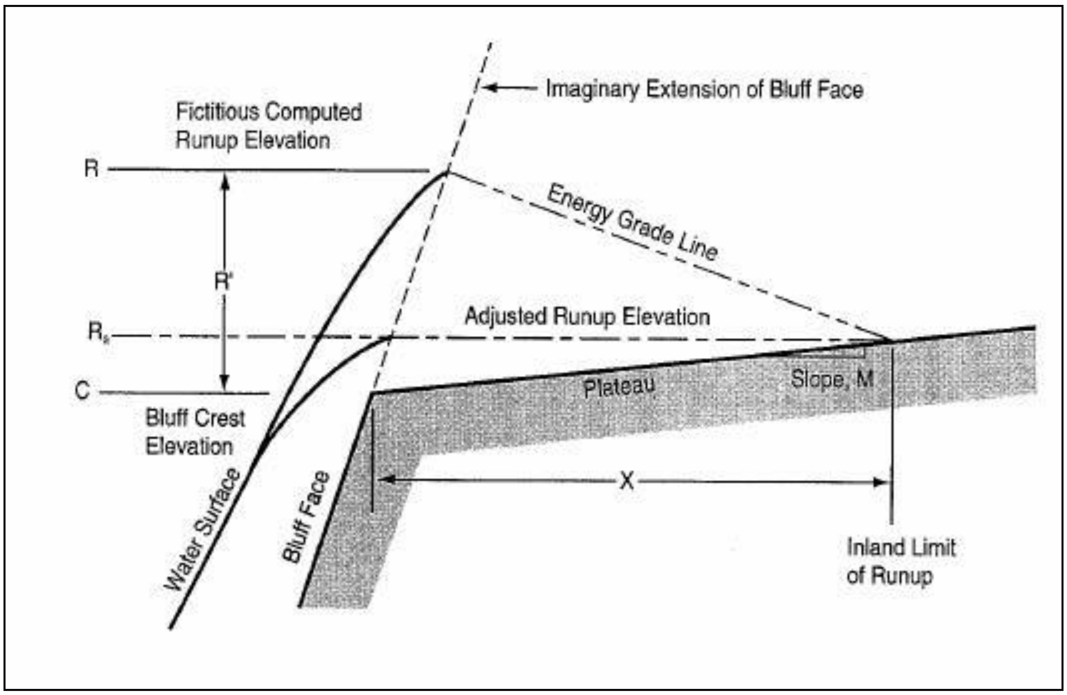


Figure 7: Treatment of Runup on a Plateau Feature

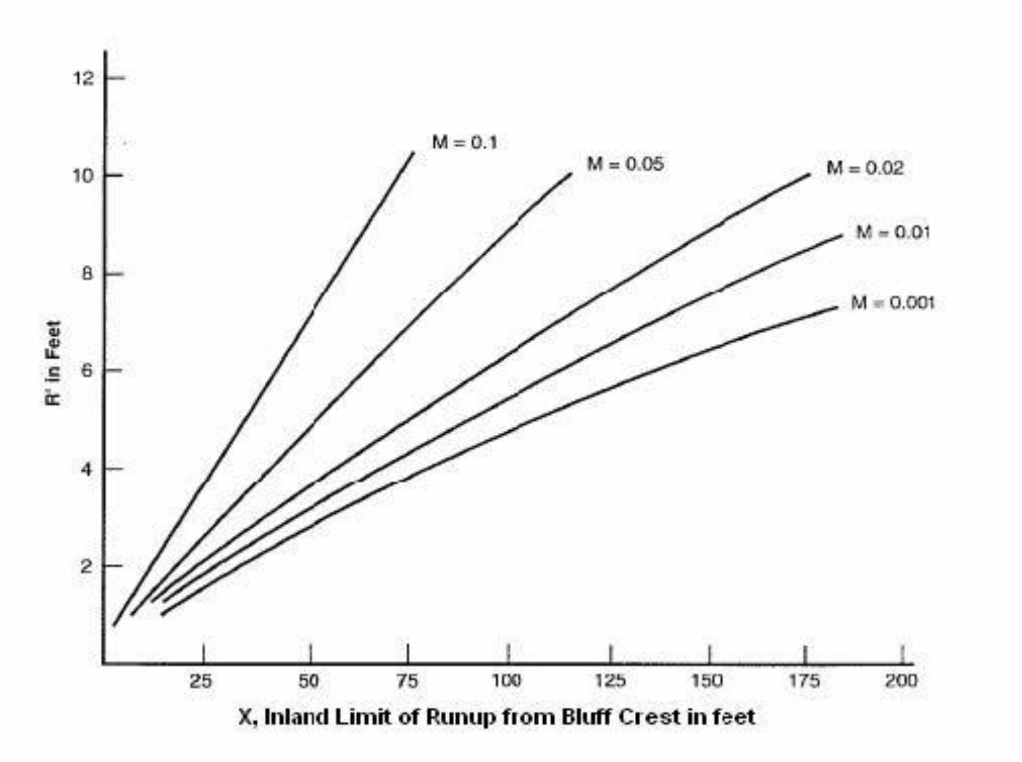


Figure 8: Curves to Compute the Inland Limit of the Adjusted Runup

4.2.2. OVERTOPPING-DOMINATED INLAND EXTENT

Shorelines for which the runup elevation exceeds the crest elevation of the barrier feature are subject to more complex mapping, as the inland extent of the floodplain is controlled by wave overtopping rather than runup. The type of feature and the overtopping flow path are critical input parameters influencing the mapped floodplain.

Sheet Flow and Ponding Overtopping

If the site conditions indicate a negative slope inland of the runup feature, either sheet flow (Zone AO) or ponding (Zone AH) overtopping occurs. Sheet flow is a temporary process whereby gravity guides the overtopping flow downslope. The overtopping flow may either return to the original flooding source through low topography elevations or it may drain into another mapped flood zone. Alternatively, ponding overtopping is observed where local dips in the topography exist and the flow cannot drain into other areas. Ponding regions are easily identified by the presence of closed contours behind a runup feature.

Based on the overtopping flow rate, the Mapping Partner can estimate the depth of flooding (Table 2). The contour applied for mapping sheet flow and ponding overtopping should not exceed the runup feature crest elevation. In some instances, sheet flow can convey water from wave overtopping to ponding areas inland of the barrier. Additional specifications for mapping sheet flow and ponding overtopping are included in Coastal Floodplain Mapping Guidance.

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 topography and drainage landward of the overtopped barrier. An estimated ponding elevation can be determined by comparing the overtopping volume to the available storage landward of the barrier. This elevation should be adjusted by the Mapping Partner depending on drainage features and systems landward of the barrier as well as 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. Table 2 provides guidance for mapping typical coastal overtopping scenarios. This table is not relevant for levee analysis.

In cases of wave overtopping where the potential runup exceeds a barrier crest by 3 feet or more, or in a high-velocity flow zone where the product of depth of flow times the flow velocity squared is greater than or equal to 200 ft^3/sec^2, the Mapping Partner should limit the mapped BFE to an elevation 3 feet above the barrier crest and map a VE splash zone landward of the crest. The Mapping Partner should consider the overtopping depth and velocity as one factor to determine the landward limit of the VE splash zone.

A final consideration in mapping runup is the representation of the overtopping high-velocity hazard. If the computed runup elevation exceeds the runup feature crest by at least 3 feet or if the calculated overtopping rate exceeds 1 cubic feet per second per foot (cfs/ft), the coastal runup BFE is displaced 30 feet inland from the crest to account for the splash zone hazard of high-velocity overtopping.

Table 2: Recommendation for Interpretation of Mean Wave Overtopping

$\overline{Q}$ Order of Magnitude	Flood Hazard Zone Behind Barrier
<0.0001 cfs/ft	Zone X
0.0001–0.01 cfs/ft	Zone AO (1 ft depth)
0.01–0.1 cfs/ft	Zone AO (2 ft depth)
0.1–1.0 cfs/ft	Zone AO (3 ft depth)
>1.0 cfs/ft*	30 ft width^{+} of Zone VE

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

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

Flood hazards inland of shore barrier crests are governed by overtopping hydrodynamics. If a study area includes areas of runup-dominated inland extents (plateau method), a comparison should be made to the inland extents based on sheet flow and overtopping calculations. Since the distinction between these two is that gravity prevents or facilitates inland bore propagations inland, the SFHA extent derived from overtopping calculations and sheet flow considerations should extend farther inland than the SFHA extent derived from the plateau method.

5. Documentation

The Mapping Partner should document the data, methods, and procedures used to perform runup and overtopping analysis to determine the 1% annual-chance flood conditions. Wave runup and overtopping analysis information is supplied to FEMA by Mapping Partners in IDS 4. Documentation should adhere to guidance detailed in Guidance Document No. 25: Guidance for Flood Risk Analysis and Mapping: Coastal Data Capture and Guidance Document No. 3: Guidance for Flood Risk Analysis and Mapping: Coastal Study Documentation and Intermediate Data Submittals. Wave runup and overtopping analysis information is supplied to FEMA in IDS 4. This document provides particular guidance on considerations for mapping runup and overtopping zones. It has been noted that Floodplain mapping guidance is described more generally in Coastal Floodplain Mapping Guidance. This document provides particular guidance on considerations for mapping runup and overtopping zones.

In addition to the required study documentation, the Mapping Partner should provide a technical report and/or supplemental data that provide details on special considerations and approaches taken to ensure the model results are technically defensible. It is best practice that this technical report and/or supplement data be adequate to allow a third party, with sufficient computing capacity and general knowledge, to replicate the results of the FIS. Considering this, the following require documentation (in a technical report or supplemental data) by the Mapping Partner:

- Selection of the model or methods for each modeling transect
- A description of how wave characteristics (wave heights, wave periods, and wave directions) for the selected model or method were applied
- A description of how cross-shore erosion was accounted for in runup and overtopping modeling (if applicable)
- A description of how structure failures were accounted for in runup and overtopping modeling and mapping (if applicable)
- A description of how toe scour was accounted for in runup and overtopping modeling and mapping (if applicable)
- A detailed description of the event-based or response-based method used to model runup and overtopping

- Approaches to determine 1% annual-chance runup elevations and overtopping rates

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Appendix A

1. Supplemental Methodology Updates

1.1. Calculation of Incident Wave Height and Slope for Use with TAW Wave Runup Method

1.1.1. WAVE HEIGHT

The wave height parameter required for use in the TAW equations is the spectral significant wave height, often using H_{mo} at the toe of the structure. 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 at the toe of the structure, unless the Mapping Partner justifies a different value. The reference water level for determining the depth-limited wave height at the toe should be the 2% Dynamic Water Level, DWL2%. The DWL2% is the sum of the stillwater elevation (SWEL) and the static and dynamic wave setup components, if present.

Dynamic wave setup is not calculated on the Atlantic Ocean and Gulf of Mexico coasts where wave and bathymetric characteristics are quite different from those on the Pacific coast. With longer wave periods and a narrower continental shelf, the Pacific wave climate has narrower wave spectra, and consequently, a substantial oscillating dynamic setup component. The dynamic setup is negligible on the Atlantic and Gulf coasts where there are broader wave spectra and shorter wave periods; therefore, the reference water level on the Atlantic and Gulf coasts for calculating the depth-limited wave height at the toe is the SWEL plus the static wave setup component. It should be noted that some methodologies for determining SWEL value implicitly include the wave setup component, and this should be confirmed prior to applying TAW so as to avoid double counting setup in the water level (Appendix A, Section 2.3).

1.1.2. SLOPE

The slope, m, to be used in the Iribarren number calculation for the TAW runup method should be calculated between two points: a lower point, defined by the seaward point, and an upland point, defined by the runup limit. Since the runup limit is initially unknown, the slope is determined using an iterative method. The first estimate of the runup limit should be set at SWEL (without any wave setup components) plus $1.5*H_{mo}$. The seaward point should be set at SWEL minus $1.5*H_{mo}$ unless this point falls below the barrier toe (Figure 9). In these cases, the seaward point should be set to the barrier toe. The TAW slope should not include any portions of the foreshore as shown in Figure 9. If SWEL plus $1.5*H_{mo}$ or the runup limit exceed the barrier crest level or face point, the face point should be selected as the upland point for the slope calculation.

In cases where the slope is uniform between the toe and face points, it may be reasonable to simplify the TAW slope computation with a non-iterative calculation using the toe and face points.

However, the profile should be inspected to verify that the slope is uniform and that this slope is comparable to the slope that would be calculated iteratively.

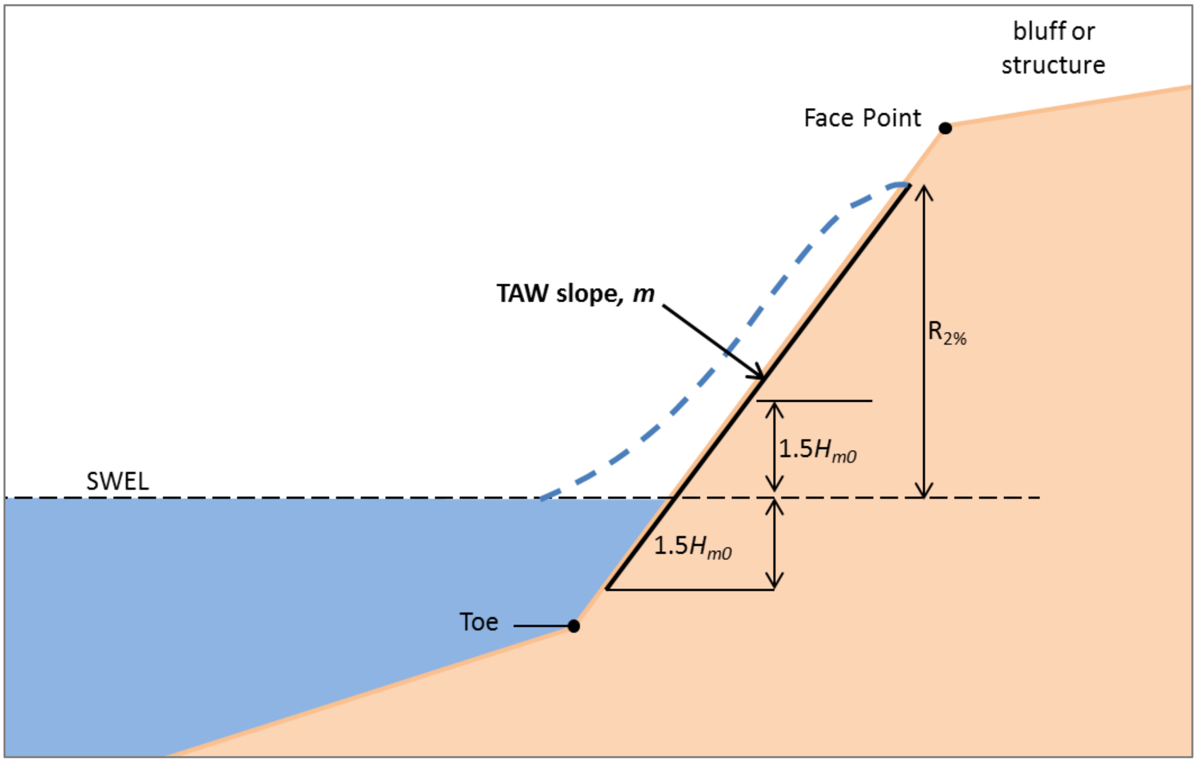

Figure 9: Determination of the TAW Slope for Computing Wave Runup

1.2. Considerations and Recommendations for the Application of CSHORE

1.2.1. CSHORE APPLICATION

CSHORE is a phase-averaged cross-shore numerical model that can be executed quickly and for a wide range of beach settings with the exception of dissipative beaches dominated by infragravity waves (Melby, 2012). Updates to the CSHORE code were implemented in 2013 to impose a limit on runup predictions for very steep and vertical slopes and to stabilize runup predictions for beaches with a large discontinuity in slope (e.g., a gentle foreshore intersecting a steep bluff). The 2013 CSHORE code or a more recent version should be utilized to ensure these code updates are captured. A repository of U.S. Army Corps of Engineers (USACE)-maintained CSHORE versions are available for download from https://sites.google.com/site/cshorecode/.

Table 3 provides a summary of model controls and parameters that should be utilized if model calibration cannot be performed. These reflect recommendations from the CSHORE manual (Kobayashi, 2013) and coordination with USACE developers (Johnson, 2013). As the focus of this document is on runup, parameters controlling erosion have not been included.

Table 3: Summary of Recommended CSHORE Parameters and Controls

Parameter	Description	Value	Notes
DX	Nodal spacing	≤ input	CSHORE nodal spacing should not exceed the input profile resolution.
GAMMA	Breaker ratio	0.7	CSHORE manual recommends 0.5–1.0, with 0.7 typical.
RWH	Runup wire height (meters)	0.03 – 0.1	CSHORE manual recommends 0.1 for prototype beaches. USACE recommends 0.3 to 0.5 for field conditions. RWH smaller than 0.03 can produce unrealistically high runup predictions.
IOVER	Wave overtopping	1 (on)	Prediction of overwash parameters.
IWTRAN	Wave transmission	0 (off)	Used to predict transmitted waves landward of an emerged structure or barrier. Only available in research version.
IPOND	Ponding	0 (off)	Used to predict the onshore migration of an emerged ridge and ponded runnel.
IWCINT	Wave-current interaction	1 (on)	Wave and current interactions.
IROLL	Roller effects	0 (off) or 1 (on)	Inclusion of volume flux due to roller development.
IWIND	Wind effects	0 (off)	Boundary condition typically close enough to shore that wind effects in CSHORE model are negligible.
ITIDE	Tide effects	0 (off) or 1 (on)	Inclusion of cross-shore volume flux associated with temporal variation of stillwater.
ILAB	Lab vs. field	1 (lab)	Lab setting requires synchronized water level and wave condition inputs. CSHORE will resample input conditions if ILAB set to field.

In addition to these parameter recommendations, there are additional considerations to evaluate when using CSHORE for runup prediction. The CSHORE manual recommends that the cross-shore position of the boundary condition be outside the surf zone where set-down or setup is very small. However, if there are any pronounced bathymetric features (e.g., shoals) observed in the offshore profile, sensitivity testing is recommended before determining placement of the boundary condition in relation to those features.

For runup predictions, CSHORE can be executed for a single, peak condition or in multiple time-steps representing some portion of a full storm duration. The peak condition option should only be used when sediment transport is not being modeled within CSHORE (e.g., using another method for erosion prediction or the ground is expected to be immobile). This approach is also only valid if the peak wave height is closely synchronized with the peak water level; otherwise, a full-storm or partial-storm time series may be necessary to capture peak runup conditions. When including sediment transport in the CSHORE simulation, a timeseries for the full duration of elevated waves (height and period) and water level should be modeled to capture the temporal changes in profile and resulting runup.

CSHORE uses a profile smoothing routine that is a function of both the DX value and root mean square wave height (H_{rms}) at the initial timestep where a larger initial H_{rms} will result in more profile smoothing. Therefore, it is recommended to apply an artificial timestep at the start of each simulation (t=0 seconds) with a wave height of 0.1 meters to minimize the smoothing algorithm applied by CSHORE. This recommendation applies to all CSHORE applications, regardless of storm duration.

As with any runup prediction tool, the results should be reviewed for reasonableness. Any spatial or temporal anomalies in runup results should be reviewed and addressed by modifying the CSHORE inputs or using an alternate runup method.

2. Supplemental Reference Tables and Figures

The tables and figures in this section are intended to serve as quick references for common empirical runup and overtopping methods. This section is not intended to be a comprehensive list of methodologies and applications. Tables in Section 2.1 and 2.2 (Table 4 and Table 5) are for reference and may contain example equations for methods that have a suite of equations applicable to multiple conditions. Refer to the documents listed in the “References” column to ensure the appropriate equation is being utilized. Table 6 in Section 2.3 contains information regarding the manual inclusion of wave setup to derive the runup BFE by the user. As noted in Section 3.5.1, this table is only applicable in cases where wave setup is not implicitly included in the SWEL values.

2.1. Wave Runup Equation Reference Table

Table 4: Reference Table for Common Wave Runup Equations and Their Usage

Runup Methods	Equation	Defined Variables	Reference Citation	Application Notes
TAW	$\frac{R_{2\%}}{H_{m0}} = \begin{Bmatrix} 1.75\gamma_b\gamma_f\gamma_\beta\xi_{0m}, 0.5 \leq \gamma_b\xi_{0m} < 1.8 \\ \gamma_b\gamma_f\gamma_\beta\left(4.3 - \frac{1.6}{\sqrt{\xi_{0m}}}\right), \gamma_b\xi_{0m} \geq 1.8 \end{Bmatrix}$	H_{m0}: significant wave height at toe ξ_{0m}: Iribarren number γ_b: influence factor for a berm γ_f: influence factor for roughness elements on slope γ_β: influence factor for angled wave attack	TAW (2002); FEMA (2007); TAW Slope H_{m0} Guidance (Section 7)	FEMA (2007) contains additional information regarding influence factor parameters. Additional details can be found in TAW (2002) including the berm reduction parameter and a more extensive variety of roughness coefficients. In the case of a depth-limited wave at the toe, the breaking wave, H_b, can be used. A spectral wave period, of $T_{m-1,0}$, is recommended for the Iribarren number calculation. The use of TAW method is questionable where the toe of structure is high relative to water levels, limiting the local wave height and calculated runups to small values. In these cases, FEMA (2007) recommends carrying out calculations at several locations across the surf zone to ensure that the highest runup result is recognized.

Runup Methods	Equation	Defined Variables	Reference Citation	Application Notes
Stockdon	$R_{2\%} = 1.1\left(0.35B_f(H_0L_0)^{1/2} + \frac{\left[H_0L_0\left(0.563B_f^{\ 2} + 0.004\right)\right]^{1/2}}{2}\right)$	L_0: deep water wavelength H_0: deep water significant wave height ξ_0: Iribarren number	Stockdon (2006)	A variation of this equation is available in Stockton (2006) to estimate runup under extreme dissipative conditions.
Van Gent	$\frac{R_{2\%}}{\gamma H_s} = \begin{Bmatrix} c_0\xi, \xi \leq p \\ c_1 - c_2/\xi, \xi > p \end{Bmatrix}$ where $c_2 = 0.25(c_1)^2/c_0$, $p = 0.5\, c_1/c_0$	$\gamma = \gamma_f\gamma_\beta$, a cumulative adjustment for slope roughness and wave directionality H_s: significant wave height at toe of the structure ξ: Iribarren number	Van Gent (2001)	Parameters c_0 and c_1 are given for different wave energy spectra. Generally, c_0 = 1.35 and c_1 = 4.7. See Van Gent (2001) for more detailed input factors and limitations. Physical and numerical model tests support the use of $T_{m-1,0}$ at the toe of coastal structures for use in Iribarren calculation.
SPM	See Figure 13 in the Shore Protection Manual	R: mean wave runup H_0': mean wave height T: mean wave period d_s: toe depth	USACE (1984); FEMA (2007); FEMA (2011b)	SPM Figure in FEMA (2007) is inconsistent and should be disregarded. Figure 13 is taken from PM-60 (noted in References); Used for vertical walls.

Runup Methods	Equation	Defined Variables	Reference Citation	Application Notes
EurOtop	$\frac{R_{2\%}}{H_{m0}} = 1.75\gamma_b\gamma_f\gamma_\beta\xi_{m-1,0}$ with a maximum of, $\frac{R_{2\%}}{H_{m0}} = 1.07\gamma_f\gamma_\beta\left(4.0 - \frac{1.5}{\sqrt{\gamma_b\xi_{m-1,0}}}\right)$	See TAW defined variables $\xi_{m-1,0}$: Iribarren number	EurOtop (2018)	Refer to EurOtop (2018) to ensure that the appropriate equations are being applied based on barrier geometry and shoreline characteristics. This general formula can be used when slope is no steeper than 1:2 and breaker parameter $\gamma_b\xi_{0m}$ is smaller than 1.8. Coefficient 1.75 is for a design and assessment approach. Other slopes and geometries should reference more specific EurOtop equations. A spectral wave period, of $T_{m-1,0}$, is recommended for the Iribarren number calculation.

2.2. Wave Overtopping Equation Reference Table

Table 5: Reference Table for Common Wave Overtopping Equations and Their Usage

Runup Methods	Equation	Defined Variables	Reference Citation	Application Notes
EurOtop	$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.026}{\tan\alpha}\gamma_b \xi_{m-1,0} \cdot exp(-\left(2.5\frac{R_C}{\xi_{m-1,0}H_{m0}\gamma_b\gamma_f\gamma_\beta\gamma_v}\right)^{1.3}$ with a maximum of, $\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.1035 \cdot exp(-\left(1.35\frac{R_C}{H_{m0}\gamma_f\gamma_\beta\gamma_*}\right)^{1.3}$	q: mean overtopping rate H_{m0}: significant wave height at toe $\xi_{m-1,0}$: Iribarren number γ_b: influence factor for a berm γ_f: influence factor for roughness elements on slope γ_β: influence factor for angled wave attack γ_*: geometric influence factor R_C: crest freeboard	EurOtop (2018)	Refer to EurOtop (2018) to ensure that the appropriate equations are being applied based on barrier geometry and shoreline characteristics. This general formula is for a sloped dike, levee, or embankment. Other structures and geometries should reference more specific EurOtop equations.
De Waal and Van Der Meer 1992	$Q^* = \bar{Q}/(gH_s{}^3)^{0.5}$ $Q^* = 8 \cdot 10^{-5} exp[3.1(rR^* - F/H_s)]$ where $R^* = [1.5\ m/(H_s/L_{0p})^{0.5}]$	$\bar{Q}$: mean overtopping rate H_s: incident significant wave height F: freeboard of structure crest above stillwater r: roughness coefficient R^*: estimated extreme runup normalized by H_s m: barrier slope L_{0p}: deepwater wavelength	FEMA (2007); Owen (1980); Vellinga (1986); De Waal and Van Der Meer (1992)	"Generalized" overtopping equation identified in the Atlantic/Gulf Coast Guidelines and Specifications (FEMA, 2007).

Runup Methods	Equation	Defined Variables	Reference Citation	Application Notes
Cox and Machemehl	$V_c = 1.1\sqrt{g\Delta R}$ $h_c = 0.38\Delta R$ $h(y) = \left[\sqrt{h_0} - \frac{5(y - y_0)}{A\sqrt{gT^2}}\right]^2$ where A is modified by $A_m = A(1 - 2m_{LW})$ for nonzero slopes	y: distance from barrier crest h: flow depth m_{LW}: slope landward of the barrier g: $acceleration\ of\ gravity$ ΔR: excess runup (runup elevation minus barrier crest elevation) A = 1 V_c = initial velocity h_c = initial flow depth T = wave period	Cox and Machemehl (1986); FEMA (2005); Oregon Department of Geology and Mineral Industries (DOGAMI) (2012)	The landward limit of the hazard zone is determined by $hV^2 = 200\frac{ft^3}{sec^2}$. More detailed information regarding this analysis can be found in FEMA (2005) and DOGAMI (2012). Evaluation of additional splashdown hazards should also be considered.

2.3. Wave Setup Reference Table

Table 6: Reference Table for Manual Inclusion of Wave Setup in Wave Runup Methodologies

Runup Method	Manually Include Setup for BFE?	Runup Elevation	Application Notes
TAW	Yes	SWEL + Setup + Runup	". . . it is recommended that the combined storm surge, astronomical tide and any wave setup at the toe of the slope be the water level to which the wave runup determined by the TAW methodology [be] added." (FEMA, 2011)
Stockdon	No	SWEL +Runup	"Runup statistics, R, were defined as the elevation of individual water-level maxima above the still-water level, merging contributions from setup and swash." (Stockdon, 2006) The detailed equation provided in Section 8.1 includes wave setup. There are other versions of the Stockdon equation that may not include setup.
Van Gent	No	SWEL + Runup	
SPM	No	SWEL + Runup	Figure 7-13 used for determining wave runup on vertical structures considers the depth of water at the structure for wave parameter selection. Best practices assume the water depth includes SWEL + setup for determining wave conditions.
ACES	No	SWEL + Runup	"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." (FEMA, 2007)
Runup 2.0	No	SWEL + Runup	"[The 2% exceedance runup height] is then added to the 1% annual-chance stillwater level without wave setup to obtain the total wave runup elevation for an FIS." (FEMA, 2007)

Runup Method	Manually Include Setup for BFE?	Runup Elevation	Application Notes
CSHORE	No	SWEL + Runup	"WSETBC(I) = wave setup (positive) or set-down (negative) ŋ(m) at x=0 relative to the still water level (SWL). If ŋ is not measured, use may be made of ŋ = 0 at x=0 as long as the seaward boundary x=0 is located outside the surf zone." (Kobayashi, 2009)
EurOtop	No	SWEL + Runup	Wave setup is implicitly reproduced in the physical model tests on which the runup and overtopping equations are based, but only over the length of foreshore reproduced in the physical model. There is, in general, no requirement to add on an additional water level increase for wave setup when calculating overtopping discharges using the methods reported in this document, unless the foreshore is very long and very gently sloped. In that case, numerical models should give the wave setup one or two wave lengths in front of the toe of the structure.

2.4. Corrected SPM Runup Figure

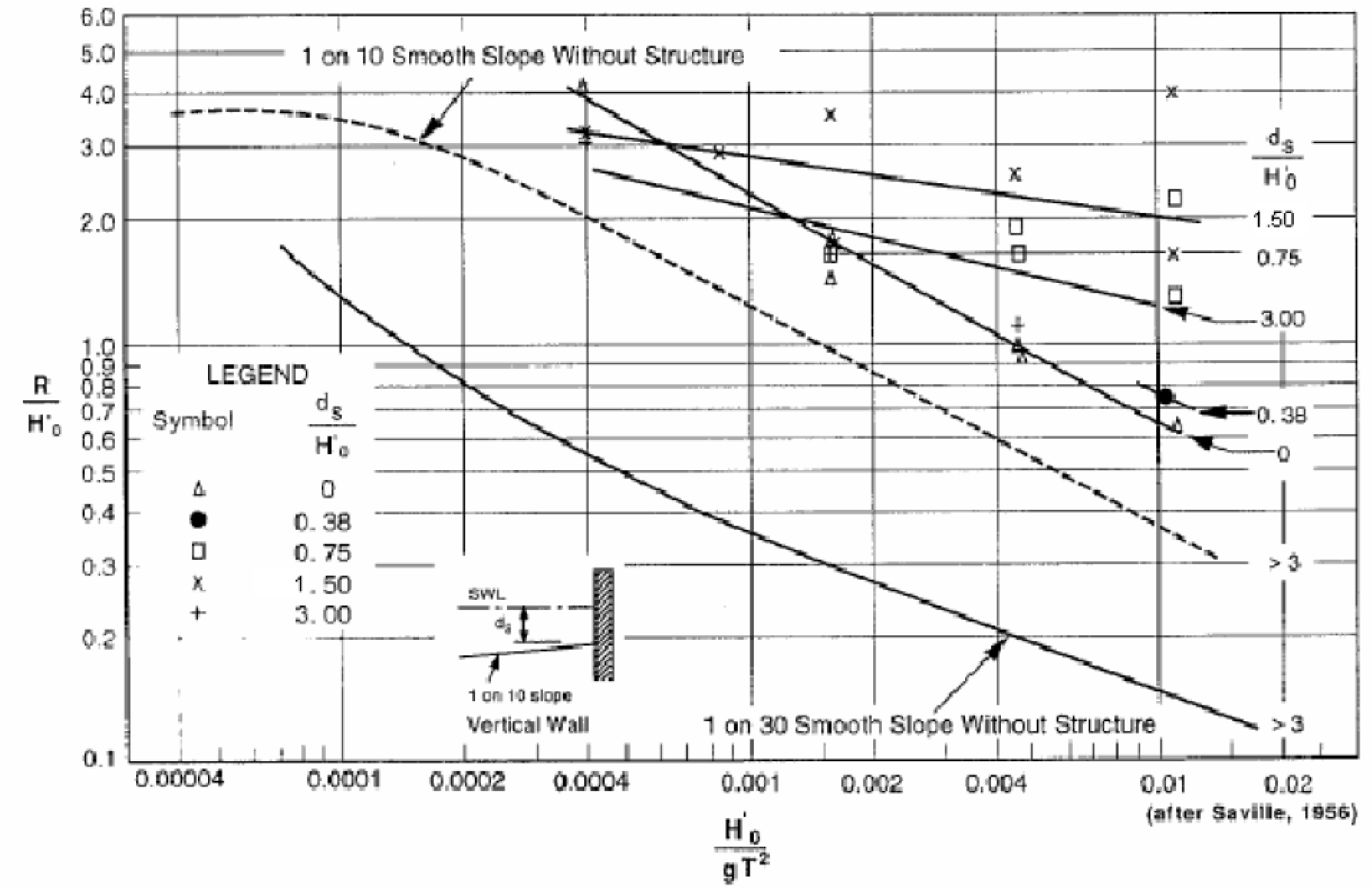

Figure 10: Corrected SPM Runup Figure 7-13