HMTAP Task Order 18 - Task 4 (Storm Surge Analysis)

Mississippi Coastal Flood Hazard Project

Wave Runup Method

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Many new methods and procedures for evaluating coastal hazards were developed over the course of the Mississippi Coastal Flood Hazard Project (Hazard Mitigation Technical Assistance Program Task Order 18). These changes promoted the adoption of advances in numerical modeling and statistical analysis for use in this Federal Emergency Management Agency (FEMA) project. However, in many cases these changes also created the need to further develop additional modifications to other previous methods.

In the Mississippi Coastal Flood Hazard Project, wave setup was included in each of the simulations of the synthetic storms. This allowed the effect of wave setup to be incorporated in the overall statistical analysis of coastal surge levels. However, this approach meant that the 1-percent-annual-chance (100-year) water level no longer corresponded to the 100-year surface used in the previous methods of analyses. As a result, the appropriate wave runup methodology required clarification.

The purpose of this report is to define a method for computing wave runup that is compatible with the new overall approach used in the project. Most of this definition has been extracted from the FEMA February 2007 update to the coastal guidelines *Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update* (FEMA, 2007) which describes a new wave runup methodology adopted from and based on the Dutch *Technical Advisory Committee for Water Retaining Structures* (hereafter called the TAW Manual) method for wave runup analysis. Information on a new methodology based on the application of a Boussinesq wave model is also provided, but for informational purposes only since it was not applied in this study. Although a Boussinesq solution of the wave equations is not new, there have been recent developments that make its application to wave runup more efficient and practical. The Boussinesq advanced numerical model of breaking wave conditions has been applied recently by the U.S. Army Corps of Engineers (USACE) to determine wave runup and overtopping conditions on levees in Louisiana. A large range of slope geometries have been modeled. The results have been tabulated and can be used with an interpolation routine for a wide range of slope shapes and wave characteristics.

As the methods in the FEMA procedures have evolved, it has become evident that it is necessary to differentiate between calculating wave runup on ocean shorelines with beaches or other slopes, and calculating wave runup on barriers such as levees, embankments, coastal armoring structures, and on some natural features with steep irregular profiles. This report addresses the use of the TAW wave runup methodology in Mississippi as applied on ocean shorelines and inland embankments and structures. Unless otherwise noted all figures are original or they were taken from the FEMA Coastal Guidelines document (FEMA, 2007).

Wave runup is the uprush of water from wave action on a shore barrier that intercepts the water level. The extent of wave runup can vary greatly from wave to wave in storm conditions, so that a wide distribution of wave runup elevations exist for a specific situation. The water wedge generally thins and slows during its flow up the barrier face as residual forward momentum in wave motion near the shore is fully reflected and transferred by friction and gravity. The notable characteristic of this process for present purposes is the wave runup elevation: the vertical elevation of the wave runup, R, above the stillwater elevation, ultimately attained by the extremity of the uprushing water (see Figure 1-1). Wave runup at a shore barrier can cause flood hazards above and beyond those from stillwater elevation inundation and incident wave geometry, as illustrated in Figure 1-1.

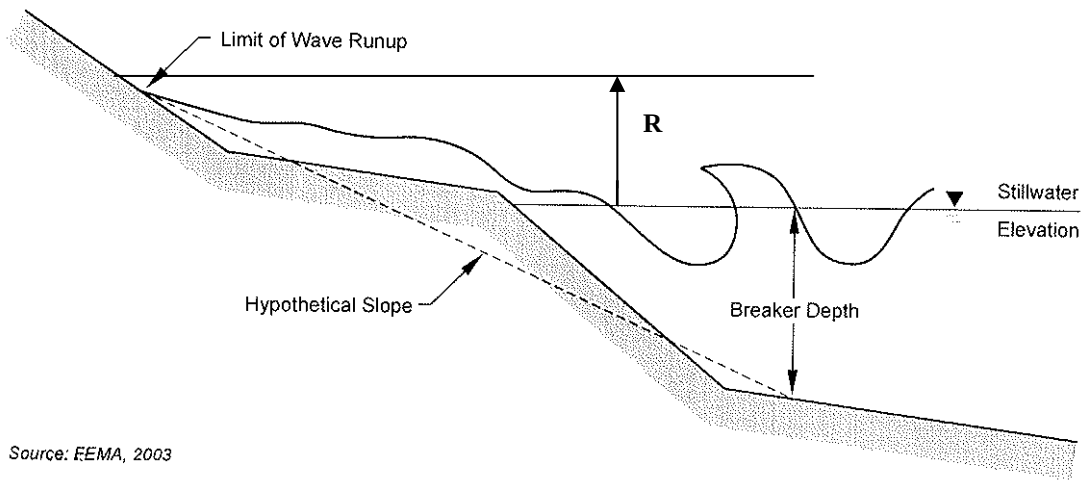

Figure 1-1: Wave Runup Sketch

Current policy for the National Flood Insurance Program (NFIP) is to define the wave runup elevation as the value exceeded by 2-percent of the wave runup events for a group of waves or a particular storm. The 2-percent value, commonly denoted as $R_{2\%}$, is defined in FEMA's *Atlantic Ocean and Gulf of Mexico Coastal Guidelines Update* (FEMA, 2007). This 2-percent wave runup elevation differs 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 wave runup statistic corresponds to a 2-percent exceedance occurrence in several hours of waves at the 1-percent storm surge conditions. To avoid confusion, the 2-percent wave runup is referred to as the "total wave runup," or just the "wave runup," and is denoted as $R_{2\%}$. Unless otherwise indicated, the wave runup referred to in all following subsections of this report is the 2-percent wave runup.

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

$$\xi = \frac{m}{\sqrt{H / L}} \tag{1}$$

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 values. For this report, two wave characteristics in the Iribarren number are used: that based on the significant deepwater wave height (*Ho*) 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 definition for ξ on a sandy beach is as follows:

$$\xi_o = \frac{m}{\sqrt{H_o \setminus L_o}} \tag{2}$$

where H_o is the unrefracted deepwater significant wave height, and L_o is the deepwater wave length. L_o is defined as follows, where g is the gravitational constant and T is the wave period

$$L_o = \frac{g}{2\pi} T^2 \tag{3}$$

The beach profile slope, m, is the average slope from the wave runup limit out to the breaking depth associated with the significant wave height.

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

$$R_{2\%} = 0.6\xi_o H_o = 0.6m\sqrt{H_o L_o} \tag{4}$$

As discussed in the next section, for the case of wave runup on a barrier, the Iribarren number is formulated using the significant wave height at the toe of the barrier.

4.1 METHODOLOGY FOR WAVE RUNUP ON BARRIERS

In this subsection, "barriers" include steep dune features and coastal armoring structures such as revetments. Wave 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. Wave runup on structures can also be affected by antecedent conditions resulting from the previous waves and structure composition. Because of these complexities, wave runup on structures is best calculated using equations developed with tests on similar structures and with similar wave characteristics and 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 Coastal Engineering Manual (CEM) (USACE, 2003). This approach, referred to as the 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 wave runup methods and equations for structures of similar form may provide more accurate results for a particular structure. For each situation, the applicability of any wave runup method should be evaluated carefully to verify its appropriateness. Figure 4-1 shows a general cross-section of a coastal structure, a conceptual diagram of wave runup on a structure, and definitions of parameters. Note that the reference elevation for wave runup calculations is the mean water level (MWL) at the toe of the slope, and wave setup landward of this point is included in the wave runup.

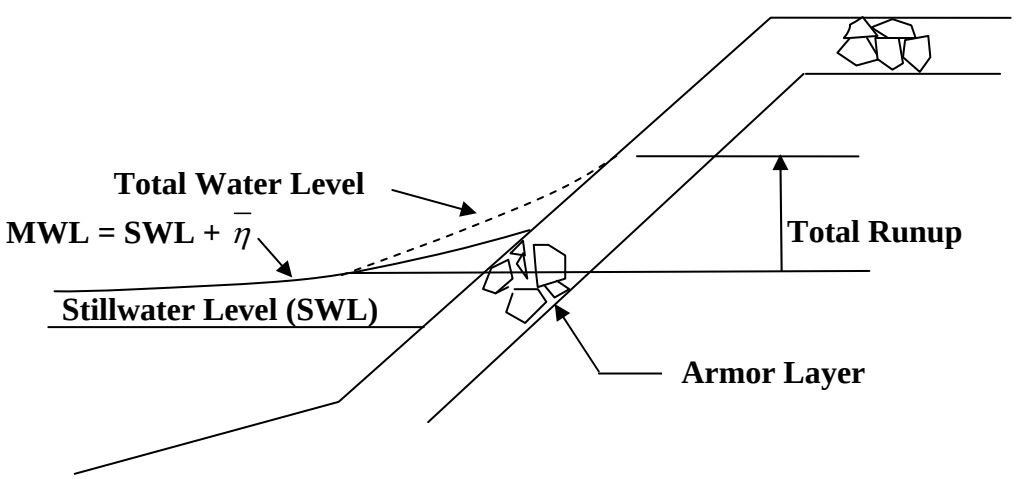

Figure 4-1: Wave Runup on Coastal Structure, Definition Sketch. Note $\bar{\eta}$ is Wave Setup.

Most of the wave runup research and literature shows a clear relationship between the vertical wave runup elevation and the Iribarren number. The wave runup on barriers is stated relative to the significant wave height at the toe of the barrier (H_{mo}), defined as $H_{mo} = 4.0\sqrt{E}$, where E is the wave energy density or, equivalently, the area under the wave energy spectrum. The wave period for this method is a relatively new parameter called the spectral wave period in the TAW methodology ($T_{m-1.0}$). This is readily obtained when the modeled or measured waves are described with a spectrum as:

$$T_{m-1.0} = \frac{\int_0^\infty \frac{2\pi}{\sigma} S(\sigma) d\sigma}{\int_0^\infty S(\sigma) d\sigma} \tag{5}$$

which is seen to emphasize the lower frequencies (longer wave periods), and where $S(\sigma)$ is the wave energy density as a function of the wave angular frequency, $\sigma(= 2\pi / T)$. For spectra with a single peak, an approximate relationship for this parameter in terms of the peak period (T_p) is:

$$T_{m-1.0} = \frac{T_p}{1.1} \tag{6}$$

For most conditions, T_p and the significant wave period $T_{1/3}$ are practically identical.

The TAW report (van de Meer, 2002) should be consulted for recommended procedures if wave parameters are not available directly at the toe of the slope. Alternatively, in situations where there is a shallow platform in front of the slope, where the wave spectra are distorted by excess flattening or are double peaked, or where the profile geometry of the slope is complex, the more comprehensive Boussinesq modeling method may be needed to transform the waves to the base of the slope.

Figure 4-2 shows the relative wave runup (R/H_{mo}) plotted against the Iribarren number for two different methods: van der Meer (2002) and Hedges and Mase (2004). In Figure 4-2, both wave 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 wave runup and ξ_{om} for values of ξ_{om} below approximately two. Hedges and Mase interpreted the vertical intercept in Figure 4-2 as wave setup, which is present even for a zero slope.[1] 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 wave runup methodology) is recommended here. In this method it is necessary to characterize the wave conditions in terms of ξ_{om} .

[1] However, wave setup does decrease with profile slope.

In calculating the Iribarren number to apply in Equation (8), use Equation (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:

$$H_{mo} = 4.0\sqrt{E} \tag{7}$$

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 a different value can be justified. 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 $0.5 < \xi_{om} < 8\text{-}10$, 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 (8) where L_o is based on $T_{m-1.0}$.

$$\xi_{om} = \frac{m}{\sqrt{H_{mo} / L_o}} \tag{8}$$

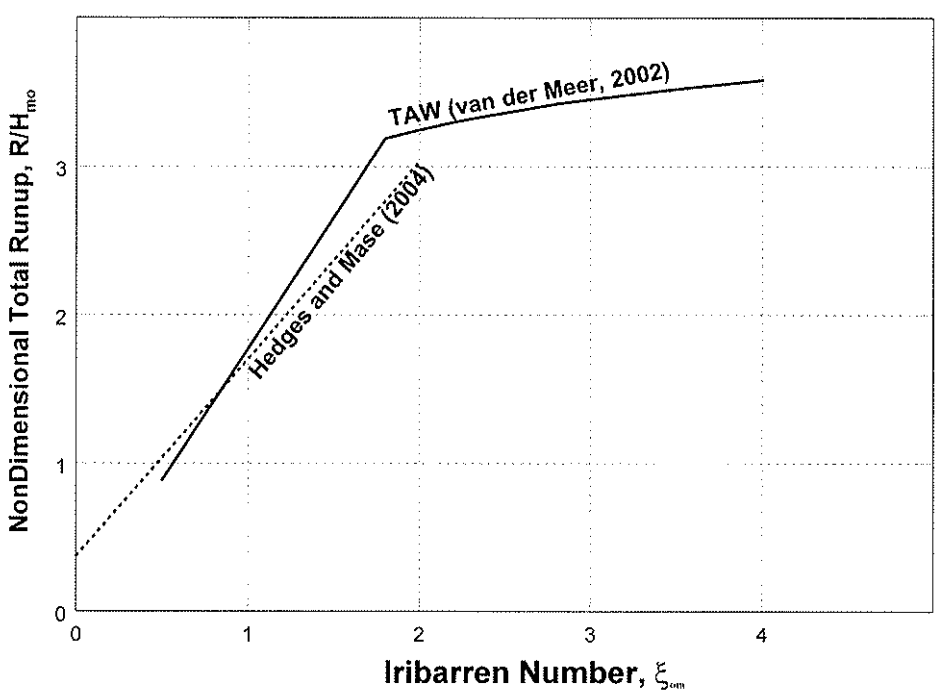

Figure 4-2: Non-Dimensional Total Wave Runup vs. Iribarren Number

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

$$R = H_{mo}\begin{Bmatrix} 1.77\gamma_r\gamma_b\gamma_\beta\gamma_p\xi_{om} & 0.5 \le \gamma_b\xi_{om} < 1.8 \\ \gamma_r\gamma_b\gamma_\beta\gamma_p\left(4.3 - \dfrac{1.6}{\sqrt{\xi_{om}}}\right) & 1.8 \le \gamma_b\xi_{om} \end{Bmatrix} \tag{9}$$

where:

- R is the 2-percent wave runup
- 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 4-1, and definition sketches are provided in Figures 4-3 and 4-4. The reference water level at the toe of the barrier for wave runup calculations is the 1-percent MWL.[2] For a smooth, impermeable structure of uniform slope and with normally incident waves, each of the γ wave runup reduction factors is 1.0.

[2] It is noted that there is some unavoidable, minor uncertainty in the wave runup calculations that stems from the unknown magnitude of the wave setup/setdown present in the model tests from which the TAW relationships were developed. If the model tests were conducted with waves that broke at the toe of the slope, there would be some setdown (at most about 5 percent of the breaking wave height) at this location; however, if the waves in the model tests commenced breaking in a considerably greater depth than the depth at the toe of slope, the associated wave runup would include an unknown (but presumably small) amount of setup. Assuming that the water level at the toe of the slope in the model tests included no (or acceptably small) setdown/setup, the proper application of the TAW method is to consider the mean water level at the toe of the slope as the reference level, and not to include wave setup explicitly landward of the toe of the slope.

Table 4-1: Summary of γ Wave Runup Reduction Factors

Wave Runup Reduction Factor	Characteristic/Condition	Value of γ for Wave Runup	
Roughness Reduction Factor, γ_r	Smooth Concrete, Asphalt, and Smooth Block Revetment	γ_r = 1.0	(10)
	One Layer of Rock with Diameter, D. H_s / D = 1 to 3	γ_r = 0.55 to 0.60	
	Two or More Layers of Rock. H_s / D = 1.5 to 6	γ_r = 0.5 to 0.55	
	Quadratic Blocks	γ_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 4-3 for Definition of *B, L_{berm}* and Other Parameters	$\gamma_b = 1 - \frac{B}{2L_{berm}}\left[1+\cos\left(\frac{\pi d_h}{x}\right)\right], 0.6 < \gamma_b < 1.0$ $x = \begin{cases} R \text{ if } \frac{-R}{H_{mo}} \le \frac{d_h}{H_{mo}} \le 0 \\ 2H_{mo} \text{ if } 0 \le \frac{d_h}{H_{mo}} \le 2 \end{cases}$ Minimum and maximum values of γ_b = 0.6 and 1.0, respectively	(11)
Wave Direction Factor, γ_β, β is in degrees and = 0° for normally incident waves	Long-Crested Waves	$\gamma_\beta = \begin{cases} 1.0, 0 < \lvert\beta\rvert < 10° \\ \cos(\lvert\beta\rvert - 10°), 10° < \lvert\beta\rvert < 63° \\ 0.63, \lvert\beta\rvert > 63° \end{cases}$	(12)
	Short-Crested Waves	$1 - 0.0022\lvert\beta\rvert, \lvert\beta\rvert \le 80°$ $1 - 0.0022\lvert 80\rvert, \lvert\beta\rvert \ge 80°$	(13)
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 and porosity = 0.5, for smaller porosities, proportion γ_p according to porosity. See Figure 4-4 for definition of porosity.	(14)

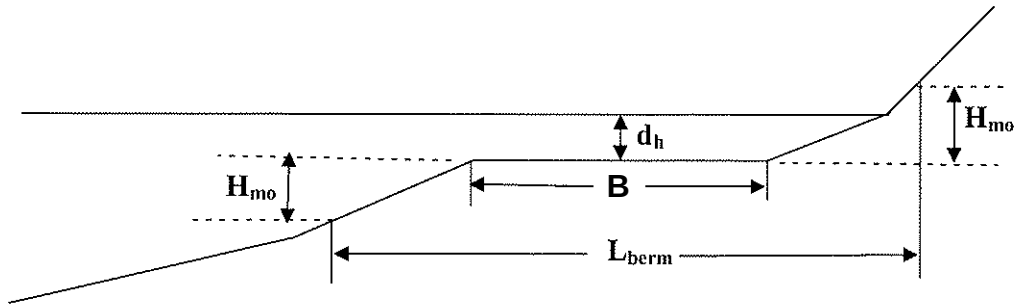

Figure 4-3: Berm Parameters for Wave Runup Calculations

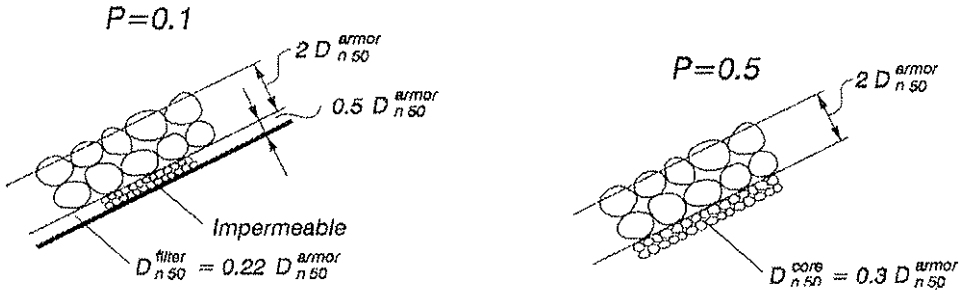

Figure 4-4: Structure Porosity Definition

Wave runup on structures is dependent on the characteristics of the nearshore and structure geometries. Hence, better wave runup estimates may be possible with other wave runup equations for particular conditions. Other wave runup methods may be used 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.

4.2 MAXIMUM WAVE RUNUP FROM SMALLER WAVES

In some cases, neither of the previously described methods for computing wave 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 wave 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 wave runups to small values. In these cases, it is necessary to calculate wave runup with equations in the form of Equation (9) 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 the calculations with the largest waves in a given sea condition may not produce the highest wave runup, but that the highest wave runup will be the result of waves breaking at an intermediate location within the breaking zone. This procedure recognizes that a range of wave heights will occur in conjunction with the 1-percent water level.

The recommended procedure is to consider a range of (smaller) wave heights inside the surf zone in wave runup calculations. The concept of a range of calculated wave runup values is depicted schematically in Figure 4-5, where an example transect and setup water-surface profile are shown. Figure 4-5 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 4-5. The calculation of ξ_{om} at each location uses the deshoaled deepwater wave height corresponding to the breaker height, and the deepwater wave length and average slope calculated from the breaker point to the approximate wave runup limit. Note that this average slope, also called composite slope as defined in the CEM (USACE, 2003) and Shore Protection Manual (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_{om} equation. Also, the wave height decreases with shallower depths, reducing the wave steepness in the denominator of the ξ_{om} equation. Hence, as plotted in Figure 4-5, ξ_{om} increases as smaller waves closer to shore are examined, increasing the relative wave runup (*R/H*). However, because the wave height decreases, the wave 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 wave runup using the methods described earlier for wave runup on a barrier. This requires iteration for this location to determine the average slope based on the differences between the wave runup elevation and the profile elevation at the location and the associated cross-shore locations. Calculations should be iterated until the wave runup converges for this location.
2. Repeat the wave runup calculations at different cross-shore locations until a maximum wave runup is determined.

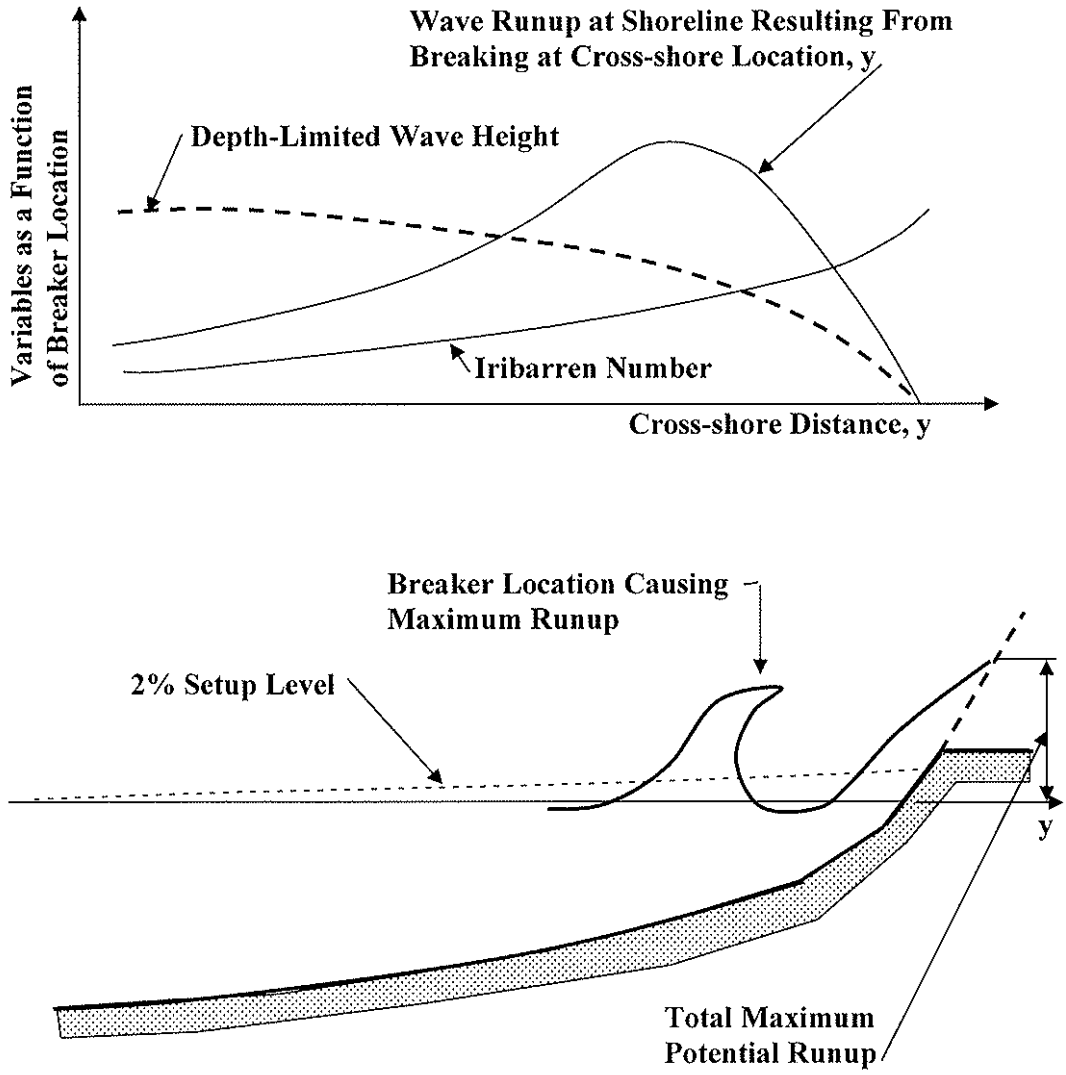

Figure 4-5: Example Plot Showing the Variation of Surf Zone Parameters

4.3 WAVE RUNUP IN SPECIAL SITUATIONS

One important consideration is that a mean wave runup elevation below the crest of a given barrier does not necessarily imply that the barrier will not occasionally be overtopped by floodwaters. Other cases may yield results of more immediate concern, in that FEMA's wave runup model (RUNUP 2.0) may calculate a wave 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 wave runup along the imaginary slope extension exceeds 3 feet. When the wave runup overtops a barrier (wave overtopping), such as a partially eroded bluff or a structure, the floodwater percolates into the bed or runs along the back slope (wave rundown as sheet flow or runoff) until it reaches another flooding source or a ponding area. The wave rundown sheet flow that occurs past the slope crest is 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 the Base Flood Elevations (BFEs) shown.

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 4-6 and discussed below.

When the potential wave runup is at least 3 feet above the barrier crest, a Zone VE is delineated landward of the barrier, as shown in Figure 4-6. The BFE for that Zone VE is capped as 3 feet above the crest of the barrier. When the wave runup depth crest is 0.1 to 1.5 feet above the barrier, the Zone VE BFE is the wave runup elevation (rounded to the nearest whole foot), and a Zone AO with a depth of 1 foot should be mapped landward until another flooding source is

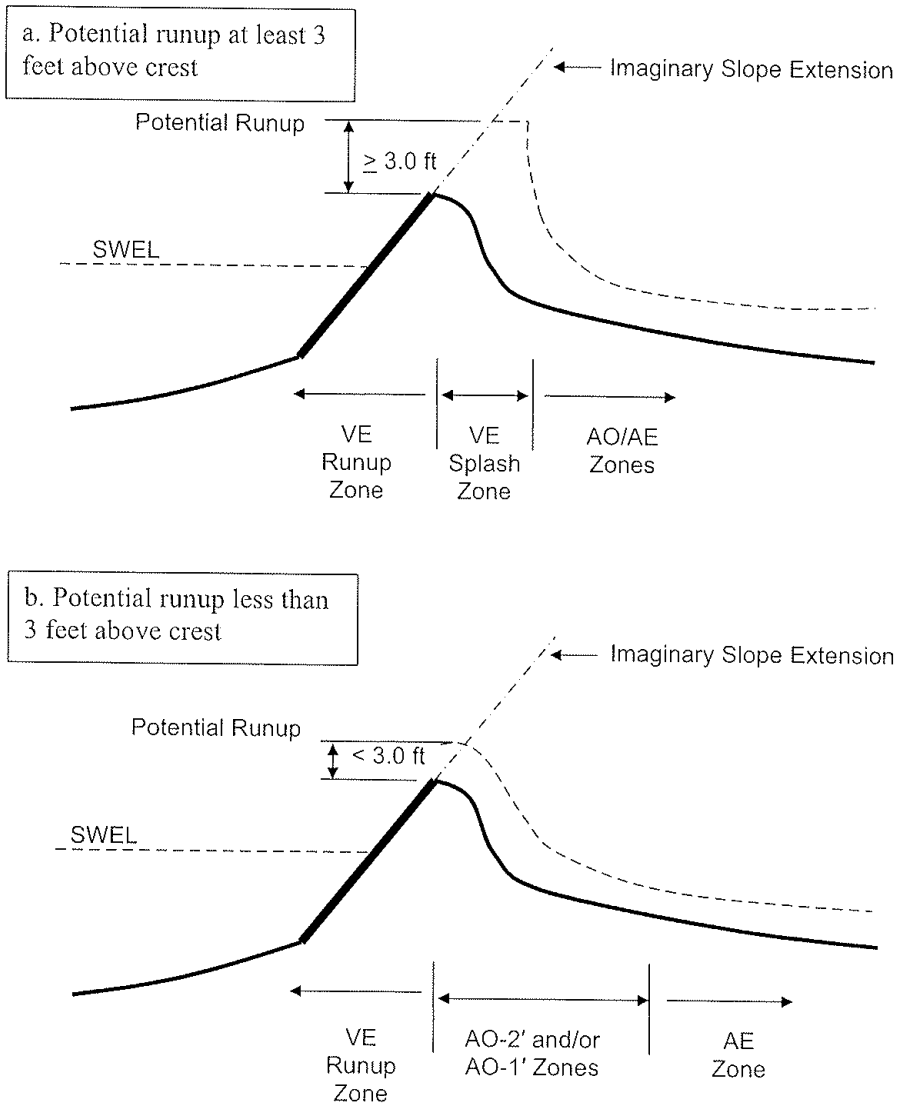

Figure 4-6: Simplified Runoff Procedures (Zone AO)

encountered (Zone AE) or the floodplain limit is reached (Zone X). Similarly, for a wave runup depth of 1.5 to 2.9 feet above the barrier crest, the Zone VE BFE is the wave runup elevation (rounded to the nearest whole foot). In this case, however, a Zone AO with depth of 2 feet should be mapped, then transitioned landward into a Zone AO with a depth of 1 foot, and then into subsequent flood insurance risk zones, if any.

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 wave runup elevations for such situations, as illustrated in Figure 4-7 (French, 1982). An extension to the bluff face slope permits the computation of a hypothetical wave runup elevation for the barrier, with the imaginary portion given by the excess height $R' = (R\text{-}C)$ between the calculated wave runup and the bluff crest. Using the height (R') and the plateau slope (m), Figure 4-8 defines the inland limit to a wave runup (X) corresponding to the wave runup above the bluff crest (mX), or an adjusted wave 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, but not used in this project is provided in "Overland Bore Propagation Due to an Overtopping Wave" (Cox and Machemehl, 1986). These wave runup assessment procedures are given for general guidance, but they may not be entirely applicable in certain situations. For example, wave runup elevations need to be fully consistent with the wave setup and wave overtopping assessments.

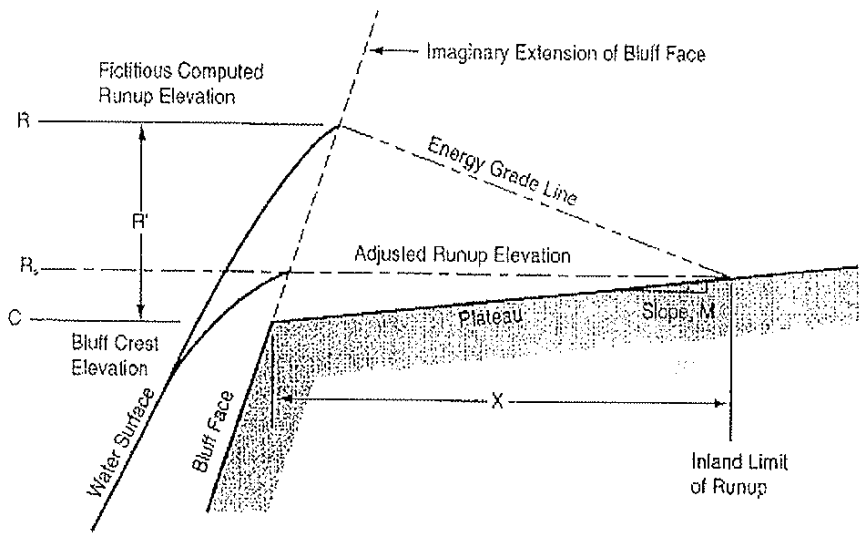

Figure 4-7: Treatment of Runup onto Plateau above Low Bluff

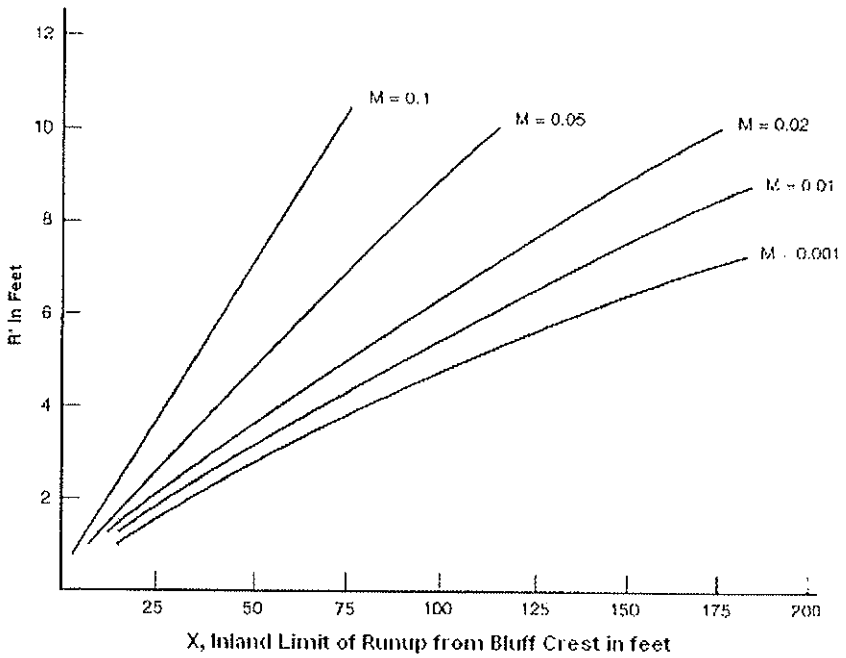

Figure 4-8: Curves for Computation of Wave Runup Inland of Low Bluffs

This section provides additional detailed description of the wave runup methodology applied in the Mississippi project.

The wave runups for steep slopes along particular transects from the Mississippi Coastal Flood Hazard Study were calculated using the TAW methodology. In the process of conducting the wave analysis using the FEMA Wave Height Analysis for Flood Insurance Study (WHAFIS) model, selected wave transects with a slope generally greater than 1:10 and intersected by the 1-percent annual flood level were designated for wave runup analyses.

The source of the wave height information is the wave height at the toe of the structure for conditions representative of the 1-percent annual flood level. This provision of the TAW method means that the wave height from the WHAFIS analysis can be used directly where the effects of wave setup have been incorporated in the surge level. However, it is important to distinguish between two cases of wave conditions. If the wave height at the toe of the structure is not controlled by depth-limited breaking (i.e., the wave height is less than 0.78 times the water depth at the toe of the structure), then it is assumed that the controlling wave height taken from WHAFIS is the maximum controlling wave height ($H_{1\%}$). This must be divided by 1.6 to obtain the equivalent H_{mo} used in the TAW analysis. In the second case, the controlling wave height taken from the WHAFIS analysis is determined by depth-limited breaking (wave height equal to 0.78 the water depth). Because WHAFIS uses a monochromatic representation of the waves, the depth-limited breaking wave height is the controlling wave height and it is also conservatively assume to be the value of H_{mo} to be used in the TAW method.

The spectral wave period (T_{m-1}) used in the TAW was computed from the peak spectral wave period (T_p) calculated by WHAFIS. The spectral wave period was calculated by dividing the peak spectral wave period by 1.1 as described in Equation (6).

All analyses assumed the wave attack was perpendicular to shore (γ_β=1.0). The influence of a slope-fronting horizontal berm was included by adjusting the γ_b factor. Where a berm was present, the γ_b factor was taken from Figure 13 of the TAW Manual (page 18), depending on H_{mo}, the water depth over the berm, and berm width B. Where a berm was not present, the γ_b was equal to 1.0. In general, the wave runup surface was considered to be smooth with an associated γ_r factor of 1.0. In most cases, this analysis is slightly conservative.

6.1 LOOK AHEAD ON WAVE MODELING ADVANCEMENTS

Wave models in general are becoming more sophisticated and able to account for more of the complexities of water waves. A rapidly developing class of these is the Boussinesq model, which is a phase resolving model both commercially and publicly available, with the commercial models being generally more user friendly. In addition to wave setup, Boussinesq models can be applied to the calculation of wave runup. In conjunction with the development of the Pacific Coast Guidelines and Specifications, two-dimensional (2-D) Boussinesq models were applied to calculate total wave runup, and the average and oscillating components were calculated separately. The comments below are based, in part, on an assessment of these Boussinesq results.[3] 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. As noted, many versions of Boussinesq models have been developed and each has its special advantages and capabilities. Two Boussinesq models which are known to have been used for FEMA-type applications are discussed in this chapter.

6.2 POTENTIAL USE OF BOUSSINESQ MODELS IN FEMA APPLICATIONS

The primary calculation for which FEMA could consider application of Boussinesq models is wave runup. As discussed previously, most present methods for calculating wave runup are empirical and based on wave tank tests. The geometries that have been tested are limited, although attempts have been made to broaden the range of these empirical methods by developing procedures that allow application to geometries beyond those tested. These empirical methods are presented in the USACE CEM and the TAW Manual, among others. In addition, results are available in various reports, journal publications, and conference proceedings. Many of the cases included in these references are for specialized geometries. Because this area is one of increasing interest, the available results are increasing rapidly, as are the capabilities of Boussinesq models.

It is estimated that at least 50 versions of Boussinesq models have been developed internationally. Many of these models have some unique feature, such as wave-breaking algorithms, two-layer systems, or special treatment of vertical acceleration. Thus, it should not be expected that all of these models will be equally applicable to FEMA needs. In particular, Boussinesq models are based on the long-wave formulation in which the vertical acceleration is small. In the wave runup problem, vertical accelerations can be large, especially on structures with steep slopes. This non-negligible, vertical acceleration can be handled in Boussinesq models, but requires special treatment (Professor Patrick Lynett, Texas A and M University, personal communication, 2007).

In various recent studies following Hurricane Katrina, Boussinesq models have been applied to represent the interaction of waves and levees. The processes of interest include wave runup,

[3] A commercially available model from the Danish Hydraulics Institute (DHI) was applied successfully by DHI in the development of the Pacific Coast Guidelines and Specifications.

wave overtopping rates, and velocities on the lee side of the levee. Wave runup appears to be the simplest of the above three processes.

From December 2005 through June 2006, the Interagency Performance Evaluation Task Force (IPET), organized by USACE, conducted a comprehensive effort to determine, among other issues, the causes of failure of the New Orleans levees. Included within this IPET study was an application of the Boussinesq model COULWAVE (Cornell University Long and Intermediate Wave Model) developed by Professors Patrick Lynett of Texas A&M University and Philip Liu of Cornell University to represent wave runup, overtopping, and rundown on the lee side of the levees. In addition to the DHI application mentioned in Section 6.1, it appears that the greatest amount of effort has been made with the COULWAVE model to represent problems of interest to FEMA. Thus, the COULWAVE model serves as the basis for the following discussion. In the application to the levee considerations noted above, maximum rundown velocities of approximately 15 feet per second were calculated. To the best available knowledge, the magnitudes of these velocities have not been verified.

Lynett has compared the wave runup calculated by COULWAVE with empirical formulations. Figure 6-1 presents (in Panel a) empirical measurements of dimensionless wave runup with the Iribarren Number ξ. It is seen that there is good agreement between the non-dimensional wave runup and the Iribarren Number. The remaining panels compare, for the various slopes indicated, the wave runup calculated by COULWAVE (shown as open circles) and standard engineering empirical methods.

Subsequent to the IPET investigation, the redesign phase of the Louisiana levees also applied the COULWAVE model; however, this application was used in the limited role of calculating the average overtopping rates due to irregular waves. It was found in that effort that the most appropriate role of the Boussinesq model was for those cases where complex levee or foreshore geometries were present such that application of the normal empirical methods (such as in TAW) did not appear valid, the Boussinesq model was applied for both wave transformation to the levee and the ensuing interaction. In cases where the levee geometry was fairly consistent with available model tests, but the foreshore geometry was complex, the model was used to transform the waves to the base of the levee slope and then the empirical methods (TAW) were applied. Figure 6-2 shows a comparison of the COULWAVE overtopping results with the TAW empirical results for relatively simple levee geometries. These COULWAVE calculations considered a TMA spectrum with a gamma value = 3.0. It is noted that Figure 6-2 encompasses a broad range of average overtopping, Q, and that the normal acceptable range is $0.01 < Q < 0.1$ cfs/ft, and in this range there are substantial differences between the two methods shown.

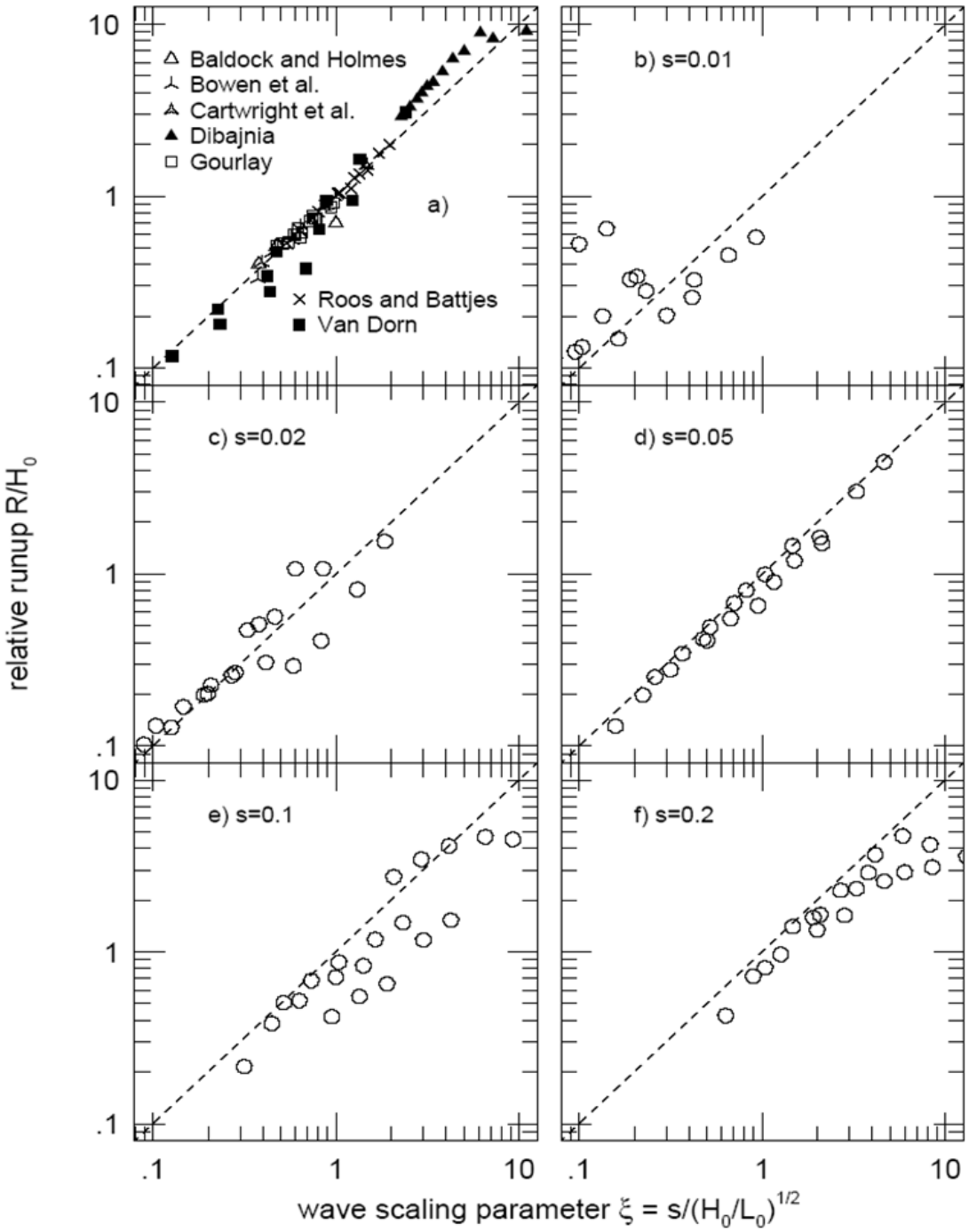

Figure 6-1: Panel a): Variation of Non-dimensional Wave Runup, R/H_o, and Iribarren Number to Show Linear Relationship. Panels b) through f) Present Non-Dimensional Runup Comparisons Between COULWAVE and Standard Engineering Empirical Relationships (Patrick Lynett, Personal Communication).

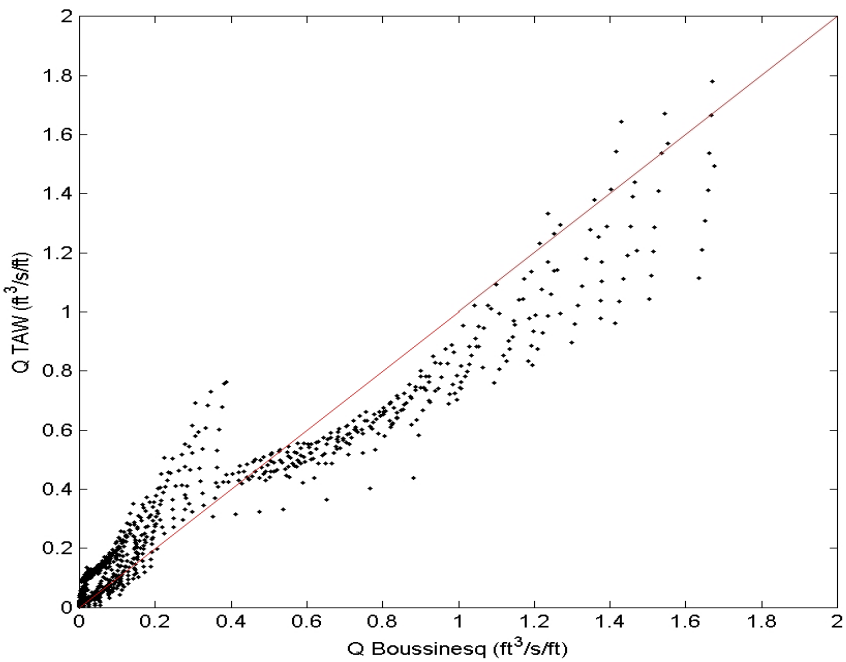

Figure 6-2: Comparison of Overtopping Rates Calculated With COULWAVE and the TAW Method (Patrick Lynett, Personal Communication).

6.3 SUMMARY OF APPLICATION OF BOUSSINESQ MODELS IN FEMA APPLICATIONS

6.3.1 Advantages of Boussinesq Models for Application to FEMA

In summary, the chief advantage of the Boussinesq models is their capability to incorporate unusual topographic features and their ability to provide responses without special considerations or approximations, or the need to couple models. These include bathymetric variations such as non-planar slopes and wave setup without additional modeling or calculations, and the capability of investigating the role of various wave spectral characteristics. As shown, Boussinesq models can represent wave runup and wave overtopping reasonably well.

6.3.2 Disadvantages of Boussinesq Models for Application to FEMA

The two most obvious disadvantages in application of the Boussinesq models are the learning curve required and the computer expense. With further development and streamlining of Boussinesq models, it may be possible to apply Boussinesq models, with minimal investment in model study, to a level where an individual with a reasonable amount of training can apply the model with confidence to new situations. Assuming that future Flood Insurance Study (FIS) contracts will be awarded to companies that may not have an in-house Boussinesq capability, and recognizing the nature of FISs, it would appear that the near-term role of Boussinesq models to FEMA needs may be as described in the following section.

6.4 RECOMMENDATIONS FOR APPLICATION OF BOUSSINESQ MODELS IN FEMA APPLICATIONS

The FEMA needs which can be best addressed by Boussinesq models should be identified. Where possible, Boussinesq models should be applied to provide the required results in parametric form in terms of a minimum number of (non-dimensional) variables. In this application, the Boussinesq models would be applied by a specialist in the same way that physical model tests have been used in the past to build up results that can be applied generically. Thus, when the need arises, the required results could be determined without actual use of the Boussinesq model for each case. In those cases in which the geometries are unique or for other reasons, cannot be represented by the non-dimensional representations established earlier, a Boussinesq model would be applied. However, in the near future, it is not reasonable to anticipate each contractor to maintain or develop an in-house Boussinesq capability. Rather, a more efficient approach would be for that capability to be provided through a consulting basis on an as-needed basis. To identify the needs for parametric development, past studies could be reviewed for such needs and as new studies progress, the needs could be assembled as they occur.

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