Wave Runup and Overtopping

FEMA Coastal Flood Hazard
Analysis and Mapping Guidelines
Focused Study Report

February 2005

Focused Study Leader

Chris Jones, P.E.

Team Members

Ida Brøker, Ph.D.
Kevin Coulton, P.E., CFM
Jeff Gangai, CFM
Darryl Hatheway, CFM
Jeremy Lowe
Ron Noble, P.E.
Rajesh Srinivas, Ph.D., P.E.

Table of Contents

<u>Tables</u>

Figures

Acronyms

ACES	Automated Coastal Engineering System
ANEMONE	Advanced Non-Linear Engineering Suite of Models for the Nearshore Environment
BFEs	Base Flood Elevations
CCSTWS	Coast of California Storm and Tidal Wave Study
CDIP	Coastal Data Information Program
CEDAS	Coastal Engineering Design and Analysis
CEM	Coastal Engineering Manual
CHAMP	Coastal Hazard Analysis Modeling Program
FEMA	Federal Emergency Management Agency
FIRM	Flood Insurance Rate Map
FIS	Flood Insurance Study
GIS	Geographic Information Systems
LIDAR	LIght Detection And Ranging
LOMR	Letter of Map Revision
NFIP	National Flood Insurance Program,
NGVD	National Geodetic Vertical Datum
PWA	Phillip Williams & Associates
SPM	Shore Protection Manual

TAW	Technical Advisory Committee for Water Retaining Structures
USACE	U.S. Army Corps of Engineers
WHAFIS	Wave Height Analysis for Flood Insurance Studies

1 INTRODUCTION

Water levels along coastal shorelines vary through time, depending upon tides and incident wave conditions. These water levels can be thought of as being composed of two components: 1) a static (or assumed static or slowly varying) mean water level associated with astronomical tides, storm surges, and wave setup; and 2) a fluctuation about that mean (swash) associated with surf beat and the motion of individual waves at the shoreline.

As used in this report*, *wave runup* refers to the height above the stillwater elevation (tide and surge) reached by the swash (see Figure 1). Runup is a very complex phenomenon, that is known to depend on the local water level (including surf beat or infragravity wave effects), the incident wave conditions (height, period, steepness, direction), and the nature of the beach or structure being run up (e.g., slope, reflectivity, height, permeability, roughness).

Runup guidance is largely empirical, and typically is based either on field measurements on beaches or on laboratory measurements on structures. Most guidance relates runup to the surf similarity parameter ξ (ratio of the barrier slope to the square root of the wave steepness) as a means of reducing the number of variables and generalizing the applicability of specific measurements or tests.

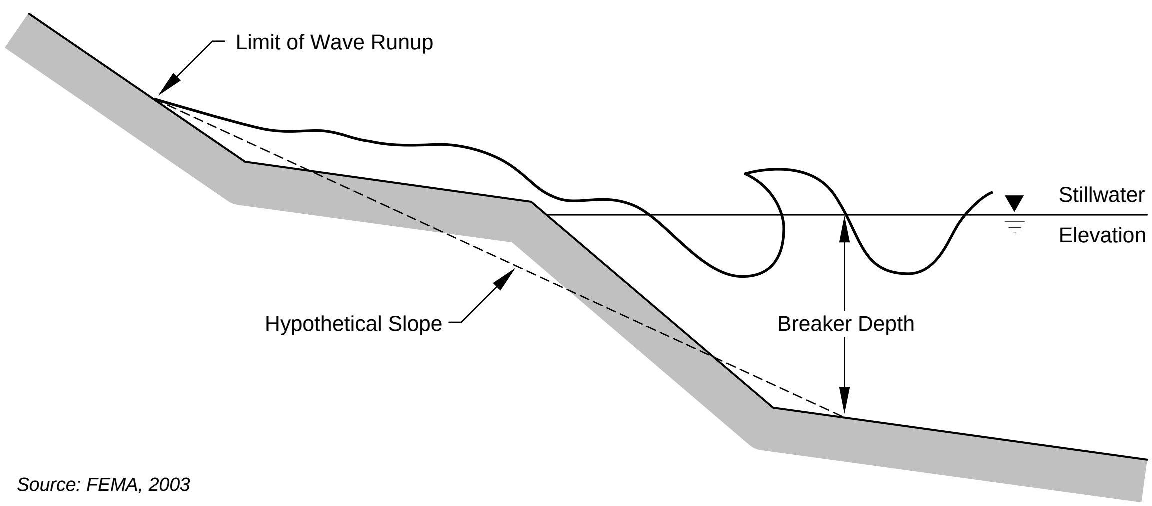


Figure 1. Wave runup sketch.

* Using this definition, which is consistent with current Federal Emergency Management Agency (FEMA) guidance, wave runup includes wave setup. An alternate definition for wave runup would exclude the wave setup component such that the runup is equal to the height above the stillwater elevation plus setup reached by the swash. The definition selected for use should be determined in conjunction with work carried out by the Wave Transformation and Wave Setup Study Groups.

As used in this report, *wave overtopping* refers to the volumetric rate at which runup flows over the top or crest of a slope, be it a beach, dune, or structure.

This report provides recommendations for:

- development of wave runup and overtopping guidance for Study Contractors completing Flood Insurance Studies (FIS) or restudies along the Pacific shorelines of California, Oregon, and Washington;
- development of wave runup and overtopping guidance for use by Study Contractors along sheltered (i.e., non-open coast) shorelines throughout the continental United States; and
- review of existing wave runup and overtopping guidance for use along the shorelines of the Atlantic Ocean and Gulf of Mexico.

Note that any recommendations or work on runup and overtopping must be integrated with recommendations and work on other topics, e.g., stillwater, wave setup, wave transformation, coastal structures, event based erosion, hazard zones, and tsunamis.

1.1 CATEGORIES AND TOPICS

Five wave runup/overtopping topics were identified at Workshop 1, and are identified below. The topic with the highest priority was Topic 12 (use of mean vs. higher values for runup and overtopping), followed by Topics 11 (review methods and models), 49 (WRUP™), 13 (overtopping volumes), and 14 (wavecast debris). Note that some of the workshop-assigned priorities and topic details were revised during the focused study.

Wave Runup and Overtopping Topics and Priorities

Topic Number	Topic	Topic Description	Priority		
			Atlantic / Gulf Coast	Pacific Coast	Non-Open Coast
11	Methods and Models	Review runup programs and methods; provide explicit guidance on where each should be applied	H *(I)*	A *(C)*	A *(C)*
12	Mean v. Higher Value	Review appropriateness of using mean vs. higher values for runup and overtopping	H *(C)*	C	C
13	Overtopping Volumes	Develop improved guidance for determining and mapping overtopping volumes	-- *(A)*	A	A
14	Wavecast Debris	Review available methods and develop guidance for wavecast debris	H	I	I
49	WRUP	Review WRUP™ (available wave runup program)	A	A	A

Key: C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)

1.2 WAVE RUNUP AND OVERTOPPING FOCUSED STUDY GROUP

This report was prepared using information and comments submitted by Ida Brøker (Danish Hydraulics Institute), Kevin Coulton (HDR), Jeff Gangai (Dewberry & Davis), Darryl Hatheway (Baker), Chris Jones (focused study leader), Jeremy Lowe (Phillip Williams & Associates), Ron Noble (Noble Engineering Consultants, Inc.), and Rajesh Srinivas (Taylor Engineering).

1.3 CURRENT FEMA GUIDANCE FOR WAVE RUNUP AND OVERTOPPING

1.3.1 Introduction

FEMA's existing guidance for runup and overtopping is limited to the coasts of the Atlantic Ocean, Gulf of Mexico, and Great Lakes*, as summarized in Appendix D of the *Guidelines and Specifications for Flood Hazard Mapping Partners* (FEMA, 2003). Although it is not stated explicitly, the inference is that existing Atlantic/Gulf and Great Lakes guidance will be appropriate for associated sheltered shorelines, given the proper selection of base flood water levels and wave conditions. There is no runup and overtopping guidance for the Pacific Coast in Appendix D.

Figures D-1 (page D-18) and D-35 (page D-113) of the *Guidelines and Specifications* (*G&S*) illustrate the overall procedures to be used for Atlantic/Gulf and Great Lakes flood insurance studies. In both cases, runup analyses must be preceded by the definition of a shore profile (transect). This shore profile must evaluate the durability (during the base flood) of any coastal structures present, and assess base flood erosion along any erodible shorelines. Runup estimates must be made along transects that have been adjusted for event-based erosion (not long-term erosion) and for any expected failures of coastal structures. Although it is not mentioned in the *G&S*, Study Contractors should check for possible breaches and failures between transects before interpolating runup and overtopping results to adjacent beaches.

FEMA calls for runup (and therefore, overtopping) analyses only in certain instances, as shown in Appendix D, Tables D-1 (Atlantic/Gulf) and D-14 (Great Lakes). These tables are summarized in Table 1 below.

FEMA presumes that runup on low-profile beaches—without a sizable landward barrier (e.g., dune, bluff, cliff, or structure)—will not be significant, and therefore need not be analyzed or calculated. This presumption is reasonable on low-profile shorelines where storm surges flood upland areas and wave heights tend to control base flood elevations (BFEs). This presumption, however, is probably invalid for the Pacific Coast, where storm surge heights tend to be small, swell periods can be large, infragravity motions can be substantial, and wave runup on beaches and structures tends to control BFEs.

---

* Note that FEMA's Great Lakes runup methods are based on the USACE Detroit District procedures (USACE, 1989).

Table 1. Shore Types where Runup Estimates are Required for Flood Insurance Studies (Atlantic/Gulf Coasts and Great Lakes)

Shore Type	Runup Analysis
Rocky bluff	yes
Sandy/sediment bluff or bank, little beach	yes
Sandy beach, small dune	no
Sandy beach, large dune	yes
Open wetlands	no
Shore protection structure	yes

Source: FEMA, 2003

1.3.2 Wave Runup

Runup guidance for the Atlantic Ocean and Gulf of Mexico is contained on pages D-42 through D-60 of FEMA (2003). FEMA calls for the use of its RUNUP 2.0 model, except for vertical- or near-vertical-faced coastal structures; on such structures, FEMA (2003) calls for use of procedures contained in the *Shore Protection Manual* (USACE, 1984). Although it is not stated in the *G&S*, FEMA also permits use of the Automated Coastal Engineering System (ACES) (USACE, 1992) for runup and overtopping calculations against vertical and sloping structures. (Note that ACES v. 1.07 is on the FEMA list of accepted models of coastal wave effects, which can be found at <http://www.fema.gov/fhm/en_coast.shtm>). It should also be noted that ACES uses more up-to-date methods than those contained in the *Shore Protection Manual* or those used in RUNUP 2.0

RUNUP 2.0 is a 1990 update and revision to FEMA's first runup model (RUNUP 1.0), which was originally developed for use in New England flood insurance studies in 1981. RUNUP 2.0 is discussed in Hallermeier, et al. (1990) and documented in Dewberry & Davis (1991).

RUNUP 2.0 is based largely on the reanalysis by Stoa (1978) of small-scale laboratory runup tests (regular waves on smooth, impermeable, uniform slopes); on the composite slope procedure developed by Saville (1958); and on roughness coefficients taken from the *Shore Protection Manual* (USACE, 1984). However, RUNUP 2.0 results were compared against field and large-scale laboratory runup measurements (using irregular waves), and Hallermeier et al. (1990) determined that the model predictions were in agreement with the measurements. Although not stated explicitly in the *G&S*, input wave conditions for RUNUP 2.0 will likely be irregular waves (specified as the equivalent deepwater mean wave height and period).

RUNUP 2.0 calculates wave runup along shore-perpendicular transects. It uses the 1% (100-year) stillwater elevation (tide plus surge, not including wave setup) and the equivalent deepwater *mean* wave conditions (height and period) as model inputs. It then estimates the *mean* wave runup height, which is added to the 1% stillwater elevation to determine the *mean* wave runup elevation. FEMA (2003) recommends using ranges of input wave heights and periods as inputs (+/- 5% or whatever percentage suits the level of uncertainty) in cases where it is difficult to specify the 1% flood conditions. The *G&S* call for averaging the RUNUP 2.0 output values for the nine input combinations of water level, wave height, and wave period.

One key difference between RUNUP 2.0 and RUNUP 1.0 is the fact that the latter predicted wave runup using unspecified combinations of offshore wave heights and periods (i.e., neither mean [50%], nor significant [33%], nor controlling [1%]) that were expected to occur during northeasters (or hurricanes). It was assumed by RUNUP 1.0 that the results (when added to the 1% stillwater elevation) represented the *maximum* runup elevation (Stone & Webster, 1981), while RUNUP 2.0 computes the *mean* runup elevation. Thus, there is a significant disparity between the results of flood insurance studies in communities based on RUNUP 1.0 and 2.0 models (Hatheway, pers. comm., 2003). This can be seen in New England, where many flood studies were based on the RUNUP 1.0 model.

Finally, unlike the case of wave height analyses using WHAFIS, FEMA (2003) states that wave setup is not to be added to the 100-year stillwater elevation before wave runup analyses, because RUNUP 2.0 assumes that wave setup is already included in the calculated wave runup. This assumption may be reasonable if the measurements and model tests used to develop the procedures contained in RUNUP 2.0 included wave setup effects (these data should be reviewed). However, the validity of this assumption should be reexamined for the Pacific Coast subject to infragravity waves, and as FEMA’s wave setup calculation methods evolve.

1.3.3 Wave Overtopping

Overtopping guidance for the Atlantic Ocean and the Gulf of Mexico is contained on pages D-61 through D-69 of FEMA (2003), and is based largely on the work of Owen (1980) and Goda (1985).

FEMA (2003) does not call for overtopping calculations in all instances. Instead it first calls for a comparison of the freeboard, F (the vertical distance between the base flood stillwater elevation and the crest elevation), and the mean runup height, $\overline{R}$. If F > 2$\overline{R}$, then the guidance assumes that overtopping can be neglected. If F ≤ 2$\overline{R}$, then the mean overtopping rate $\overline{Q}$ for a nonvertical slope is calculated according to:

$$\overline{Q} = Q^*(gH_S^3)^{0.5} \tag{1}$$

$$Q^* = 8 \bullet 10^{-5}\ \exp[3.1\ \left(rR^* - F/H_S\right)] \tag{2}$$

$$R^* = [1.5m/(H_s/L_{op})^{0.5}] \tag{3}$$

where:

Q^* = dimensionless overtopping,

R^* = estimated extreme runup normalized by Hs (note: the *G&S* do not define “extreme” runup),

r = the roughness coefficient,

F = freeboard,

H_s = incident significant wave height at toe of overtopped barrier,

g = gravitational constant,

m = the cotangent of the slope angle of the overtopped barrier, and

L_{op} = deepwater wavelength.

FEMA (2003) also includes guidance (Figure D-19) that can be used to estimate the dimensionless overtopping on smooth slopes (see Figure 2), from which $\overline{Q}$ can be calculated (adjustments for roughness can be made according to the text).

Overtopping of a vertical wall is calculated using the methods of Goda (1985) and summarized in *G&S* Figure D-20 (page D-68).

Table 2 (Table D-7 on page D-69, repeated below) relates flood hazard zones landward of an overtopped structure/feature to the mean overtopping rate.

Table 2. Interpretation of Mean Wave Overtopping Rates

$\overline{Q}$ Order of Magnitude	Flood Hazard Zone Behind Barrier
<0.9991 cfs/ft	Zone X
0.0001-0.01 cfs/ft	Zone AO (1 ft depth)
0.01-0.1 cfs/ft	Zone AO (2ft depth)
0.1-10. cfs/ft	Zone AO (3ft depth)
>1.0 cfs/ft*	30-ft width** of Zone VE (elevation 3 ft above barrier crest), landward Zone AO (3 ft depth)

*With estimated $\overline{Q}$ 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 structure width, incident wave period or wavelength, and other factors.

Source: FEMA, 2003

Note that one hazard zone associated with overtopping and rapid sheet flow—the VO zone—has been designated in the National Flood Insurance Program (NFIP) regulations, but is not contained in Table 2 and has not been implemented. The Hazard Zone Focused Study may recommend use of the VO zone; if so, procedures governing its use should be coordinated with the Runup/Overtopping Study Group.

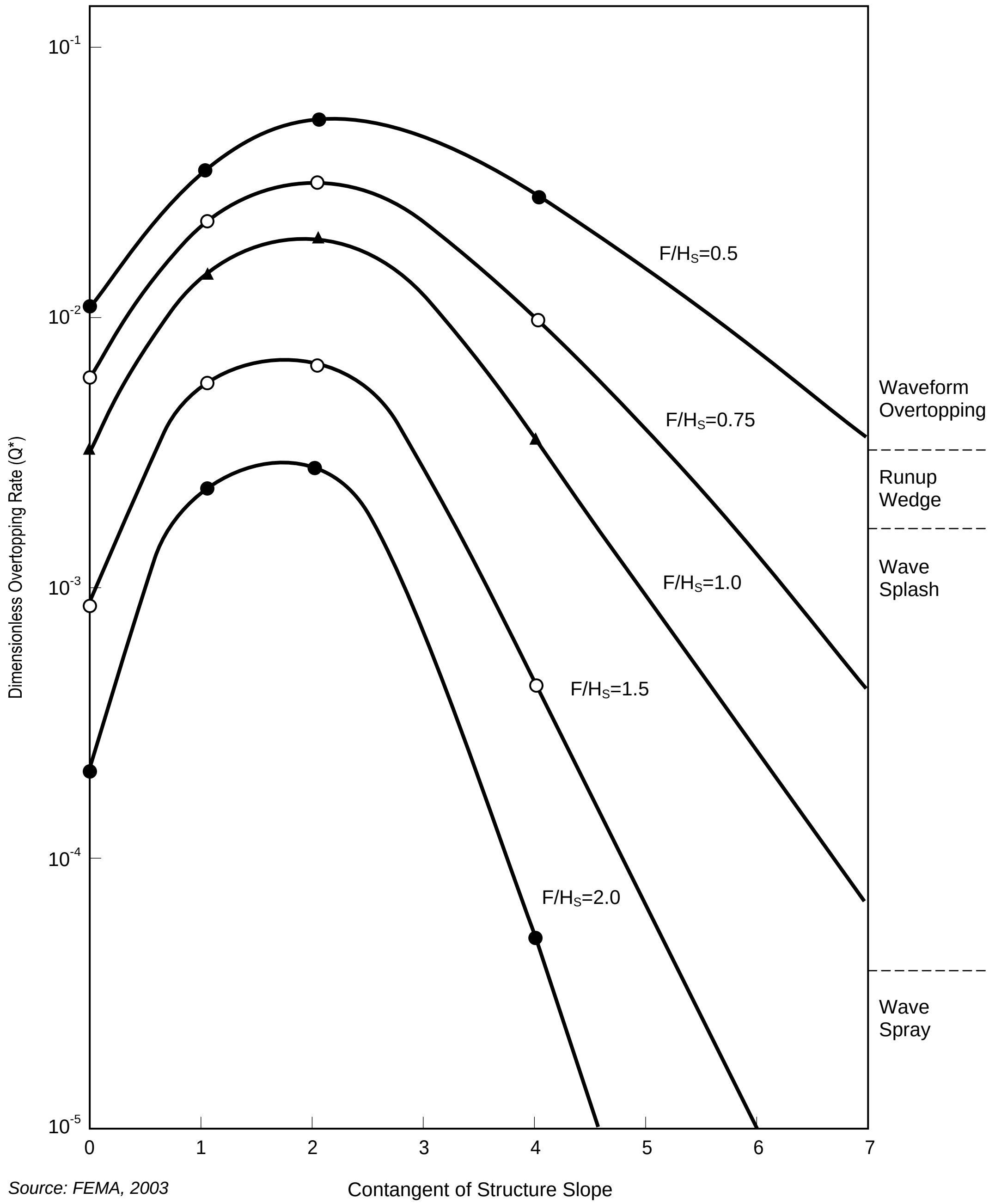


Figure 2. Overtopping of smooth, sloping structures.

FEMA (2003) provides simplified guidance for mapping flood hazard zones on overtopped dunes/barriers without calculating overtopping values (see Figure 3), and provides some guidance for runup onto low bluffs and plateaus, based largely on the work of Cox and Machemehl (1986)—see Figure 4. These procedures should be reviewed based on recent experience and other more recent methods.

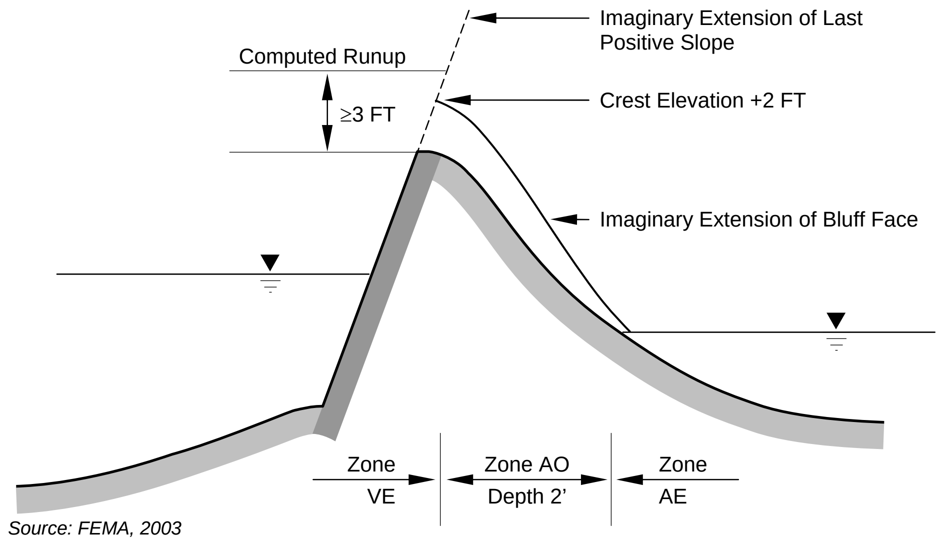


Figure 3. Simplified mapping of overtopped dune where runup exceeds crest by 3 feet or more.

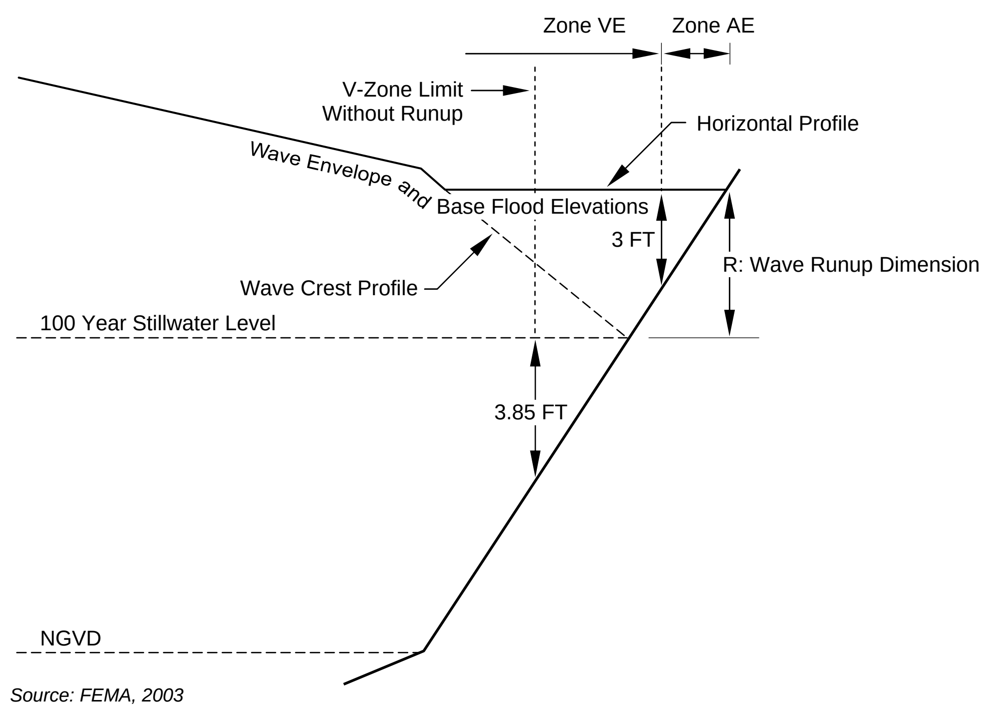


Figure 4. Wave envelope and base flood elevations resulting from combination of wave heights and wave runup.

2 CRITICAL TOPICS

2.1 TOPIC 12: REVIEW APPROPRIATENESS OF USING MEAN VS. HIGHER VALUES FOR RUNUP AND OVERTOPPING

2.1.1 Description of the Topic and Suggested Improvement

This topic can be summarized by asking three questions:

- Is calculating the mean runup elevation consistent with other FEMA guidance and procedures?
- Does mapping to the mean runup elevation provide adequate protection for building's which are in compliance with NFIP requirements?
- Does mapping to the mean overtopping rate provide adequate protection for NFIP-compliant buildings?

The conclusion of the Focused Study Group is that the answer to the first two questions is no, and the study group recommended that consideration be given to calculating and mapping to a higher runup level (the exact level is yet to be determined).

The answer to the third question is closely tied to how the overtopping rate is used to identify hazard zones. Use of the mean overtopping rate may be acceptable for calculation purposes, but the hazard zone delineations based on the mean overtopping rate may need to be revised.

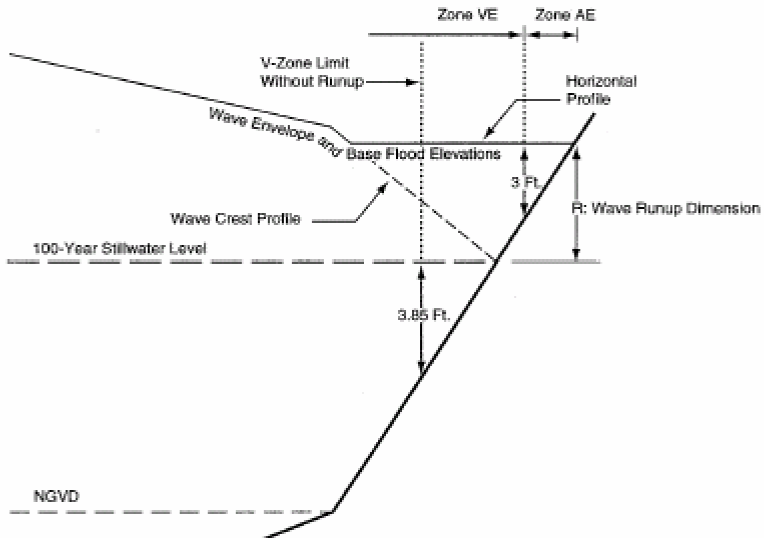
2.1.2 Description of Procedures in the Existing Guidelines

Current FEMA guidance calls for calculating (and mapping based on) the mean runup elevation and the mean overtopping rate.

Although there may be some exceptions, the average of the RUNUP 2.0 computed mean runup elevations is used to establish the BFE and flood hazard zones on the slope/structure subject to runup. The crest elevation and mean overtopping rate are used to establish the BFE and flood hazard zone landward of the overtopped structure/feature.

In areas not dominated by storm surge and wave heights, or by primary frontal dune considerations (see Hazard Zone Topics 17 and 39), FEMA differentiates between V zones and A zones based on the wave runup depth and the overtopping rate, as follows:

Areas on slopes subject to runup, where the ground is lower than 3.0 feet below the mean runup elevation (i.e., where the runup "depth" is greater than or equal to 3.0 feet), are classified as V zones. Where runup "depths" are less than 3.0 feet the areas are classified as A zones. Note the similarity to V zones based on wave heights (V zones have runup depth $\geq$ 3.0 feet or breaking wave heights $\geq$ 3.0 feet). See Figure 5.



Source: FEMA, 2003

Figure 5. Wave envelope and base flood elevations resulting from combination of wave heights and wave runup.

Landward areas subject to mean overtopping rates $\geq$ 1.0 cubic foot per second (cfs)/foot are mapped as V zones (see Table 1 above); otherwise, they are mapped as AO zones.

2.1.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

There are three key implications associated with application of the existing guidance. These implications are described below.

Consistency with Other FEMA Procedures*

FEMA typically—but with an important exception—maps hazards associated with the 100-year event at the mean (50%) level. Review of the *G&S* shows that the mean runup elevation, mean overtopping rate, and median erosion value are all used in mapping the 1% flood elevations in coastal areas. However, for Atlantic/Gulf of Mexico situations, FEMA uses its WHAFIS model to establish BFEs using the "controlling" (1%) wave height, not the mean wave height. The controlling wave height is equivalent to approximately 1.6 times the significant wave height (or approximately 2.6 times the mean wave height) in deepwater, but all

* Another inconsistency can be found with the incident wave conditions used as model inputs for RUNUP 2.0 versus WHAFIS. Although the inconsistency may be correct technically, it can be confusing to those using the RUNUP 2.0 and WHAFIS models: RUNUP 2.0 requires input of the equivalent deepwater mean wave height and period (approximated as 0.65 times the equivalent deepwater significant wave height, and 0.85 times the peak wave period); WHAFIS requires input of the significant wave height and peak wave period at the start of the analysis transect (which WHAFIS converts to the controlling [1%] wave height, assumed to be 1.6 times the significant wave height).

reduce to the depth-limited wave height (0.78 times stillwater depth) in shallow water. The WHAFIS model calculates wave crest elevations based on the controlling wave height. This procedure can be traced to the National Academy of Sciences (1977).

Dewberry & Davis (1991) acknowledges this discrepancy (between mapping the controlling wave height and the mean runup height), but calls for use of the mean runup value because there are "limitations in assuming a Rayleigh probability distribution for runup elevations." In other words, use of the mean runup value avoids having to estimate what a maximum runup elevation might be, when there is uncertainty associated with the actual runup distribution. Uncertainty arguments aside, there can sometimes be an inconsistency between mapping wave heights to a 1% level and mapping wave runup to a 50% level. The significance of this inconsistency increases as the runup velocity increases, and will be most apparent for mapping tsunami runup. The inconsistency may also be important in Pacific regions where infragravity motions can be substantial.

Adequacy of Base Flood Elevations and Hazard Zones Identified using Mean Values

This issue should be viewed in light of the principal purposes of the NFIP—to map flood and flood-related hazards, and to establish minimum development regulations (principally those related to the design and construction of buildings) using those maps.

If one examines the history of NFIP coastal mapping, the original coastal BFE was simply the stillwater level, and wave effects were ignored. Insurance premiums for areas subject to wave heights were surcharged, and building standards for V zones were more restrictive than those in A zones, but BFEs ignored the presence of waves. The National Academy of Sciences recognized the problem, as did those who inspected new homes in coastal Alabama, built to the stillwater elevation but destroyed by Hurricane Frederic in 1979. It was after Hurricane Frederic that the NFIP produced *Wave Height Supplement* reports and modified BFEs to reflect the 1% wave crest elevation.

Ignoring runup elevations above the 50% level means that buildings elevated to the mean runup elevation may be reached many times (and likely damaged) by wave runup during a coastal storm event. Although the impact of wave runup of a certain depth is generally less than that contained in a breaking wave of similar height (and, therefore, building damage may be less), the omission seems similar in nature (if not in magnitude) to the early omission of wave heights by the NFIP. This argument is supported by a recent flood insurance study on the Pacific Coast at Sandy Point, in Whatcom County, Washington. This study determined that use of the mean runup calculation procedure could under-predict damage to upland structures caused by flooding and associated wavecast debris. The determination was based on observed flooding and damage during a 5% (20-year) flood event (Phillip Williams & Associates, 2002).

The design of coastal structures is not the main focus of the NFIP (although coastal structure design is considered in mapping flood hazards). However, the present project can be informed by guidance on the design of coastal structures. The durability and crest elevation of a coastal structure are usually dictated by the importance of the area being protected, and by the frequency and rate of overtopping deemed acceptable. Structural designs are typically based on wave heights greater than $H_{50\%}$, and crest elevations are usually set to prevent overtopping at runup elevations higher than the mean value. These practices indicate that protection at a level higher than 50% is common. Regarding overtopping, mean overtopping rates are generally used for coastal structure design purposes. This practice may underestimate flooding in some cases, however. For example, if the structure has a high crest elevation but is attacked by several large, unbroken waves over a short period of time, the mean overtopping rate may be low, but the overtopping associated with those few large waves may cause significant flooding behind the structure.

RUNUP 1.0 vs. RUNUP 2.0

In 1991, FEMA adopted RUNUP 2.0 and discontinued use of RUNUP 1.0. RUNUP 1.0 calculated maximum runup elevations for a variety of combinations of input wave heights and periods assumed to be representative of conditions for a northeaster (or hurricane), not mean runup elevations. No systematic comparison of the results has been made for communities where Flood Insurance Rate Maps (FIRMs) are based on RUNUP 1.0. However, such a comparison might reveal substantially lower BFEs would result from use of RUNUP 2.0 mean runup elevations. Granted, some of the differences would be the result of other revisions made between versions 1.0 and 2.0, but the difference attributable to mapping a mean vs. maximum runup level could be significant. Further comparisons should be made for the northeastern Atlantic Coast to better define the difference between the results of runup models 1.0 and 2.0.

2.1.4 Alternatives for Improvement

Wave Runup

Several alternative runup values are considered for flood hazard mapping purposes:

- Maintain present FEMA use of $\overline{R}$,
- $R_{33\%}$ (significant runup, R_s),
- $R_{10\%}$,
- $R_{2\%}$, and
- R_{max} (maximum runup).

The selected value should account for the duration, frequency, and magnitude of runup elevations that may potentially damage upland structures. Use of FEMA's present $\overline{R}$ guidance seems to violate this criterion. However, the selected value need not be so conservative that it precludes all contact between runup and upland structures during the base flood event (use of the R_{max} value clearly violates this criterion), nor must it prevent contact by runup that has a low frequency of occurrence and/or a low likelihood of causing structural damage to upland structures (use of the $R_{2\%}$ value may violate this criterion).

Thus, use of a runup value in the range of $R_{33\%}$ to $R_{10\%}$ seems reasonable. Once a runup value is adopted, the next step is to define the $R_{x\%}$ height and elevation based on an existing runup calculation procedure that calculates $R_{x\%}$ directly (or uses a runup distribution relating $R_{x\%}$ to $\overline{R}$), or based on a more rigorous analysis (e.g., Monte Carlo). As a first approximation, and for the purposes of the present analysis, the $R_{33\%}$ and $R_{10\%}$ values would correspond to approximately $1.5\overline{R}$ and $2.0\overline{R}$, respectively. Incorporation of conversion factors such as these would allow the continued use of the RUNUP 2.0 model and methods in their present form, with only a scaling of the output runup height—an easy adjustment.

Wave Overtopping

As was the case with runup, several alternative overtopping values could be considered:

- Maintain present FEMA use of mean overtopping rate $\overline{Q}$,
- Q33% (significant overtopping rate, Qs),
- Q10%,
- Q2%, and
- Q_{max} (maximum overtopping rate).

However, overtopping calculations are subject to much more uncertainty than runup calculations, and selection of a specific $Q_{x\%}$ may be problematic. Kobayashi (1999) points out that while mathematical and numerical runup models may replicate measured runup values with errors of about 20%, predicted overtopping rates are often in error by a factor of 2 or more. Some overtopping predictions may be even less accurate, given the fact that subtle changes in wave conditions, water levels, barrier geometry and characteristics, or wave breaking can have a very large effect on overtopping rates. Unlike the case of wave runup, there appears to be no compelling reason to adopt an overtopping value different from $\overline{Q}$. It is recommended that FEMA continue to use the $\overline{Q}$ calculation, but reevaluate flood hazard zone designations based on mean overtopping rates (see Table 1 above and Section 3.2).

2.1.5 Recommendations

Recommendations for Topic 12 are as follows (see Table 5 at the conclusion of this report):

1. Revise the guidance to call for runup analyses in the sandy beach, small dune shore type (because runup will control BFEs on many low-profile beaches along the Pacific and sheltered shorelines).

2. Evaluate use of the mean runup $\overline{R}$ with a value; if $\overline{R}$ fails to capture historical evidence of damaging runup, then consider an alternate value for mapping purposes (probably in the range of $R_{33\%}$ to $R_{10\%}$, or as indicated by historical data).

3. Develop an interim procedure for adjusting the results of RUNUP 2.0 (for FIS or Letter of Map Revision [LOMR] evaluations).

4. Conduct a similar analysis specific to the tsunami runup value appropriate for flood hazard mapping.

5. Retain use of the mean overtopping rate $\overline{Q}$ for overtopping calculation purposes, but consider revising overtopping values that distinguish among flood hazard zones.

2.1.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

2.1.7 Related Available and Important Topics

Available and Important Topics related to Topic 12 are listed in Table 5, at the conclusion of this report.

2.2 TOPIC 11: REVIEW RUNUP METHODS AND PROGRAMS; PROVIDE EXPLICIT GUIDANCE ON WHERE EACH SHOULD BE APPLIED

Overtopping considerations have been removed from Topic 11 and grouped with those in Topic 13; although overtopping depends upon runup, it can be treated differently for NFIP flood hazard mapping purposes.

2.2.1 Description of the Topic and Suggested Improvement

Current FEMA runup guidance has been developed on an ad-hoc basis over the years. The guidance may or may not represent the procedure(s) most appropriate for a contemporary FIS. It may or may not be transferable to the Pacific Coast.

In fact, experience suggests that this guidance may not be directly transferable without some revision or modification. The Pacific Coast, unlike the open-coast Atlantic and Gulf of Mexico, does not lend itself to a simple characterization of the 1% flood event. Much of the Pacific Coast is composed of dissipative beaches, and the relative contributions of storm surge, wave setup, and wave runup can differ substantially from those along the coasts of the Atlantic Ocean and Gulf of Mexico. Pacific wave spectra may differ substantially from those used to develop the FEMA runup methods used along the coasts of the Atlantic Ocean and Gulf of Mexico.

This is not to say that wave runup has not been computed for the Pacific Coast. It has been computed using a variety of available methods: the FEMA RUNUP 2.0 model, ACES, *Shore Protection Manual* SPM (1984) methods, tsunami runup models, and other methods, some of which are based on local experience.

The issue is not whether runup methods are available; the issue is which of the available methods are best suited to FISs and yield the best results for the Pacific Coast. Therefore, the Focused Study Group has chosen to revise the Topic 11 priorities assigned at Workshop 1 from “Available” to “Critical” for the Pacific, and from “Helpful” to “Available” for the Atlantic and Gulf Coasts.

Clearly, the identification of appropriate runup guidance is most needed for Pacific FISs, and that issue is given the highest priority. Existing guidance for the Atlantic and Gulf can be used without major modification (notwithstanding the mean runup issue discussed in Topic 12), but the New England Coast especially will benefit from the development of guidance for the Pacific Coast.

The Focused Study Group for Topic 11 sought to facilitate the development of sound, practical runup guidance for the Pacific Coast, and to evaluate similar guidance for the coasts of the Atlantic Ocean and Gulf of Mexico. With this in mind, the study group’s primary recommendation is to develop test scenarios and perform side-by-side comparisons of existing runup methods and models. The testing should include evaluation of the sensitivity of the various runup methods and models to various parameters (e.g., profile shape and roughness, incident wave characteristics, infragravity motions). Infragravity motions must be included in any Pacific Coast testing; infragravity waves are more common on the Pacific Coast than on the Atlantic and Gulf Coasts, and such waves can amplify runup and overtopping considerably.

A similar approach may be useful for evaluating Pacific Coast event-based erosion or wave setup and wave transformation. As many categories as possible should be evaluated using common test conditions.

2.2.2 Description of Procedures in the Existing Guidelines

See Sections 1.2 and 2.1.2.

2.2.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

See Section 2.1.3.

2.2.4 Alternatives for Improvement

At least a dozen methods and models can be used to predict wave runup, not counting site-specific field measurements and laboratory modeling (both of which are unlikely during an FIS). Relevant issues and parameters associated with these methods and models are as follows:

- Each method or model is based on certain assumptions and empirical data, and each is valid over a range of morphologic, hydraulic, and sometimes geographic conditions.
- Some use deepwater wave conditions as input; others use local (i.e., transformed) wave conditions at the toe of the barrier.
- Some methods or models are applicable to beaches and others to coastal structures.
- Some are applicable to transect-type analyses while others are appropriate to grid- or element-based analyses.
- Each requires tradeoffs among simplicity, accuracy, data requirements, ease of use, and economy.

Wave Runup

The runup methods and models considered are described below.

RUNUP 2.0

This model was described in Section 1.2.1.

Shore Protection Manual

The Shore Protection Manual (SPM) (USACE, 1984) contains several graphs that relate the runup of normally incident regular (monochromatic) waves on impermeable slopes to deepwater wave steepness, barrier slope, and deepwater wave height. Refraction, diffraction, and bottom friction are not considered. Graphs are provided for smooth slopes, quarrystone and stepped revetments, and vertical and curved-face seawalls. These graphs are based on small-scale laboratory work; guidance is provided for adjustment of calculated runup for scale effects and roughness. Any effects of wave setup are included in the computed runup values. Saville's (1958) composite slope procedure is included.

The SPM gives limited guidance for estimating runup resulting from irregular waves. According to Dewberry & Davis (1991), the 1984 SPM did not make use of Stoa's (1978) reanalysis of wave runup data.

WRUP™

WRUP™ was developed by Noble Software, Inc., for the runup of regular waves (Noble, 1984). A menu-driven program designed to facilitate the calculation of wave runup based on SPM methods, WRUP™ uses equations, curves, and methodology presented in the 1984 edition of the SPM.

The program can be applied to composite slopes (up to eight variable slopes per profile) including revetted slopes, vertical slopes, and three defined complex structures. It can calculate runup that exceeds the top of a vertical wall or other steep slope by adding a fictitious flat slope directly behind the top of vertical or steep slopes. Wave input can be at deepwater, intermediate water, or depth-limited breaking waves. WRUP™ has been applied to the Coast of California Storm and Tidal Waves Study (CCSTWS) in Orange County for the U.S. Army Corps of Engineers (USACE). The advantage of using WRUP™ is that it is faster and more convenient than interpolating from graphs in the SPM. A flow chart for WRUP™ is shown in Figure 6.

Parabolic Profile Representation

Taylor et al. (1980) developed an alternate to the composite-slope approach by describing the beach profile between the seaward edge of the dune and the wave breakpoint by an equilibrium profile, a parabolic function of the form:

$$x = a\ y^{\upsilon} \tag{4}$$

The formulation does not include longshore bars. It uses small-scale laboratory data of Saville (1956, 1958), Savage (1958) and Hunt (1959) to relate runup to the deepwater wave height and period.

Limited comparisons with the profiles produced by the composite-slope method for Volusia County, Florida, show generally poor agreement, with the parabolic method producing generally lower runup. This was thought to have occurred partly because the parabolic approach smoothed the bar and resulted in seaward shifting of the wave breakpoint, which reduced the mean slope relative to the composite-slope method. It was not possible at the time of the study to determine which approach more accurately predicted runup.

ACES v. 1.07

The most widely used version of ACES is the freely distributed ACES v. 1.07 (USACE, 1992). Later versions are available only as part of the CEDAS (Coastal Engineering Design and Analysis System) software sold by Veritech.

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.

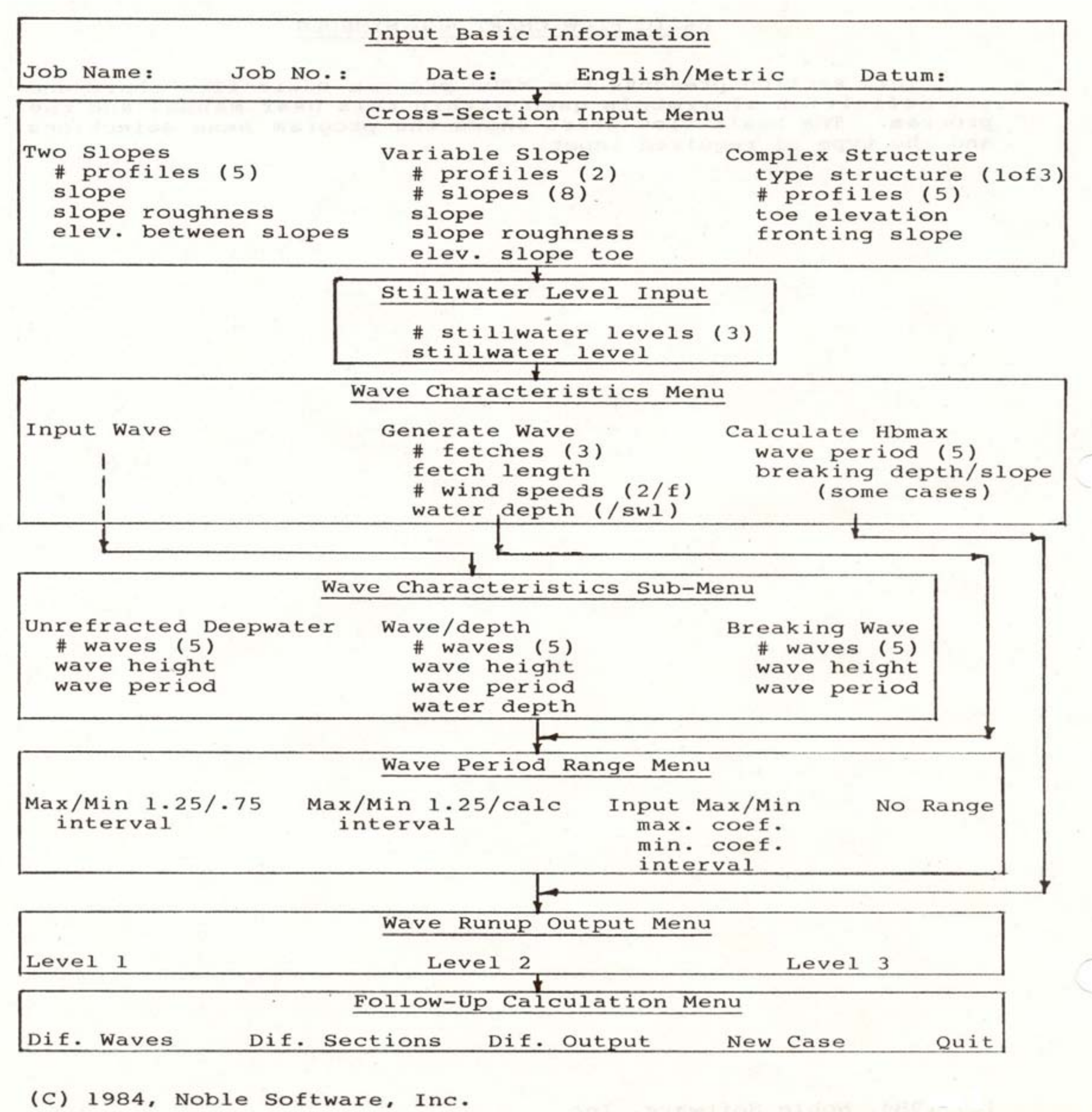


Figure 6. Flow chart for WRUP™.

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

$$\frac{R_{x\%}}{H_0} = a\xi^{\ b} \tag{5}$$

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

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

$$\frac{R_{\max}}{H_0} = a\,\xi\big/(1 + b\,\xi) \tag{6}$$

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

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

$$\frac{R}{H_i} = c\,\xi\big/(1 + d\,\xi) \qquad \text{for rough slopes} \tag{7}$$

$$\frac{R}{H_i} = C \qquad \text{for smooth slopes} \tag{8}$$

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

The ACES runup modules represent improved guidance over that contained in the SPM. ACES guidance may be preferable to RUNUP 2.0 in some instances. The *Irregular Wave Runup on Beaches* calculation is maintained in the Coastal Engineering Manual (CEM). The *Irregular*

Wave Runup on Riprap calculation is reported to be advantageous because it works well for both shallow water and deep water at the toe of the revetment.

Coastal Engineering Manual

A replacement for the *Shore Protection Manual*, the CEM (2003) (Section II-4-4) contains guidance for calculation of regular and irregular wave runup on beaches (Smith, 2003). Wave setup contributions are included in the runup results. Runup by regular breaking waves on smooth impermeable slopes is based on small-scale model tests and is a function of the deepwater wave conditions (expressed using the surf similarity parameter). Such runup is calculated using relationships developed by Hunt (1959), and rewritten in nondimensional form by Battjes (1974):

$$\frac{R}{H_0} = \xi_0 \qquad \text{for} \qquad 0.1 \le \xi_0 \le 2.3 \quad \text{with} \quad \xi_0 = \tan\beta\left(\frac{H_0}{L_0}\right)^{-\frac{1}{2}} \tag{9}$$

Walton et al. (1989) revised the formulation to determine the upper limit of runup by nonbreaking regular waves:

$$\frac{R}{H_0} = (2\pi)^{\frac{1}{2}}\left(\frac{\pi}{2\beta}\right)^{\frac{1}{4}} \tag{10}$$

where β = slope (in radians).

The guidance for runup from irregular breaking waves on smooth impermeable slopes is similar to the guidance contained in ACES 1.07 (see above). The CEM (2003) (Section VI-5-2) contains guidance for calculation of irregular wave runup on structures (Burcharth and Hughes, 2003). The guidance is based largely on the small- and large-scale laboratory tests summarized in van der Meer and Stam (1992), and van der Meer and Janssen (1995). It uses a Battjes-type formulation

$$\frac{R_{x\%}}{H_s} = (A\xi + C)\gamma_r\gamma_b\gamma_h\gamma_\beta \tag{11}$$

where A and C are coefficients related to the surf similarity parameter and runup probability for the reference case (smooth, straight impermeable slope, normally incident long-crested waves with wave heights given by a Rayleigh distribution); and where the coefficients γ_r , γ_b , γ_h , γ_β adjust for surface roughness, influence of a berm, shallow water, and angle of wave incidence (γ = 1.0 for reference case).

The CEM provides several graphs and formulas for $R_{2\%}$ and R_S as a function of the significant wave height at the toe of the structure, not as a function of the deepwater wave height. Also, note that $R_{2\%}$ refers to the runup level exceeded by 2% of the incoming waves, not by 2% of the runup levels, etc.

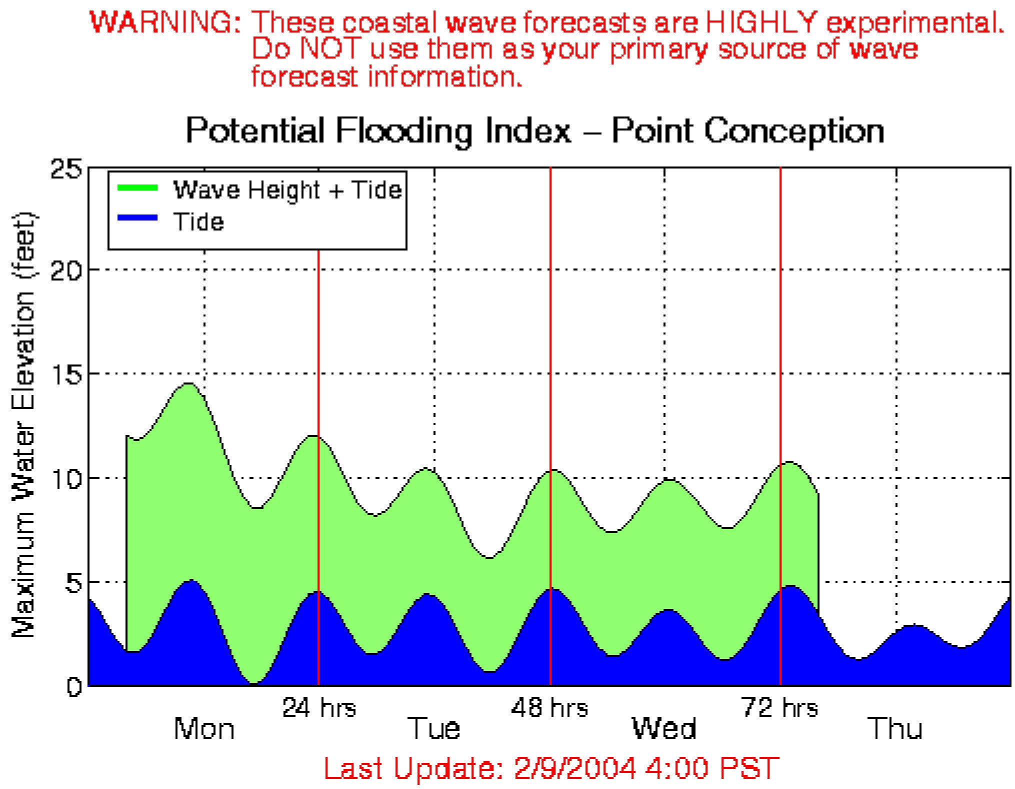
The CEM provides no methods for calculating irregular wave runup against vertical walls, although the method of Walton et al. (1989) mentioned above in the *Regular Wave Runup on Beaches* section could be used.

Wave Momentum Flux Parameter

Hughes (2003a, 2003b) developed and used a wave momentum flux parameter to improve on the predictive accuracy of the CEM's irregular wave runup guidance for smooth-sloped, impermeable structures. Like the CEM, this revised method calculates the $R_{2\%}$ value using inputs of local wave height and period, structure slope, and depth at structure toe.

Coastal Data Information Program (Potential-Flooding Index for Southern California)

The Coastal Data Information Program (CDIP) is an experimental tool used to forecast the maximum runup elevation based on predicted (astronomical) tide elevations and the predicted significant wave height outside the surf zone (Seymour, 2003). The experimental CDIP tool is illustrated in Figure 7.



Source: CDIP 2004

Figure 7. Coastal Data Information Program, potential flood index tool.

The CDIP is not a wave runup model per se; therefore, use of the CDIP Potential Flood Index Tool as a proxy for runup elevations should be considered an interim approach until runup analyses are completed. Actual forecasts can be found under *Wave Forecast Models* (see “Coast Waves + Tide, southern California”) at <http://cdip.ucsd.edu/elnino_htmls/homepage.shtml>. The Potential Flood Index Tool assumes that the combined setup plus runup at the shoreline is equal to the significant wave height beyond the surf zone. (The latter can be forecast using wave buoy data and numerical models.)

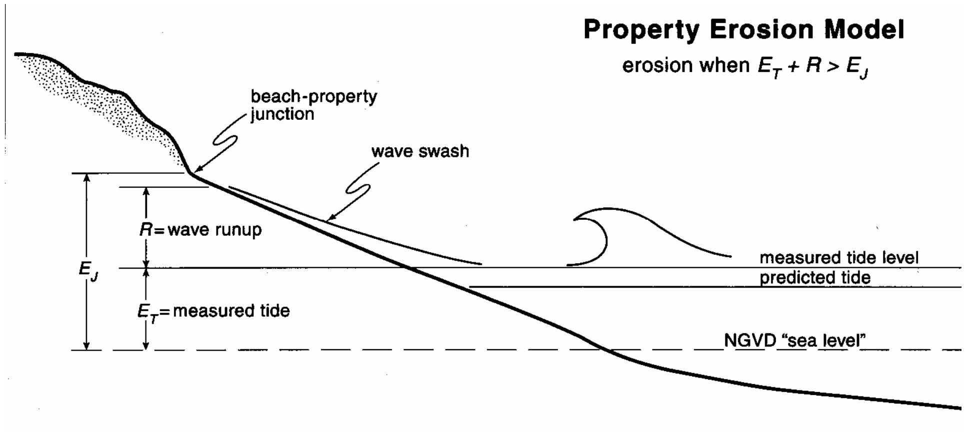
Oregon Property Erosion Model

Ruggiero et al. (2001) summarize development of a model to evaluate the susceptibility of coastal property to wave-induced erosion. The model is predicated on the observation that foredune erosion occurs when the runup elevation (actual tide elevation plus runup height) exceeds the elevation of the beach-foredune junction (see Figure 8). Wave setup is embedded in the runup.

The study points to the importance of both runup elevation and duration (hours/year) of high runup elevations. It found a good correlation between the number of hours per year that the predicted $R_{2\%}$ elevation would exceed the beach-foredune elevation, and observed erosion characteristics. Using field data from Oregon and North Carolina (USACE Field Research Facility, Duck, North Carolina), the predicted $R_{2\%}$ (2% exceedance elevation, measured in meters above National Geodetic Vertical Datum [NGVD]) was defined using beach slope, and deepwater significant wave height and wavelength as:

$$R_{2\%} = 0.27\,(S\,H_{os}\,L_o)^{1/2} \qquad \text{(metric units)} \qquad (12)$$

Where the shore was subject to less than 1 hour of attack per year (“attack” is defined as when $R_{2\%}$ exceeds the beach-foreshore junction), the shore tended to be stable or accretional. Where the shore was subject to more than 10 hours of attack per year, the shore was erosional. Higher durations were associated with greater erosion.



Source: Ruggerio et al., 2001

Figure 8. Oregon property erosion model.

Technical Advisory Committee for Water Retaining Structures

The TAW (2002) report updates the earlier guidance of van der Meer (upon which much of the CEM runup guidance is based). This report is available at <http://www.tawinfo.nl/engels/downloads/TRRunupOvertopping.pdf>. It includes the results of recent model tests, and considers cases with very shallow foreshores and with vertical walls atop slopes. The report also replaces use of the peak wave period at the structure toe with the spectral wave period, and increases estimates of maximum wave runup.

Boussinesq Wave Models

This type of model solves the so-called Boussinesq type equations in the time domain. It resolves the waves in detail, and is suited for simulation of propagation and interaction of nonlinear directional waves. It is capable of reproducing the combined effects of most wave phenomena of interest in ports, harbors, and coastal engineering: shoaling and refraction, diffraction, bottom dissipation, partial reflection and transmission, nonlinear wave-wave interactions, and wave breaking for directional, irregular waves.

DHI's suite of models, MIKE 21, includes two Boussinesq modules, 2DH and 1DH. The "2DH" module calculates wave disturbance in ports and harbors; the 1DH module calculates wave transformation across an arbitrary profile from offshore up to the shoreline for the study of surf zone and swash zone dynamics (see Figure 9). The 1DH module solves the equations along a transect, and can therefore represent the dynamics for unidirectional, irregular waves.

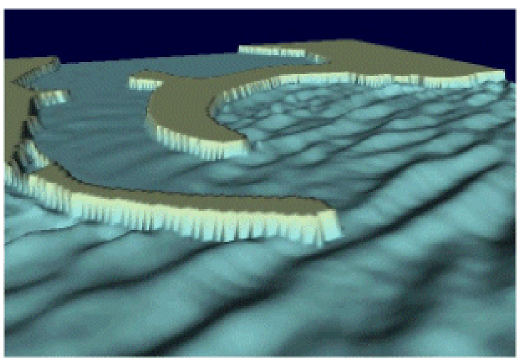
The 1-D BW model is a relevant tool for the study of runup, and its strength is its computational speed. The 1-D BW can simulate the combination of setup and runup, and phenomena such as wave groups and surf beat can be included (provided that the driving forces are included in the boundary conditions). The results can be analyzed into frequency of exceedance runup levels.

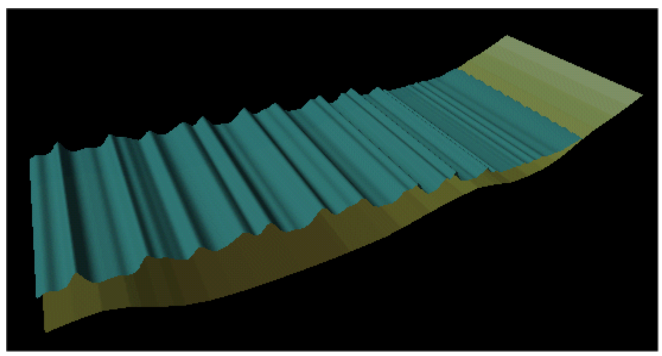
Detailed 3-D Hydrodynamic Model, Navier-Stokes Solvers

DHI's Navier-Stokes solver, NS3, is a numerical model that solves the full three-dimensional Navier-Stokes equations including modeling of the free surface. The model is designed especially for modeling of refined flow problems, such as eddies around structures, details of run-up on structures, etc.

The model can be run in full 3-D or can be used as a "slice model" representing, for instance, a coastal transect. Figure 10 shows an example where NS3 has been used along a transect analysis to calculate runup and overtopping of a solitary wave on a dike. The example shows a comparison between modeled and measured water levels on the crest and behind the dike.

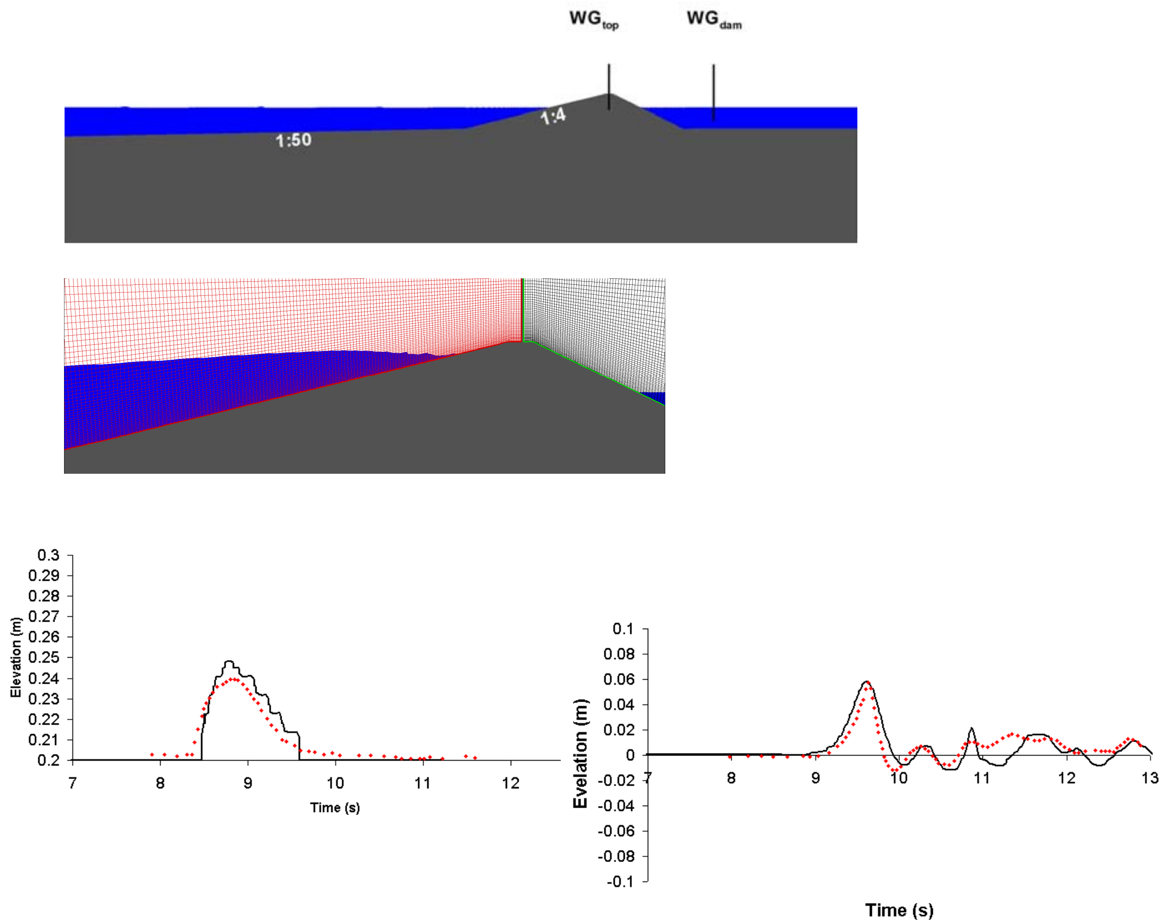
NS3 can be used as a numerical tool that replaces physical model tests in a flume. Output from the model is a time series of water levels, velocity fields, overtopping rates, and pressure fields. This model is also a useful tool for the calculation of forces on structures, e.g., wave forces on a wave screen. The Navier-Stokes solver is more accurate in the prediction of wave overtopping than the Boussinesq models, which are strong tools for wave runup calculations.





Source: Danish Hydraulics Institute

Figure 9. Illustration of the 2-D BW Model (wave penetration into a harbor) and the 1D BW Model (wave transformation across a beach profile).



Notes:
Upper panel, layout of experiment; middle panel, close-up of computational grid near the crest of the dike; lower left, comparison of measured water level at the crest (dots) and modeled level (line); lower right, measured water level behind the structure (dots) and modeled (line).

Source: Danish Hydraulics Institute

Figure 10. Runup and overtopping calculated by DHI NS3.

The numerical model is complex and computationally demanding. NS3 is presently not released as a commercial software product and runs presently without Graphical User Interfaces. However, conceptual model setups can be prepared so experienced modelers can adjust the boundary conditions and the geometry and can run specific simulations without detailed knowledge of the coding.

Deterministic vs. Statistical Approaches

Two general methods for computing 1% annual chance flood elevations were discussed in Workshop 2: the Event Selection Method and the Response Method.

- The Event Selection Method is deterministic; it uses one or more user-identified combinations (each defined as a 1% flood event) of water level and wave conditions, and computes the resulting flood elevation for each combination. The user then selects a flood elevation for mapping purposes.

The Response Method is based on a statistical approach, where input parameter values are selected (randomly) from defined parameter distributions, and are then used to compute a flood elevation (response). The process is repeated many times, a response distribution is developed, and the 1% response is determined.

Given the difficulties (particularly on the Pacific Coast and on sheltered shorelines) in defining the 1% flood event, including all relevant parameters—water level, transformed wave conditions, wave setup, erosion, and runup—it may be useful to consider a statistical type analysis for determining the $R_{x\%}$ elevation used for flood hazard mapping. A statistical (response) approach can account for the random combination of storm wave conditions, tide elevations, and other parameters, and can determine a statistical distribution of wave runup frequency and wave runup elevations.

The statistical approach requires distributions and constraints for input parameters to be defined. It allows determination of the wave-tide combination(s) responsible for the $R_{x\%}$ elevation. The statistical approach is not limited to a single runup calculation procedure (it can be employed with many different procedures), but can provide statistical meaning to the results from the runup calculation procedure employed. A flow chart for one statistical approach is shown in Figure 11.

Using Models vs. Using Simple Procedures

The main advantage of numerical runup (and overtopping) models over simple procedures (empirical formulas) is that with models, arbitrary profile shapes can be studied in combination with widely varying water level and wave parameters. The utility of simple formulas is restricted by the empirical data and conditions that led to their development, and extrapolation to other geometries and conditions may be questionable.

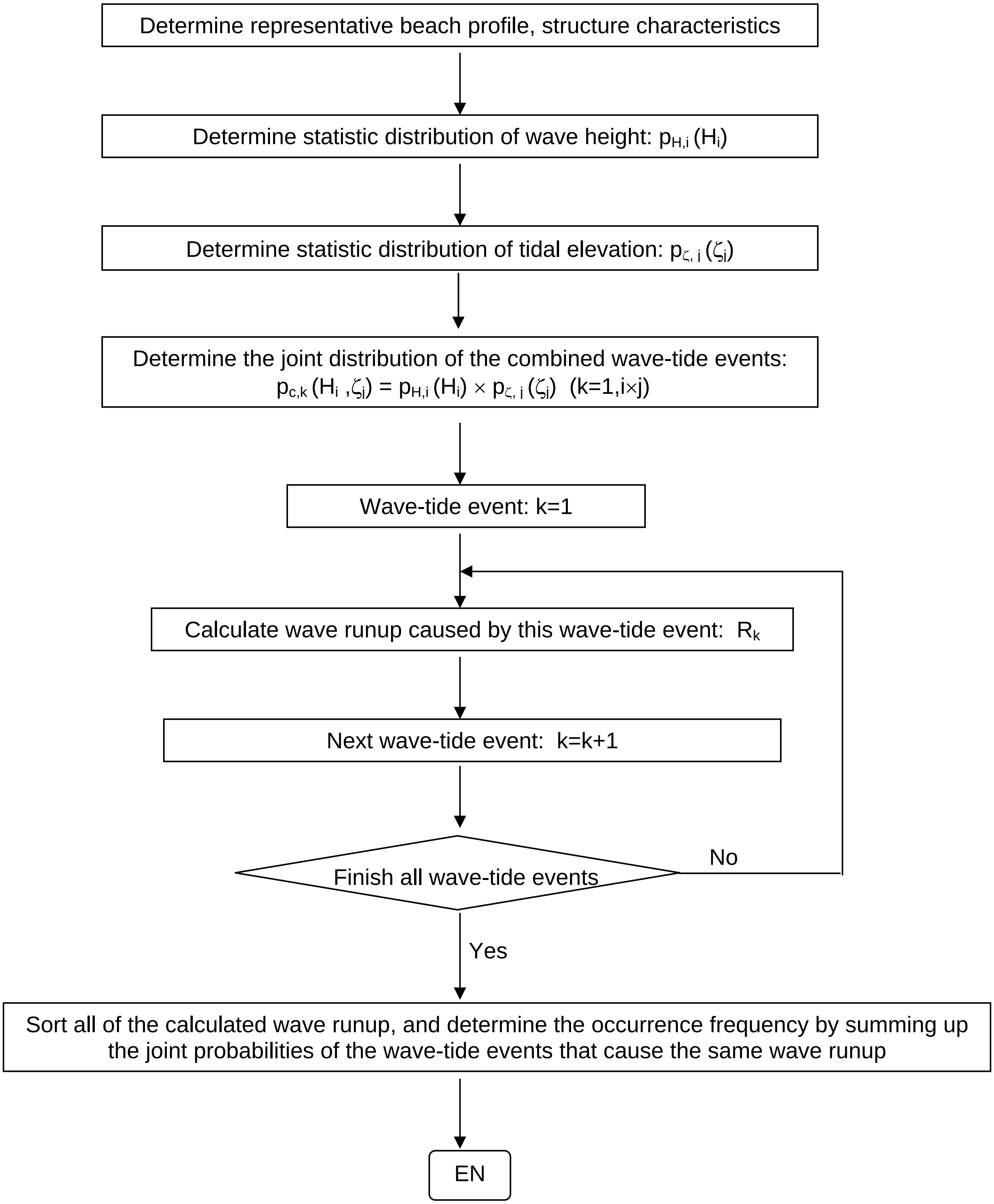


Figure 11. Flow chart for statistical approach.

However, if a shoreline/structure profile under investigation has geometric characteristics and hydraulic conditions similar to those that form the basis for a simple procedure, use of the simple procedure will be acceptable, and will probably be more cost effective for FEMA. Numerical models may be better suited to complex shoreline shapes, geometries, and situations (and less restricted by ranges of conditions over which they are applicable), but they also require more data, preparation, expertise and expense to yield acceptable results.

Numerical models, computing capabilities, and data acquisition/manipulation techniques (including Light Detection and Ranging [LIDAR] and Geographic Information Systems [GIS]) have advanced significantly over the past two decades. During that time, however, FEMA’s basic approach to identifying coastal flood hazards has remained unchanged. (Improvements have been made to various FEMA methods, but the basic transect analysis process has remained intact.) Model development has been driven, in large part, by the need for improved coastal structure design capabilities, and for shoreline management purposes. Flood hazard mapping can benefit from these advancements.

Ultimately, FEMA’s methods will be overtaken and replaced by numerical models. This is likely to occur first for large study areas where coastal storm surges (including wave transformation, wave setup, and other wave effects) must be recomputed, and last for situations where previously computed storm surges and related parameters are judged adequate for FEMA use. This evolution should also occur first where critical infrastructure and development exist, and where the uncertainty associated with use of the simple formulas may not be acceptable. Note that FIS and FIRM appeals may hasten this evolution, through the use of more advanced models by appellant representatives.

In the interim, runup (and overtopping) calculations can be carried out by a variety of methods (which may include numerical models), but carefully chosen and applied simple procedures should be adequate for most coastal FISs and restudies.

The Runup/Overtopping Study Group recommends that the procedures and models described above be evaluated carefully, with an eye toward improving the accuracy of flood hazard maps using simple procedures (where possible), and eventually migrating to numerical models for most flood hazard mapping tasks.

Wave Runup, Wave Setup, and Wave Transformation

Wave runup is typically estimated using the stillwater elevation (without wave setup) as an input, and runup estimates generally include the combined effects of swash and wave setup. This has been the tendency because the majority of field and laboratory runup measurements to date—upon which most estimation procedures are based—have made no attempt to separate out the exact effects of wave setup. Relying on wave inputs is likewise a function of the evolution of empirical runup methods; some rely on deepwater wave conditions while others rely on the local waves at the structure toe.

As models advance, the capacity to resolve water level constituents, wave transformation, and complex hydraulic interactions will increase. It is important to take advantage of these capabilities where they serve flood mapping needs, but the need should drive the technique (not the other way around).

Irrespective of the exact path, as FEMA's coastal flood hazard mapping methods change, the treatment of wave setup and wave runup (and other components, e.g., stillwater elevations, event-based erosion, overland wave propagation) must be consistent. Thus, the Runup/Overtopping Study Group sees the need for close coordination with other Focused Study Groups, particularly the Wave Setup and Wave Transformation groups.

2.2.5 Recommendations

Recommendations for Topic 11 are as follows (see Table 5, at the conclusion of this report):

Investigate use of Oregon-type and/or CDIP-type methods as interim methods for all of California, Oregon, and Washington. While not probability-based at present, it is reasonable to expect that probabilities could be assigned and a base flood runup elevation could be estimated using these methods. Bear in mind the previously mentioned caution, that the CDIP does not resolve the surf zone and compute wave runup—its Potential Flood Index Tool is an experimental proxy for runup.

Develop test scenarios for side-by-side comparisons of existing runup methods and models (give priority to the Pacific Coast, followed by New England, then the south Atlantic and Gulf of Mexico). This will require selecting representative beach profiles and structure geometries—including low-profile, sandy-beach, small-dune barriers not presently modeled for runup (see Table 1)—then locating existing data sets that can be used as a basis for comparing the accuracy and sensitivity of results. These data sets may also serve as historical data of potential use in future FISs. (Coordinate development of test scenarios with other study groups.)

Perform the side-by-side comparisons. Eliminate methods or models that do not provide acceptable results or that cannot be used efficiently. (Remember that these will have to be used for FISs with time, budget, and expertise constraints.) Identify which methods and models are appropriate for use in various geographic areas and morphologic/hydraulic conditions. Consider appropriate ranges of input parameters to address event definition uncertainty.

Coordinate work with the Wave Setup and Wave Transformation Study Groups. Inputs to wave runup methods/models must be available and consistent with the results of wave setup and transformation tasks.

2.2.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

2.2.7 Related Available and Important Topics

Available and Important Topics related to Topic 11 are listed in Table 6 at the conclusion of this report.

3 AVAILABLE TOPICS

3.1 TOPIC 49: REVIEW WRUP^{TM} (AVAILABLE WAVE RUNUP PROGRAM)

3.1.1 Description of the Topic and Suggested Improvement

See Section 2.2.1.

3.1.2 Description of Procedures in the Existing Guidelines

See “Wave Runup” in Section 2.2.4.

3.1.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

FEMA *G&S* are predicated on SPM calculations for many items, including wave runup on vertical walls. WRUP^{TM} is a program built around SPM methods, and therefore it should satisfy current flood hazard calculation requirements. However, the model has not been accepted by FEMA per se, and its widespread use would not be permitted. (The developer is free to use the model and submit its results for specific projects; this is one issue that has not been clarified by FEMA.) Formal acceptance and widespread use of WRUP^{TM} should be predicated upon: 1) the continued use of SPM methods by FEMA, and 2) a detailed technical review of WRUP^{TM} for consistency with the SPM.

3.1.4 Alternatives for Improvement

See “Wave Runup” and “Deterministic vs. Statistical Approaches” in Section 2.2.4.

3.1.5 Recommendations

The recommendation for Topic 49 is to include the evaluation of WRUP^{TM} in the Topic 11 evaluation of runup methods and models.

3.1.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

3.2 TOPIC 13: DEVELOP IMPROVED GUIDANCE FOR DETERMINING AND MAPPING OVERTOPPING VOLUMES

3.2.1 Description of the Topic and Suggested Improvement

Current FEMA overtopping guidance has been developed on an ad-hoc basis over the years. The guidance may or may not represent the procedure(s) most appropriate for contemporary FISs.

There are a variety of overtopping methods and procedures that should be evaluated as part of this topic. The focus of the work should be on the following steps:

Review available overtopping methods and models, and determine appropriate procedure(s) for calculating the mean overtopping discharge, including those over low-profile beaches and barriers, dune remnants, revetments, and vertical walls.

Evaluate FEMA's current guidance, which limits the runup elevation to 3 feet above a barrier's crest elevation

Evaluate procedures for calculating overtopping onto low bluffs with gently sloping, flat, or adverse slopes. Evaluate methods for determining ponding landward of overtopped barriers

Review the current literature on "acceptable" overtopping, and work with the Hazard Zone Study Group to evaluate the overtopping rates FEMA (2003) uses to identify flood hazard zones landward of an overtopped barrier.

3.2.2 Description of Procedures in the Existing Guidelines

See Section 1.2.3.

3.2.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

See "Wave Overtopping" in Section 2.1.4.

3.2.4 Alternatives for Improvement

Calculating Wave Overtopping

The overtopping methods and models to be considered are described below.

FEMA Guidelines and Specifications Method
See Section 1.2.3.

Shore Protection Manual
For regular waves, an empirical expression is used based on a reanalysis of laboratory data reported by Saville (1955) and by Saville and Caldwell (1953):

$$q = \sqrt{gQ_0^* H_0^3}\exp\left[-\frac{0.217}{\alpha}\tanh^{-1}\left(\frac{h-d_s}{R}\right)\right] \quad \text{(Equation 7-10 in the SPM)} \qquad (13)$$

where α and Q^*_0 are empirical coefficients given in SPM Figures 7-24 to 7-32, based on experiments for various wave conditions, structure slopes and structure types. Weggel (1976) provided guidance on determining approximate values of α and Q^*_0 when better estimates are not available. Inputs are deepwater wave height, runup, height of structure, depth of water at the structure, and various coefficients. A procedure is included in the SPM to estimate the increase in overtopping rate with wind speed (Equation 7-12).

Ahrens (1977) extended the formula for regular waves by applying a method for determining runup for irregular waves. This procedure was included in the SPM as an interim procedure.

$$q_{p\%} = \sqrt{gQ_0^* H_{0,s}^3}\exp\left[-\frac{0.217}{\alpha}\tanh^{-1}\left(\frac{h-d_s}{R_s}\right)\frac{R_s}{R_{p\%}}\right] \quad \text{(Equation 7-14 in the SPM)} \qquad (14)$$

ACES v. 1.07

Wave overtopping is provided in ACES for both monochromatic waves and irregular waves. For monochromatic wave overtopping, ACES uses the SPM method developed by Weggel (1976). For irregular wave overtopping, ACES uses a method based on Ahrens (1977) and Douglass (1986), which uses Weggel's monochromatic formula, but uses the significant deepwater wave height. The method computes and sums overtopping contributions of the individual members of the runup distribution.

Cox and Machemehl (low bluff)

See Section 1.2.2.

Coastal Engineering Manual

The CEM presents a variety of wave overtopping formulas from many different sources (see Table 3). Each source presents wave overtopping for a different structure configuration or scenario and is based mostly on empirical formulas from laboratory testing. Two types of overtopping formulations dominate the literature:

$$Q = a\,e^{-(bR)} \qquad (15)$$

$$Q = a\,R^{-b} \qquad (16)$$

where Q is a dimensionless average overtopping rate per meter, R is a dimensionless freeboard, and a and b are coefficients related to structure geometry.

Table 3. Summary of CEM Overtopping Guidance

EM 1110-2-1100 (Part VI)
Proposed Publishing Date: 30 Apr 03

Table VI-5-7
Models for Average Overtopping Discharge Formulae

Authors	Structures	Overtopping model	Dimensionless discharge Q	Dimensionless freeboard R
Owen (1980,1982)	Impermeable smooth, rough, straight and bermed slopes	$Q = a\ exp(-b\,R)$	$\frac{q}{g\,H_s\,T_{om}}$	$\frac{R_c}{H_s}\left(\frac{s_{om}}{2\,\pi}\right)^{0.5}\frac{1}{\gamma}$
Bradbury and Allsop (1988)	Rock armored impermeable slopes with crown walls	$Q = a\ R^{-b}$	$\frac{q}{g\,H_s\,T_{om}}$	$\left(\frac{R_c}{H_s}\right)^2\left(\frac{s_{om}}{2\,\pi}\right)^{0.5}$
Aminti and Franco (1988)	Rock, cube, and Tetrapod double layer armor on rather impermeable slopes with crown walls, (single sea state)	$Q = a\ R^{-b}$	$\frac{q}{g\,H_s\,T_{om}}$	$\left(\frac{R_c}{H_s}\right)^2\left(\frac{s_{om}}{2\,\pi}\right)^{0.5}$
Ahrens and Heimbaugh (1988b)	7 different seawall/revetment designs	$Q = a\,exp(-b\,R)$	$\frac{q}{\sqrt{g\,H_s^3}}$	$\frac{R_c}{(H_s^2\,L_{op})^{1/3}}$
Pedersen and Burcharth (1992)	Rock armored rather impermeable slopes with crown walls	$Q = a\,R$	$\frac{q\,T_{om}}{L_{om}^2}$	$\frac{H_s}{R_c}$
van der Meer and Janssen (1995)	Impermeable smooth, rough straight and bermed slopes	$Q = a\,exp(-b\,R)$	$\frac{q}{\sqrt{g\,H_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}}$ for $\xi_{op} < 2$	$\frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{\tan\alpha}\frac{1}{\gamma}$ for $\xi_{op} < 2$
			$\frac{q}{\sqrt{g\,H_s^3}}$ for $\xi_{op} > 2$	$\frac{R_c}{H_s}\frac{1}{\gamma}$ for $\xi_{op} > 2$
Franco, de Gerloni, and van der Meer (1994)	Vertical wall breakwater with and without perforated front	$Q = a\,exp(-b\,R)$	$\frac{q}{\sqrt{g\,H_s^3}}$	$\frac{R_c}{H_s}\frac{1}{\gamma}$
Pedersen (1996)	Rock armored permeable slopes with crown walls	$Q = R$	$\frac{q\,T_{om}}{L_{om}^2}$	$3.2\cdot10^{-5}\,\frac{H_s^5\,\tan\alpha}{R_c^3\,A_c\cdot B}$

Source: USACE, 2003

The method by Owen (1980), adopted by FEMA (2003), is still presented in the CEM for runup on impermeable, smooth and rough bermed slopes. The work of Goda (1985), also referenced by FEMA (2003), is mentioned in the CEM. The CEM provides a method to estimate the overtopping volume of an individual wave. (The average overtopping rate provides no information on the overtopping of single waves, yet most overtopping damage occurs with single large waves.)

Wallingford (W178 Method)

The HR Wallingford Ltd. (1999) report summarizes the current United Kingdom methodology for determining wave overtopping for a variety of structures. The report is available at <http://www.environment-agency.gov.uk/commondata/105385/w178.pdf>.

Design curves are based on small-scale (1:40, 1:20) laboratory tests performed on a variety of seawall configurations, beach slopes, and wave angles. Prototype measurements of overtopping have been made to validate the laboratory tests, but the results are seen as conservative, when compared with the Delft (TAW) guidance. Guidance was developed with pseudo-random waves described by a JONSWAP spectrum (the spectrum does not include a swell component). Therefore, its application is most applicable to unimodal, narrow banded seas (i.e., storm seas with a single spectral peak).

The guidance is summarized in Table 4. The required inputs are structure geometry and characteristics, significant wave height and mean wave period at the toe of the structure, height of the crest of the wall above the stillwater level, angle of wave attack, etc. (Note: The input stillwater level does not include wave setup.)

The procedures allow calculation of the mean overtopping discharge, as well as the maximum individual wave overtopping discharge (using a method similar to CEM).

A discussion of tolerable discharges (for seawalls, pedestrians, vehicles, buildings) is also presented; this appears to have been adopted by the CEM.

Technical Advisory Committee for Water Retaining Structures)

TAW (2002) provides revised procedures for calculating overtopping discharge for breaking and nonbreaking waves. This guidance supersedes the older guidance (which is included in the CEM). Higher-than-average overtopping discharge levels are recommended for structure design (see Figure 12). Procedures for computing overtopping volumes per wave are provided.

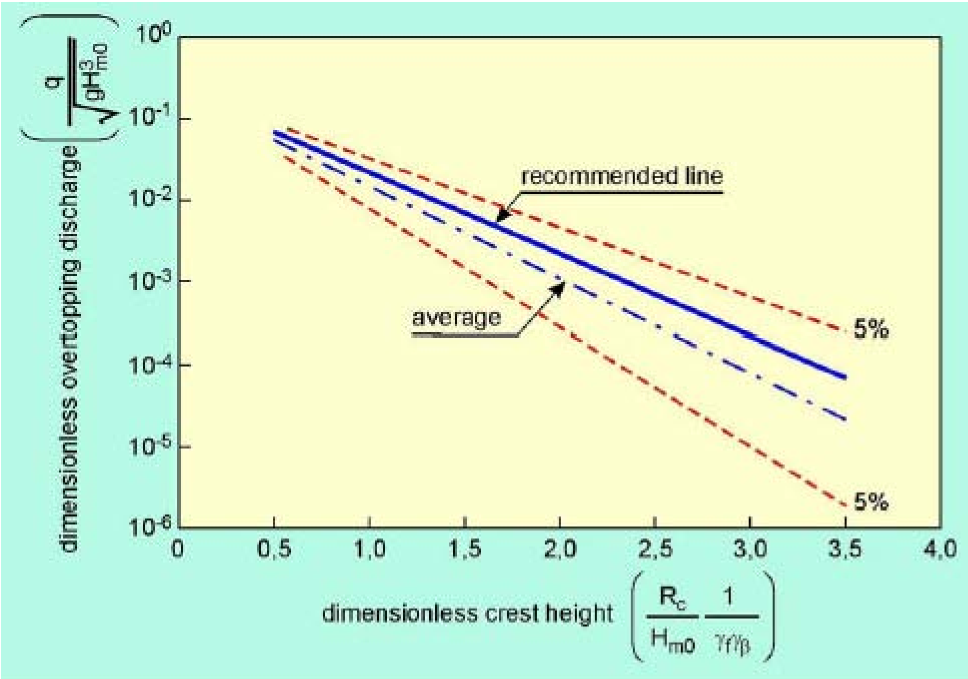
Numerical Models

See "Wave Runup" in Section 2.2.4. Other runup/overtopping models exist or are under development, such as the OTT-1d and OTT-2d models, which are part of HR Wallingford Ltd.'s ANEMONE (Advanced Non-linear Engineering suite of Models for the Nearshore Environment). More information has been requested.

Table 4. Methods to Determine Wave-Overtopping Discharges for Various Structures

	Mean overtopping discharges Derived from small-scale laboratory experiments			Maximum individual wave discharge $V_{max}=a(\ln(N_{ow}))^{1/b}$	Other
	Normal wave attack	Angled wave attack	Return walls	Number of overtopping waves	
Smooth impermeable simple and bermed slopes	Owen (1980): $Q^* = A \exp(-BR^*)$ where Q^* is dimensionless overtopping rate and R^* is dimensionless freeboard. *A* and *B* are empirically derived coefficients.	Banyard and Herbert (1995): $O_r = f(\beta)$ where O_r is the ratio of overtopping at a given wave attack angle, β, compared to that under normal wave attack.	Owen and Steele (1991) is used to determine a discharge factor, D_f, which is the ratio of overtopping for a return wall that without a return wall. Dependent mainly upon the height of the wall and the incident overtopping rate. Banyard and Herbert (1995) provide a method for calculating D_f with angled wave attack.	Owen (1982): $N_{ow}/N_w=\exp(-C(R^*/r)^2)$ where *C* is an empirical coefficient dependent upon slope. Determined by the number of waves with calculated runup greater than the crest height. For slopes between 1:1 and 1:4. Advice is given for angled wave attack.	Advice is given in HR Wallingford Ltd. (1999) for estimating rates for composite slopes and multiple berms
Rough and armored slopes	Owen (1980): $Q^* = A \exp(-BR^*/r)$ where *r* is a roughness coefficient based upon the relative runup performance of the different surfaces (e.g., smooth concrete, single layer armor unit, one layer of rock with impermeable core, two layers of rock).	Advice for angled wave attack is to use the method of Banyard and Herbert (1995) as for smooth slopes.	Bradbury and Allsop (1988), reanalyzed in HR Wallingford Ltd. (1999), used to determine D_f. Banyard and Herbert (1995) provide a method for calculating D_f with angled wave attack.	Owen (1982): $N_{ow}/N_w=\exp(-CR^{*2})$ for slopes between 1:1 and 1:2. Advice is given for angled wave attack.	Permeable crest berms are accounted for with a reduction factor, C_r, based on the crest width.
Plain vertical walls	Allsop et al. (1995): Functions provided for calculating overtopping for both impacting and reflecting waves.	Franco (1996) gives O_r function for reflecting waves only.		HR Wallingford Ltd. (1999) gives functions to determine N_{ow} for impacting and reflecting waves. Advice is given for angled wave attack	
Composite vertical walls (sitting on a mound)	Allsop et al. (1995): Overtopping is dependent upon whether the mound is large or small compared to the depth of water.	Advice for angled wave attack is to use the method of Franco (1996) as for plain vertical walls.			

Source: HR Wallingford Ltd., 1999



Source: TAW, 2002

Figure 12. Maximum wave overtopping by nonbreaking waves.

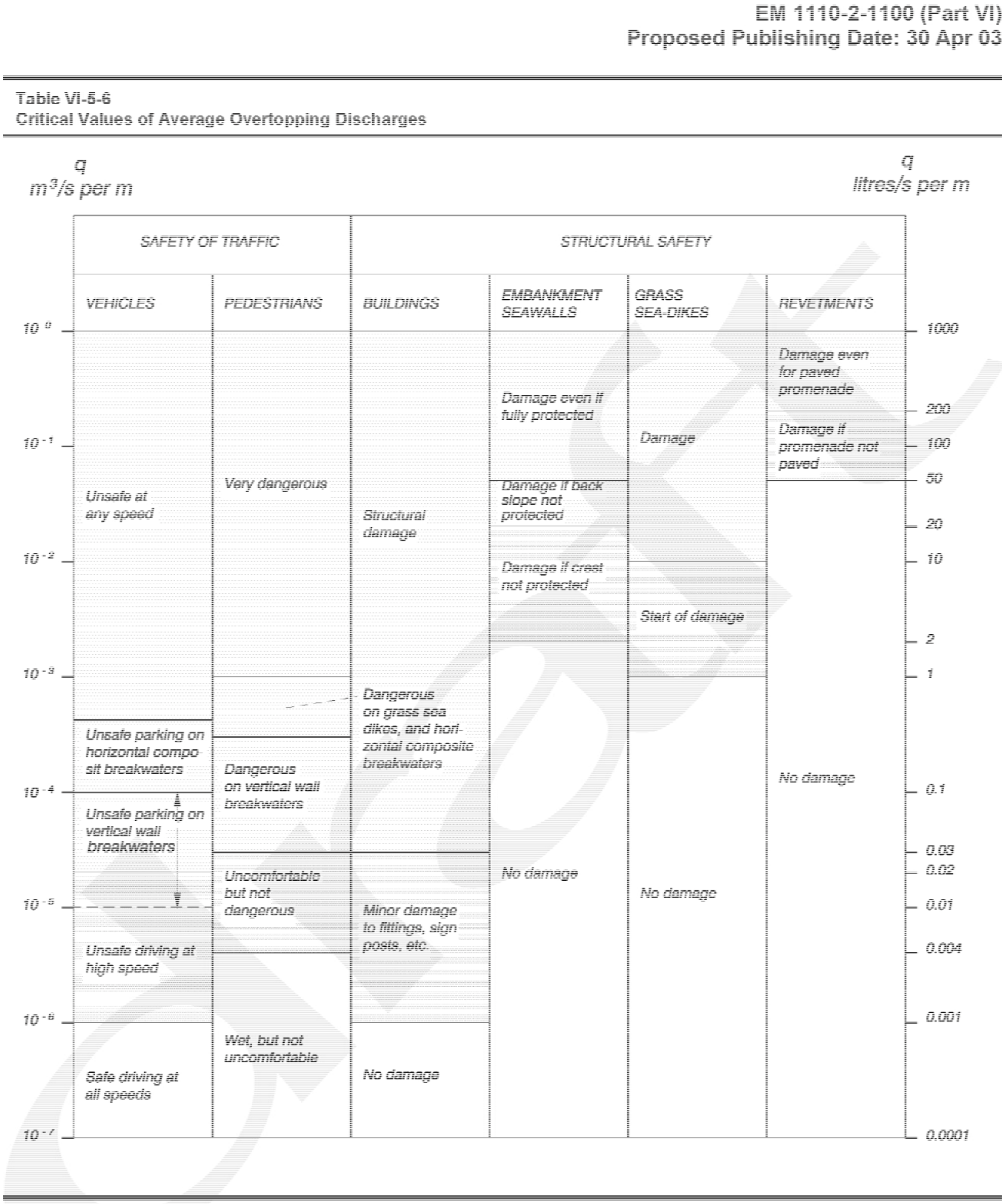
"Acceptable" Overtopping

FEMA (2003) maps flood hazard zones landward of an overtopped barrier using the mean overtopping rate—the higher the rate, the higher the flood elevation/depth and the more hazardous the zone designation (see Table 2). The source of the overtopping rates separating the zones and depths is unknown.

Several authors and studies have attempted to define "tolerable" or "critical" rates of overtopping, which will vary with the object being affected by the overtopping, the distance from the overtopped barrier, etc. The CEM has assembled much of this information into a single figure, which is reproduced here as Figure 13. A more recent study (Geeraerts et al., 2003) provides field measurements of overtopping velocities and overtopping forces (on vertical walls, window glass, people [using dummies], and pipelines). These data should be reviewed to evaluate whether FEMA's overtopping rates are appropriate. (The building/wall/glass data should be especially pertinent for NFIP mapping purposes.) This work should be coordinated with the Hazard Zone Study Group.

Tsunami Overtopping

FEMA (2003) does not contain any guidance for estimating overtopping of coastal structures by tsunamis. A cursory review of the literature located a USACE document, *Tsunami Engineering* (Camfield, 1980), which contains two empirical methods for estimating tsunami overtopping of



Source: USACE, 2003

Figure 13. Critical values of average overtopping values.

seawalls, the Kaplan (1955) method and the Wiegel (1970) method. These empirical methods are described below.

Kaplan (1955) Method

Under this method,

$$V = 21.65\left(Kh_s - h_w\right)^3 \big/ K^2 h_s \tag{17}$$

where: V = volume of overtopping the wall in cubic meters per meter (m^3/m) or cubic feet per foot (ft^3/ft);

h_s = wave height at the shoreline in meters or feet;

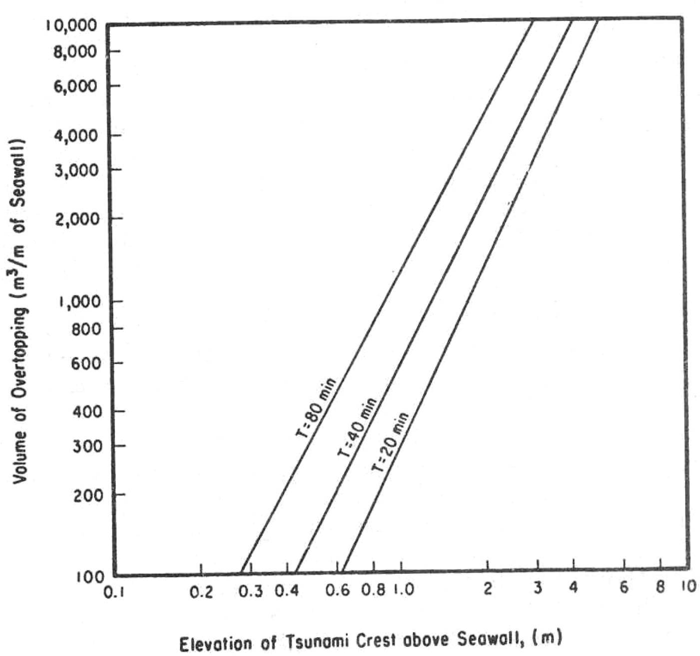
h_w = height of wall in meters or feet; and

K = R/h_s where R is the wall height required to prevent overtopping.

Wiegel (1970) method

Wiegel gives a relationship for estimating tsunami overtopping volumes that includes tsunami period and time dependence. The results of this relationship are summarized in Figure 14.

A more thorough literature search and coordination with the Tsunami Study Group should be undertaken for this topic.



Source: Wiegel, 1970

Figure 14. Tsunami overtopping volume at a seawall.

3.2.5 Recommendations

Recommendations for Topic 13 are as follows (see Table 5, at the conclusion of this report):

1. Review available overtopping methods and models, and determine appropriate procedure(s) for calculating the mean overtopping discharge, including those over low-profile natural barriers, dune remnants, revetments, and vertical walls.

2. Evaluate procedures for calculating overtopping onto low bluffs with gently sloping, flat, or adverse slopes. Evaluate methods for determining ponding landward of overtopped barriers.

3. Review the current literature on "acceptable" overtopping, and work with the Hazard Zone Study Group to evaluate the overtopping rates that FEMA (2003) uses to identify flood hazard zones landward of an overtopped barrier.

4. Evaluate FEMA's current guidance, which limits the runup elevation to 3 feet above a barrier's crest elevation.

5. Coordinate work with the Tsunami Study Group.

3.2.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

3.2.7 Related Available and Important Topics

Available and Important Topics related to Topic 13 are listed in Table 5, at the conclusion of this report.

4 IMPORTANT TOPICS

4.1 TOPIC 14: REVIEW AVAILABLE METHODS AND DEVELOP GUIDANCE FOR WAVECAST DEBRIS

4.1.1 Description of the Topic and Suggested Improvement

The existing *G&S* do not provide any guidance for estimating the hazards caused by wavecast debris, e.g., waterborne logs and wave-sprayed stone. Some guidance on estimating debris characteristics and its effects (on both upland structures and shore protection structures) may exist in the literature, however, and this should be reviewed. For example:

- Knowles and Terich (1977) described the hazards associated with logs and debris at Sandy Point, Whatcom County, WA (see Figure 15).

Source: Knowles and Terich, 1977

Figure 15. March 1975 storm, drift logs driven into coastal houses at Sandy Point, Washington.

- Edens (pers. comm., 1978) acknowledged the relative importance of floodborne debris in a memorandum that outlined a coastal flood study methodology for Puget Sound. The memorandum stated, "There was a general agreement…that damage due to water-borne logs and other forms of debris is the greatest danger to the destruction of property associated with the breaking wave of the magnitude that is experienced in Puget Sound."

- Kriebel, Buss, and Rogers (2000) reviewed the literature on impact loads caused by floodborne debris, including riverine debris, hurricane debris, tsunamis, and West Coast log debris. The report was background for a study on floodborne debris impacts, which helped plan the laboratory study of Haehnel and Daly (2002), and informed floodborne debris impact load calculations in *ASCE 7-02* (ASCE, 2002).

- Allan and Komar (2002) documented the inland penetration of small stone from a revetment at Cape Lookout State Park (see Figure 16).

Anticipated revisions to the *G&S* will include more discussion and guidance on defining hazards to insured property from wavecast debris, and will provide Mapping Partners with more information on how drift logs can contribute to the failure of coastal structures and shoreline erosion. Work on this topic will be coordinated with the Sheltered Water, Hazard Zone, Coastal Structures, Event Based Erosion, and Tsunami Study Groups.

Haehnel and Daly (2002) used a laboratory flume with logs (ranging in size from 380 pounds to 730 pounds) and traveling at speeds up to 4 feet per second to measure debris impact loads, and to develop a method for estimating floodborne debris impact loads.

Source: Allan and Komar, 2002

Figure 16. Inland Penetration of small revetment stone during 1998-1999 winter, Cape Lookout State Park.

4.1.2 Description of Procedures in the Existing Guidelines

Current coastal flood study guidance from FEMA (2003) indicates that the landward extent of the VE Zone is established at a point where the runup depth drops below 3 feet (see Figure 5). The VE zone may be extended inland by 30 feet if overtopping rates exceed 1.0 cfs/foot (see Table 2).

Some accounts of flooding at flood insurance study communities along Puget Sound indicate that flooding, overtopping, and/or ponding can extend more than 30 feet inland at many locations, even during storms much less severe than the base flood (e.g., Phillip Williams & Associates, 2002). Thus, the current guidance may not capture all of those coastal areas subject to high hazards during the base flood.

4.1.3 Application of Existing Guidelines to Topic—History and/or Implications for the NFIP

See Section 2.1.3.

4.1.4 Alternatives for Improvement

Given the lack of guidance for determining hazards from wavecast debris, FIS contractors have had to develop methods to address these hazards during past flood insurance studies in FEMA Region X. Among these studies have been a 1989 sheltered water flood study in the harbor of Port Angeles, Washington, and the Sandy Point and Birch Bay studies in Whatcom County, Washington, in 2001 and 2002. More details on these studies are provided in Section 2.g. of the Sheltered Water Focused Study report.

The resulting methods represent simple efforts that were developed, applied, and approved by FEMA within existing flood study budget and schedule limitations at the time. These methods should be reviewed, refined, and considered for adoption as guidelines for defining flood hazards from wavecast debris.

4.1.5 Recommendations

Recommendations for Topic 14 are as follows (see Table 5, at the conclusion of this report):

1. Review the current literature and quantify the significance of coastal flood damages from drift logs and wave-sprayed stone.

2. Review past flood insurance studies that have resulted in methods for defining flood hazards from wavecast debris, and refine these methods for possible incorporation into the *G&S*.

3. Incorporate results into flood zone mapping. Do not attempt to map debris specifically; map the water that carries the debris. Coordinate work with other Focused Study Groups as appropriate.

4.1.6 Preliminary Time and Cost Estimate for Guideline Improvement Preparation

Table 6 at the end of this report presents estimates of times required to accomplish the tasks in this topic.

4.1.7 Related Available and Important Topics

Available and Important Topics related to Topic 14 are listed in Table 5, at the conclusion of this report.

5 ADDITIONAL OBSERVATIONS

5.1 SEPARATING WAVE SETUP FROM WAVE RUNUP

FEMA (2003) methods currently add wave setup to the 1% water level for wave height (WHAFIS) calculations, but do not do so for wave runup calculations (also note that FEMA's event-based erosion calculations use the stillwater elevation without setup). This inconsistency results from the underlying data and methods used by FEMA to develop its wave height and wave runup procedures. In effect, FEMA has determined that its computed wave runup already includes a wave setup component.

In Phase 2 of the current project, Pacific Coast methods will be developed and wave setup calculations will be reconsidered. The issue of how wave setup is treated relative to wave runup, wave heights, and event-based erosion must be resolved in a consistent and sound manner during Phase 2.

5.2 IMPLICATIONS OF USING THE RESPONSE METHOD

The Event Selection Method is relatively easy (and appropriate) to employ along the Atlantic and Gulf of Mexico—there is a high correlation between storm surge and wave conditions, and combining the 1% stillwater elevation with the 1% wave conditions is appropriate in most situations.

In general, use of this simple procedure is not valid along the Pacific Coast (and along many sheltered shorelines on all coasts) where water levels and wave conditions are not highly correlated. In these cases, either the Mapping Partner must identify other water level–wave condition combinations (which can be difficult and subject to error), or resort to a statistical analysis of response. The Response Method may be preferable for FISs.

However, use of the response method to determine the 1% flood elevation (or 1% profile geometry) will likely introduce extreme complexity into the flood map revision process. Coastal map revision requests are usually submitted to and processed by FEMA based on a defined event and improved (or altered) topography. Methods should be sought to avoid requiring all map revision requestors to also use the Response Method. One approach might be to back-calculate a 1% event (or events) based on the results of the Response Method, and allow revisions to be based on the event(s). Obviously, the details need to be worked out and this procedure needs to be tested during Phase 2.

5.3 USE OF 2-D MODELS

Procedures currently approved by FEMA for use in coastal FISs include both simple 1-D approaches and more complex 2-D models. At present, the only approved wave runup procedures are 1-D procedures (e.g., RUNUP 2.0, ACES, CHAMP, GLWRM). 2-D models have been approved for storm surge calculations (e.g., RMA2, MIKE 21, FLOW2D) and for wave height modeling (e.g., RCPWAVE, MIKE 21 offshore and nearshore wave models), although use of the 1-D WHAFIS methodology is dominant for overland wave height calculations.

FEMA's Approved Models Committee has and will continue to evaluate other 2-D models for use by Mapping Partners. Undoubtedly, more and more 2-D models will be approved for FISs, including models that calculate wave runup and overland wave heights. The migration away from the transect approach will continue. Phase 2 of the current study should consider how 2-D models, especially those on the approved models list (<http://www.fema.gov/mit/tsd/en_coast.shtm>), can be incorporated into Pacific flood studies.

6 SUMMARY

Focused Study findings and recommendations for runup and overtopping are summarized in Table 5 below.

Table 5. Summary of Findings and Recommendations for Runup and Overtopping

Topic Number	Topic	Coastal Area	Priority Class	Availability/ Adequacy	Recommended Approach	Related Topics
12	Runup and Overtopping	AC	H *(C)*	MIN	1. Revise guidance to call for runup analyses for sandy beach, small dune shore type. 2. Review runup distributions for beaches and structures during El Niño, coastal storm, and hurricane conditions; review runup damages; evaluate use of $R_{50\%}$ and select alternate $R_{x\%}$ value (probably between $R_{33\%}$ and $R_{10\%}$) if $R_{50\%}$ understates the hazard. 3. Tsunami runup should be treated by runup procedures developed specifically for tsunami events (rely on Tsunami Study Group). 4. Investigate feasibility of interim procedure for modifying the results of RUNUP 2.0.	11 16
		GC	H *(C)*	MIN		
		PC	C	MIN		
		SW	C	MIN		
11	Runup and Overtopping	AC	H *(A)*	Y	1. Evaluate expansion of “Oregon-type” and “CDIP-type” methods as interim Pacific runup method 2. Develop test scenarios for side-by-side comparisons of existing runup methods, models (give priority to Pacific and New England scenarios) 3. Perform comparisons and sensitivity tests, eliminate methods, models; identify appropriate runup methods, models by location, morphology and hydraulic conditions	4, 5 7, 8 12 16 44-48 49
		GC	H *(A)*	Y		
		PC	A *(C)*	MAJ		
		SW	A	Y		
49	Runup and Overtopping	AC	A	Y	Evaluate with other runup methods and models in Topic 11 work.	11
		GC	A	Y		
		PC	A	Y		
		SW	A	Y		

Table 5. Summary of Findings and Recommendations for Runup and Overtopping

Topic Number	Topic	Coastal Area	Priority Class	Availability/ Adequacy	Recommended Approach	Related Topics
13	Runup and Overtopping	AC	NE *(A)*	Y	1. Evaluate existing methods and models for calculating mean overtopping rates. 2. Determine appropriate procedures for calculating overtopping at structures, remnant dunes, low-profile beaches and barriers. 3. Evaluate procedures for calculating overtopping at low bluffs. 4. Review literature for data on "acceptable" overtopping rates, revise landward flood hazard zones. 5. Review FEMA practice to limit runup elevations to 3 feet above barrier crests.	11 12 14
		GC	NE *(A)*	Y		
		PC	A	Y		
		SW	A	Y		
14	Runup and Overtopping	AC	H	PRODAT	1. Review the literature and quantify the significance of coastal flood damages from drift logs and wave-sprayed stone. 2. Review past flood insurance studies that have resulted in methods for defining flood hazards from wavecast debris, and refine methods where appropriate. Incorporate results into flood hazard zone mapping, but do not attempt to specifically map debris (map the water that carries debris, not debris itself).	6 13 18 20 22
		GC	H	PRODAT		
		PC	I	PRODAT		
		SW	I	PRODAT		

Key:
Coastal Area
AC = Atlantic Coast; GC = Gulf Coast; PC = Pacific Coast; SW = Sheltered Waters
Priority Class
C = critical; A = available; I = important; H = helpful
(Recommend priority italicized if focused study recommended a change in priority class)
Availability/Adequacy
"Critical" Items: MIN = needed revisions are relatively minor; MAJ = needed revisions are major
"Available" Items: Y = availability confirmed; N = data or methods are not readily available
"Important" Items: PRO = procedures or methods must be developed; DAT = new data are required;
PRODAT = both new procedures and data are required

Table 6. Time Estimates for Runup and Overtopping Topics		
Topic Number	Topic	Time (person months)
12	**Review Appropriateness of Using Mean vs. Higher Values for Runup and Overtopping**	
	Make final recommendation regarding appropriate $R_{X\%}$ value for use in wave runup calculations; coordinate with Tsunami Study Group (topic 16)	1
	Develop interim procedure for modifying the results of RUNUP 2.0 outputs (until RUNUP 2.0 is modified or replaced)	0.5
	TOTAL	1.5
11 / 49	**Review Runup Methods and Programs; Provide Explicit Guidance on Where Each Should Be Applied / Review WRUPTM (Available Wave Runup Program)**	
	Evaluate Oregon and CDIP methods for use as interim runup methods	1
	Develop test scenarios for side-by-side comparisons of existing runup methods, models (give priority to Pacific and New England scenarios); include sandy beach small dune scenario	1
	Perform comparisons, eliminate methods, models; identify appropriate runup methods, models by location, morphology, hydraulics; consider input condition uncertainties	2
	Coordinate work with Wave Setup and Wave Transformation groups – make sure required wave runup inputs are available and methods are consistent	1
	TOTAL	5
13	**Develop Improved Guidance for Determining and Mapping Overtopping Volumes**	
	Review available overtopping methods and models, and determine appropriate procedure(s) for calculating the mean overtopping discharge	0.7
	Evaluate FEMA's current guidance which limits the runup elevation to 3 feet above a barrier's crest elevation	0.1
	Evaluate procedures for calculating overtopping onto low bluffs with gently sloping, flat or adverse slopes. Evaluate methods for determining ponding landward of overtopped barriers	1
	Review the current literature on "acceptable" overtopping, and coordinate with the Hazard Zone Study Group	0.2
	TOTAL	2
14	**Review Available Methods and Develop Guidance for Wavecast Debris**	
	Review the current literature and quantify the significance of coastal flood damages from drift logs and wave-sprayed stone	0.75
	Review past flood insurance studies that have resulted in methods for defining flood hazards from wave-cast debris, and refine these methods for possible incorporation into the *G&S*	0.75
	TOTAL	1.5

7 REFERENCES

7.1 PRINTED REFERENCES

Ahrens, J.P. 1977. *Prediction of Irregular Wave Overtopping*. CETA 77-7. U.S. Army Corps of Engineers Coastal Engineering Research Center. Fort Belvoir, VA.

Ahrens, J.P., and M.S. Heimbaugh. 1988. *Approximate Upper Limit of Irregular Wave Runup on Riprap*. Technical Report CERC-88-5. U.S. Army Corps of Engineers. Vicksburg, MS.

Ahrens, J.P., and B.L. McCartney. 1975. Wave Period Effect on the Stability of Riprap. Pages 1019-1034 in *Proceedings of Civil Engineering in the Oceans III*.

Ahrens, J.P., and M.F. Titus. 1985. Wave Runup Formulas for Smooth Slopes. *Journal of the Waterway, Port, Coastal, and Ocean Engineering* 111(1):128-133. American Society of Civil Engineers.

Allan, J.C., and P.D. Komar. 2002. A Dynamic Revetment and Artificial Dune for Shore Protection. In J. M. Smith (ed.), *Proceedings of the 28th International Conference on Coastal Engineering*. American Society of Civil Engineers.

American Society of Civil Engineers. 2002. *Minimum Design Loads on Buildings and Other Structures*. SEI/ASCE 7-02.

Battjes, J.A. 1974. *Computation of Set-Up, Longshore Currents, Runup and Overtopping Due to Wind Generated Waves*. Ph.D. Dissertation. Technische Hogeschool, Delft, Netherlands.

Burcharth, H.F., and S.A. Hughes. 2003 (April). *Fundamentals of Design*. Chapter VI-5 in S. A. Hughes (ed.), *Coastal Engineering Manual, Part VI, Design of Coastal Project Elements*. Engineer Manual 1110-2-1100. U.S. Army Corps of Engineers. Washington, D.C.

Camfield, F.E. 1980. *Tsunami Engineering*. Special Report No. 6. U.S. Army Corps of Engineers Coastal Engineering Research Center.

Coastal Data Information Program (CDIP). 2004. Potential Flooding Index. Available <http://cdip.ucsd.edu/elnino_htmls/homepage.shtml>. Accessed February 9, 2004.

Cox, J., and J. Machemehl. 1986. Overland Bore Propagation Due to an Overtopping Wave. Technical Note. *Journal of Waterway, Port, Coastal and Ocean Engineering* 112 (1):161-163. American Society of Civil Engineers.

Dewberry & Davis. 1991 (April). *Investigation and Improvement of Capabilities for the FEMA Wave Runup Model (Technical Documentation for Runup Program 2.0).*

Douglass, S.L. 1986. *Review and Comparison of Methods for Estimating Irregular Wave Overtopping Rates*. Technical Report CERC 86-12. U.S. Army Corps of Engineers Waterways Experiment Station. Vicksburg, MS.

Federal Emergency Management Agency (FEMA). 2003 (April). *Guidelines and Specifications for Flood Hazard Mapping Partners*. Appendix D: Guidance for Coastal Flooding Analyses and Mapping.

Geeraerts, J., P. Troch, J. De Rouck, L. Van Damme, and T. Pullen. 2003. *Hazards Resulting from Wave Overtopping—Full Scale Measurements.* Preprint, Third International Coastal Structures Conference. American Society of Civil Engineers.

Goda, Y. 1985. *Random Seas and Design of Maritime Structures*. University of Tokyo Press. Tokyo, Japan.

Haehnel, R., and S. Daly. 2002. *Maximum Impact Force of Woody Debris on Floodplain Structures.* Technical Report ERDC/CRREL TR-02-2. U.S. Army Cold Regions Research and Engineering Laboratory. Hanover, NH.

Hallermeier, R.J., K.B. Nosek, and C.J. Andrassy. 1990. Evaluation of Numerical Model for Wave Runup Elevations. Pages 41-54 in B. L. Edge (ed.), *Proceedings of the 22nd International Conference on Coastal Engineering.* American Society of Civil Engineers.

Hughes, S.A. 2003a. *Wave Momentum Flux Parameter for Coastal Structure Design*. ERDC/CHL CEH Technical Note III-67. U.S. Army Engineer Research and Development Center. Vicksburg, MS.

———. 2003b. *Estimating Irregular Wave Runup on Smooth, Impermeable Slopes*. ERDC/CHL CEH Technical Note III-68. U.S. Army Engineer Research and Development Center. Vicksburg, MS.

Hunt, I.A. 1959. Design of Seawalls and Breakwaters. *Journal of Waterways and Harbor Engineering* 85(WW3):123-152. American Society of Civil Engineers.

Kaplan, K. 1955. *Generalized Laboratory Study of Tsunami Run-up.* TM-60. U.S. Army Beach Erosion Board.

Knowles, S., and T. Terich. 1977. Perceptions of Beach Erosion Hazards and Sandy Point, Washington. *Shore and Beach* 45(3).

Kobayashi, N. 1999. Wave Runup and Overtopping on Beaches and Coastal Structures. Pages 95-154 in P. L-F. Liu (ed.), *Advances in Coastal and Ocean Engineering* Vol. 5. World Scientific.

Kriebel, D., L. Buss, and S.M. Rogers, Jr. 2000. *Impact Loads from Floodborne Debris*. Phase I Project Report prepared for the American Society of Civil Engineers by Academy Engineering LLC. Millersville, MD.

Mase, H. 1989. Random Wave Runup Height on Gentle Slopes. *Journal of Waterway, Port, Coastal, and Ocean Engineering* 115(5):649-661. American Society of Civil Engineers.

National Academy of Sciences. 1977. *Methodology for Calculating Wave Action Effects Associated with Storm Surges.* Washington, D.C.

Noble Software, Inc. (Noble). 1984. WRUP™ Wave Runup Software Program, User Manual Version 1.0.

Owen, M.W. 1980. *Design of Seawalls Allowing for Wave Overtopping*. Report Ex. 924. Hydraulics Research Station, Wallingford, United Kingdom.

Phillip Williams & Associates. 2002. *Coastal Hydraulics Phase Report, Sandy Point, Whatcom County, Washington, Coastal Flood Insurance Study.*

Ruggiero, P., P.D. Komar, W.G. McDougal, J.J. Marra, and R.A. Beach. 2001. Wave Runup, Extreme Water Levels and the Erosion of Properties Backing Beaches. *Journal of Coastal Research* 17(2):407-419.

Savage, R.P. 1958. Wave Runup on Roughened and Permeable Slopes. *Journal of Waterways and Harbor Division* 84(WW3):1-38. American Society of Civil Engineers.

Saville, T., Jr. 1955. *Laboratory Data on Wave Runup and Overtopping on Shore Structures*. Beach Erosion Board TM-64. U.S. Army Corps of Engineers. Washington, DC.

———. 1956. Wave Runup on Shore Structures. *Journal of Waterways and Harbor Division* 82(WW2):1-114. American Society of Civil Engineers.

———. 1958. Wave Runup on Composite Slopes. Pages 101-108 in *Proceedings of the 6th Coastal Engineering Conference held in Gainesville, FL*. Council of Wave Research, University of California, 1958.

Saville, T., Jr., and J.M. Caldwell. 1953. Experimental Study of Wave Overtopping on Shore Structures. In *Proceedings of the Minnesota International Hydraulics Convention*. IAHR.

Smith, J.M. 2003 (July). Surf Zone Hydrodynamics. Chapter II-4 in L. Vincent and Z. Demirbilek (eds.), *Coastal Engineering Manual, Part II, Hydrodynamics*. Engineer Manual 1110-2-1100. U.S. Army Corps of Engineers. Washington, DC.

Stoa, P.N. 1978. *Reanalysis of Wave Runup on Structures and Beaches*. Technical Paper 78-2. U.S. Army Corps of Engineers Coastal Engineering Research Center. Fort Belvoir, VA.

Stone & Webster Engineering Corporation. 1981. *Manual for Wave Runup Analysis, Coastal Flood Insurance Studies.* Boston, MA.

Taylor, R. B., E. Ozoy and T.J. Turco. 1980. *Wave Runup on Variable Beach Profiles*. in *Journal of Waterway, Port, Coastal, and Ocean Engineering*, 106(WW2):169–182. American Society of Civil Engineers.

Technical Advisory Committee for Water Retaining Structures (TAW). 2002 (May). *Technical Report—Wave Run-up and Overtopping at Dikes.* The Netherlands.

U.S. Army Corps of Engineers (USACE). 1984. *Shore Protection Manual*. Washington, DC.

———. 1989. *Great Lakes Wave Runup Methodology Study*. Detroit District. Detroit, MI.

———. 1992. *Automated Coastal Engineering System (v. 1.07d), Users Guide*. Compiled by D. A. Leenknecht, A. Szuwalski, and A. R. Sherlock.

———. 2002, 2003. *Coastal Engineering Manual*. Engineer Manual 1110-2-1100. Washington, DC.

van der Meer, J.W., and C.J.M. Stam. 1992. Wave Run-up on Smooth and Rock Slopes of Coastal Structures. *Journal of Waterway, Port, Coastal and Ocean Engineering* 118(5):534-550. American Society of Civil Engineers.

van der Meer, J.W., and J.P.F.M. Janssen. 1995. *Wave Run-up and Overtopping at Dikes*. Pages 1-26 in American Society of Civil Engineers, *Wave Forces on Inclined and Vertical Wall Structures*.

H.R. Wallingford Ltd. 1999. *Wave Overtopping of Seawalls—Design and Assessment Manual*. R&D Technical Report W178.

Walton, T.L., J.P. Ahrens, C.L. Truitt, and R.G. Dean. 1989. *Criteria for Evaluating Coastal Flood-Protection Structures*. Technical Report CERC 89-15. U.S. Army Corps of Engineers Waterways Experiment Station. Vicksburg, MS.

Weggel, J.R. 1976. Wave Overtopping Equation. Pages 2737-2755 in *Proceedings of the 15th Coastal Engineering Conference*. American Society of Civil Engineers.

Wiegel, R. L. 1970. Tsunamis. Chapter 11 in *Earthquake Engineering*. Prentice-Hall.

7.2 PERSONAL COMMUNICATIONS

Edens, W. March 10, 1978—FEMA Region X Interagency Memorandum (coastal flooding study methodology for Northern Puget Sound) to Gregg Chappell.

Hatheway, Darryl. *S*enior coastal scientist**,** Dewberry & Davis. Fairfax, VA. December 3, 2003—personal communication with Chris Jones, runup-overtopping focus study leader, at NHC Workshop 1.

Seymour, Richard. Scripps Institute of Oceanography. December 32003—Workshop 1 presentation: CDIP potential flood index tool.